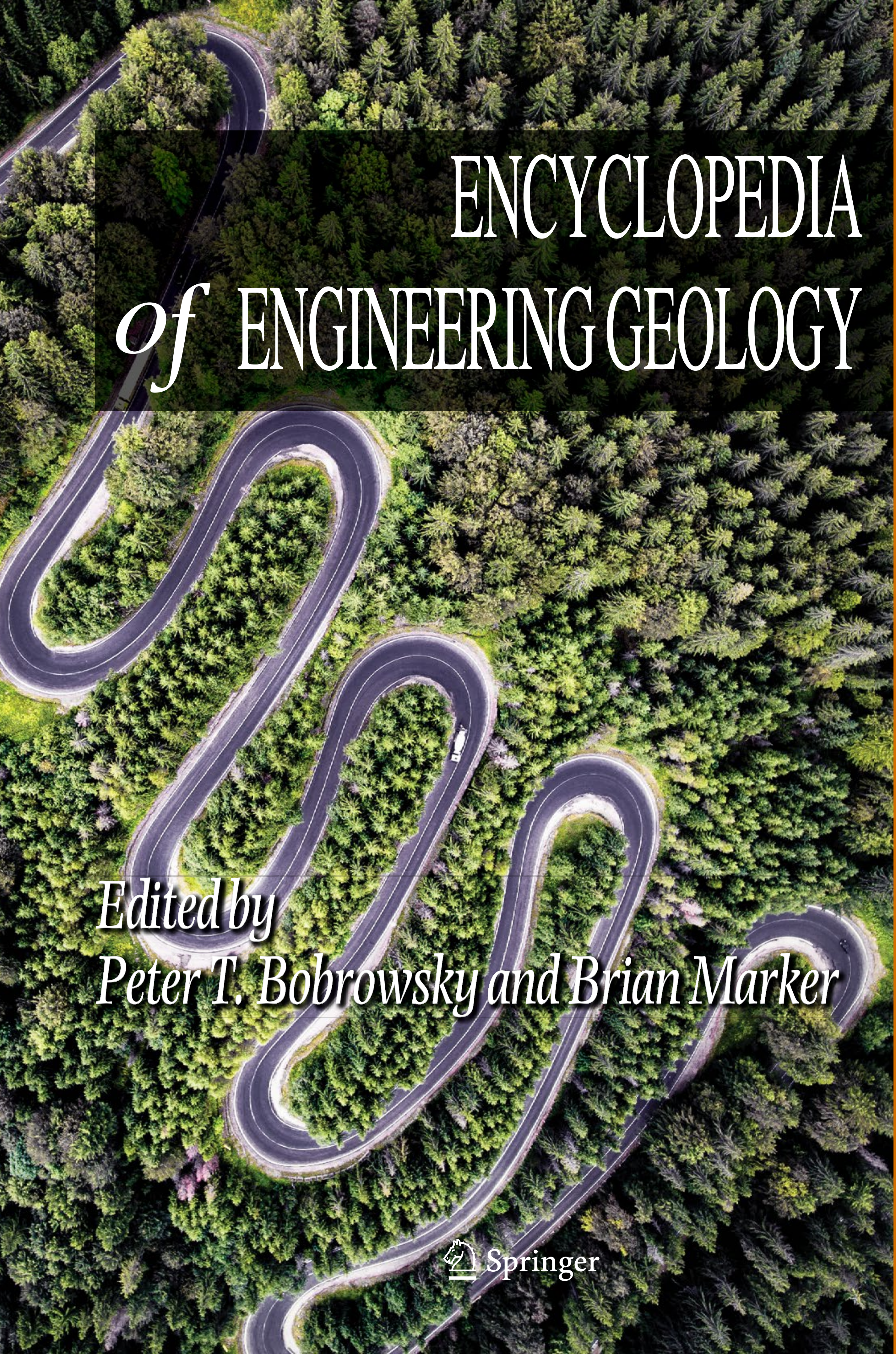
ENCYCLOPEDIA *of* ENGINEERING GEOLOGY



Edited by
Peter T. Bobrowsky and Brian Marker

Springer

ENCYCLOPEDIA *of* EARTH SCIENCES SERIES

ENCYCLOPEDIA *of* ENGINEERING GEOLOGY

Encyclopedia of Earth Sciences Series

ENCYCLOPEDIA OF ENGINEERING GEOLOGY

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Professor Charles W. Finkl has edited and/or contributed to more than eight volumes in the *Encyclopedia of Earth Sciences Series*. He has been the Executive Director of the Coastal Education and Research Foundation and Editor-in-Chief of the international *Journal of Coastal Research* for the past 35 years. He is also the Series Editor of the Coastal Research Library (Springer). In addition to these duties, he is Distinguished University Professor Emeritus at Florida Atlantic University (FAU) (Boca Raton, Florida). He is a graduate of Oregon State University (Corvallis) and the University of Western Australia (Perth). Work experience includes the International Nickel Company of Australia (Perth), Coastal Planning & Engineering (Boca Raton, Florida), and Technos Geophysical Consulting (Miami, Florida). He has published numerous peer-reviewed technical research papers and edited or co-edited and contributed to many books. Dr. Finkl is a Certified Professional Geological Scientist (Arvada, Colorado), a Certified Professional Soil Scientist (Madison, Wisconsin), a Certified Wetland Scientist (Lawrence, Kansas), and a Chartered Marine Scientist (London). Academically, he served as a Demonstrator at the University of Western Australia, Courtesy Professor at Florida International University (Miami), Program Professor and Director of the Institute of Coastal and Marine Studies at Nova Southeastern University (Port Everglades, Florida), and Full Professor at FAU. During his career, he acquired field experience in Australia; the Bahamas; Puerto Rico, Jamaica; Brazil; Papua New Guinea and other SW Pacific islands; southern Africa; Western Europe; and the Pacific Northwest, Midwest, and Southeast USA. Dr. Finkl is a member of several professional societies including the Geological Society of America; Soil Science Society of America; Institute of Marine Engineering, Science and Technology; and the Society of Wetland Specialists. He is a recipient of the International Beach Advocacy Award (Florida Shore & Beach Preservation Association), Certificate of George V. Chilingar Medal of Honor (Russian Academy of Natural Sciences), and Lifetime Commitment to Coastal Science Award (International Coastal Symposium).

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Cover photo: Cheia, Prahova county, Romania. The DN1A is an alternative transit road that goes from Ploiesti to Brasov (Transylvania), through the Carpathian Mountain pass Bratocea (at the peak altitude of 1.263 m). The road was built in the 60's to accommodate heavy transport trucks that are banned from using the main road - DN1 (E87) that goes through well-known touristic cities like Sinaia and Predeal. Image shot in July 2016 using a drone. Photo by Călin-Andrei Stan.

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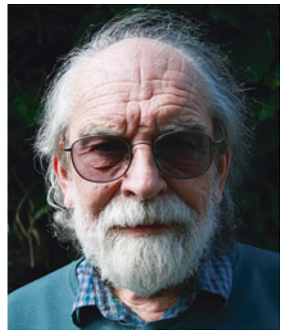

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Preface

Engineering geology provides a vital link between the established, traditional, and relevant disciplines that link its identity and name; that is, "engineering" and "geology." More recently, a variety of other fields of study are viewed as synonymous with engineering geology including but not limited to geotechnics, geological engineering, environmental engineering, environmental geology, applied geology, and so on. Although generally related, each of these "labels" do indeed represent different aspects of specialization that reflect their own sets of objectives, skills, and approaches, as adopted by the respective practitioners, in their goals of understanding and managing the complex relationship between geosciences, engineering, and society. In contrast, the practitioners of engineering geology rely fundamentally on the basic principles and tenets of geosciences. Secondly, they extract, compile, and assess critical knowledge and information about the Earth's properties and the behavior of nature. Next, they analyze the available evidence and then translate the results into a practical format used by others such as civil, structural, mining, construction, geotechnical, and environmental engineers, who are pursuing answers to their own questions and obligations.

Engineering geology deals with a broad but unique spectrum of topics: environments of deposition; types, compositions, and relevant physical and chemical properties of rocks and soils; the behavior of materials under changing climatic conditions; the influence of natural hazards on the natural and built environment; performance modeling; etc., on land and in the water, are a few of the many aspects that characterize the scope of the practice of engineering geology.

Although a long-standing discipline, engineering geology is currently experiencing a surge and peak in visibility, utility, relevance, and appreciation. A great many students are now pursuing this exciting field of study as both employment and research prospects for specialists grow considerably. In response to this positive trend, the need and critical timing for providing a compendium of formal definitions and descriptions in the format of the *Encyclopedia of Engineering Geology* was obvious.

The aim of this volume is to provide technical definitions of the most basic and common terms, principles, phrases, concepts, and issues influencing the field of engineering geology. This treatise defines and provides a ready source of explanation of those primary topics that engineering geologists often cross in their daily lives. This volume does not replace the technical rigidity and knowledge provided by a great many essential and useful educational and practical textbooks that students and professionals depend upon in their careers. Rather, this encyclopedia minimizes the efforts needed when searching for a clear and concise explanation and understanding of particular topics, and how those topics are related to other terms. Definitions include recent and relevant technical references for further information including well-illustrated graphic, detailed tabular, or striking visual images that also explain the term or concept in question.

The *Encyclopedia of Engineering Geology* comprises three categories of definitions. Our flexibility for the length of contributions was constrained by the practical limit to the length of the printed volume. This required us to assign individual terms to one of the three "length" categories. Topics that require a lengthy, elaborate, and well-illustrated explanation are included but are limited in their number. Broader issues and terms that need less explanation and illustration are more frequent, whereas those concepts and terms that are straight forward and easily explained in fewer words and illustration are the most numerous. Collectively, just under 300 topics are included herein for the sake of completeness. Each topic is accompanied by a useful cross-reference list of topics related to the term itself and defined elsewhere within the volume.

Individual editorial team members were assigned a number of topics from the total list that aligned most closely with their areas of expertise. Each editor was responsible for appointing qualified authors, managing the editorial process, and seeing the works to their completion.

The *Encyclopedia of Engineering Geology* provides a single platform for contributions from an international cross-section of some 200 specialists whose range of expertise competently addresses the nearly 300 topics that are most pertinent to the field of engineering geology. A common structure and format is used for each of the topics to ensure that readers did not experience changes in style that is common to some works written by multiple authors. The volume benefits considerably from an exceptional number of tables, illustrations, and numerous color images to ensure our aim of communication and education is satisfactorily achieved. This *Encyclopedia of Engineering Geology* provides a standard reference of technical reliability and should remain a primary choice for years to come.

The Editors
Peter Bobrowsky
Brian Marker
February 2018

Acknowledgments

The editorial team is extremely grateful to the many authors who graciously contributed their time, knowledge, and expertise to this volume. This effort, over several years, was efficiently and patiently handled by the managing staff at Springer International Publishing, in particular Petra van Steenbergen, Sylvia Blago, Kavipriya Venkataraman and Johanna Klute. Their dogged determination and relentless reminders provided the drive to complete the book in time for the 2018 IAEG Congress to meet our primary goal, when starting this project several years ago, to make this treatise accessible to a large gathering of engineering geologists under the umbrella of the IAEG.

Peter Bobrowsky expresses his sincere thanks to his wife Theresa for her understanding and tolerance during his execution of duties associated with the publication of this book. Special thanks also to Michiko and Toba, for providing a stress release during certain times in the production of this work.

Brian Marker also thanks his wife, Barbara, who was equally tolerant of the demands of this work.

We very much hope that this book delivers a ready and reliable source of information and facts for practicing engineering geologists, students, and all other related professional communities who rely on our profession for providing society with a healthier, safer, and more prosperous existence.

A

Abrasion

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Synonyms

Attrition; Erosion

Definition

Erosion of surfaces by impacts of harder particles on softer surfaces propelled by a dynamic medium.

In engineering geology, abrasion is significant in three main ways:

- Erosion of the Earth's surface
- Damage caused by abrasive minerals and rocks to machinery
- Selection of minerals that are suitable for use as industrial and domestic abrasives.

This is a complicated topic that is not yet fully resolved. Matters of specific interest include:

- The relative ease or difficulty of (resistance to) excavation, drilling, or cutting of rocks and soils
- Susceptibility to abrasion of surfaces including aggregates in highway pavements, machinery and natural stone used in buildings to abrasion.

Abrasion is often a two way process with the harder material affected less by wear than the softer material.

In general, abrasion increases with hardness, grain size, and angularity of mineral content; type of cementation; degree of alteration and discontinuities in the rock; forces of impacts; and the overall mechanical properties of the soil or rock mass (West 1989).

Erosion

Mineral particles carried by dynamic media cause abrasion of natural landforms and buildings and constructions. The media are:

- Wind – sediment carried by the wind sculpts distinctive landforms especially in arid areas but also during dry periods in less arid areas particularly where vegetation cover is absent.
- Sea water and lakes – breaking waves carry sand and coarser particles against coastlines causing undercutting of slopes and cliffs and consequent landslides. The resulting cliff retreat forms abrasion platforms adjacent to the shoreline and "wave cut notches" (Fig. 1). Similar processes occur in large lakes.
- Rivers – rivers carry sand and coarser particles, depending on the strength of the current, that cause channel scour and erosion of river banks with consequent changes in geomorphology and bank and slope stability.
- Ice – debris frozen into the beds and lateral margins of glaciers and ice sheets erode and smooth adjacent rock surfaces (Bennett and Glasser 2011).

These processes erode, smooth, and polish rock surfaces but also round and polish the debris that impact on those rocks.

Effects on Plant and Machinery

Plant, machinery, and other equipment are abraded during use causing wear and blunting and, therefore, contribute to

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Abrasion, Fig. 1 Stack showing a well-developed wave cut notch, Cathedral Beach, North Island, New Zealand (Photograph by Dr. Alan Thompson, Cuesta Consulting Ltd.)

Abrasion, Fig. 2 Wire cutting of blocks of marble, Favrizel, Portugal (Photograph by the author)

significant costs of repair, replacement, and delays. It is important to understand the potential for abrasion when costing engineering projects that involve drilling, tunneling, and mechanized excavation, as well as grading, cutting, and shaping of mineral products such as aggregates and dimension stones (Majeed and Abu Bakar 2016). Conversely, abrasion is used in extraction of building stone by wire cutting techniques in which an armored wire and abrasives are used to cut and shape blocks (Fig. 2).

Minerals Used Abrasives

A variety of minerals are used as abrasives. Selection of suitable pure or refinable materials depends on the minimum effective hardness. These range from less demanding uses (e.g., toothpastes and some domestic cleansers using calcite or feldspars) to demanding industrial uses (e.g., cutter heads or wires of strengthened metal and armored with very hard minerals) (West 1986).

Tests

A variety of abrasiveness tests have been developed for different purposes. Currently, there is no universally accepted test for soil abrasiveness but much research is in progress (Mirmehrabi et al. 2016). There is also an issue of the scale of tests. These can be at a real scale, in the tunnel, excavation, and borehole by examining resistance, but this procedure is expensive and too late to do more than adjust the pattern of works. It is more usual to undertake laboratory tests using scaled down equipment on samples, which is more practical but less representative of the natural situation. An alternative option is provided by geotechnical tests.

There have been two main approaches to testing abrasion/abrasiveness. The earliest focused on relative hardness of constituent minerals. Friedrich Mohs defined a 10-stage hierarchy on the basis of which a mineral was strong enough to scratch another in that sequence: talc (softest), gypsum, calcite, fluorite, apatite, orthoclase, quartz, topaz, corundum, and diamond (hardest). Other minerals are then placed in this sequence between pairs of the listed minerals based on their ability to scratch or be scratched. Mohs' Scale is a relative scale. It was later developed by Rosiwal into absolute values, measured in the laboratory, taking corundum as having a value of 1000. An approach for use in the field is given in Mol (2014). Hardness may be determined in the field or laboratory by rebound devices such as the Schmidt Hammer.

Most rocks and soils contain a variety of minerals which are variably affected by diagenetic and weathering processes that affect hardness. Therefore tests on individual minerals, while useful, are not wholly adequate for engineering geology purposes which requires examination of mechanical properties of rock and soil masses. Mineral-based texts also neglect the sizes and shapes of grains.

The Cerchar Abrasive Test was developed to assess the potential abrasion damage to plant and equipment, for instance, cutter life in the field, and is also significant for building and construction materials including dimension stone (Deliormanli 2011). It involves the use of an abrasive stylus to scratch a broken or cut surface of a sample. Strong correlation exists between the Cerchar Abrasivity Index (CAI) and rock strength and abrasion.

The Rock Abrasivity Index was developed to take account of the content of abrasive minerals and the strength of the rock and is based on multiplying the unconfined compressive strength (UCS) and equivalent quartz content (EQC) of the sample (Plinninger 2010).

The Los Angeles Abrasion Value Test examines the resistance to degradation of bound aggregates in highway pavements. A sample of coarse aggregate retained by a No. 12 (1.7 mm mesh) sieve is weighed and is then subjected to abrasion and grinding in a steel drum. The sample material is once again passed over a No.12 sieve and weighed. The difference between the two weights is a measure of susceptibility to abrasion and, therefore, of the performance of the aggregate when subjected to abrasion during use (JSA 2007).

Summary

Abrasion is important in wind, water, and glacial erosion. But it is also significant for determining the suitability of minerals for use as industrial and domestic abrasives, blunting and wear on machinery such as that used in drilling and tunneling, and the performance of aggregates during wear. Tests relate to the determination of hardness by scratching or by rebound on hammering or abrading samples to determine the rate of wearing down.

Cross-References

▶ Aggregate Tests
▶ Aggregate
▶ Coastal Environments
▶ Drilling
▶ Erosion
▶ Fluvial Environments
▶ Glacier Environments
▶ Mechanical Properties
▶ Rock Field Tests
▶ Rock Laboratory Tests
▶ Rock Properties
▶ Tunnels

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Accreditation

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Synonyms

Geoscience standards; Professional recognition; Professional registration; Professional licensure

Institutional Accreditation

Accreditation of institutions of higher education (colleges and universities) is a process by which the quality of the overall academic experience that a student can expect is evaluated and found to meet standards set by an accrediting body. In the United States, the accrediting body is typically a regional or national organization made up of representatives from the colleges and universities.

Program (or Specialized) Accreditation

The curricula within an academic department may be accredited at the program level, either department wide or limited to specific course programs (majors). This type of accreditation attests to the content and, by implication, the overall learning experience the student receives by completing a program (courses of study, major) within an academic department. The organizational structure of this type of accreditation varies according to national custom.

Advantages of Undergraduate Academic Program Accreditation

In the design professions (a group that includes geology and engineering geology), the accreditation of undergraduate academic programs works in connection with the process of professional credentialing (licensure or certification) and thus eases the path to the credential, which in turn enhances career advancement and a sense of professionalism. Accreditation assures the credentialing organization, whether a statutorily authorized licensure board or a certifying or chartering professional organization, that the academic coursework of the candidate meets the standards of the accrediting agency. (In practice, licensure boards or other credentialing organizations must first accept the standards of the accrediting agency.) In the absence of an academic program accreditation, the credentialing body must undertake independent evaluation of the candidate's academic background, adding complexity and cost to the credentialing process.

Status of Accreditation

On a worldwide basis, academic program accreditation in geology or engineering geology is sparsely implemented. The Accreditation Board for Engineering and Technology (ABET) has offered (on a world-wide basis) geology program accreditation since 2016. In the United States, the first geology program to be accredited was at the University of Arkansas at Little Rock.

In the United Kingdom, the Geological Society of London accredits programs in geology and engineering geology. This extends to some universities in Hong Kong.

In Canada, university-wide accreditation is similar to the system used in the United States but is normally addressed through provincial Associations of Professional Engineers and Geoscientists. Program-level (or specialized) accreditation in geology or engineering geology is not extant.

The Future of Accreditation

ABET offers its accreditation programs worldwide. The ABET framework allows for geology departments to offer curricula with different emphases. Hence, it is possible for an accredited geology program to offer an emphasis ion engineering geology or environmental geology, for example. The future of accreditation depends on demand from students and credentialing agencies and acceptance of it by geology departments.

Cross-References

▶ Ethics
▶ International Association of Engineering Geology and the Environment (IAEG)
▶ International Association of Hydrogeologists (IAH)
▶ International Society for Rock Mechanics (ISRM)
▶ International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)
▶ Professional Practice

A

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Acid Mine Drainage

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Synonyms

Acid rock drainage; Acidic mine drainage; Acidic rock drainage; Mine water pollution; Polluted mine water

Definition

Water encountered in and/or draining from active or abandoned mines which has a low pH and/or highly elevated concentrations of potentially ecotoxic metals.

Mining disrupts the natural hydrogeological conditions in the subsurface often increasing the through-flow of aerated waters, resulting in oxidative dissolution of sulfide minerals. The ferrous sulfide (FeS_2) minerals (pyrite and its less common polymorph marcasite) release acidity when they dissolve. (This is *not* true of the non-ferrous sulfide minerals.) This acidity can attack other minerals, releasing further metals to solution. Clay minerals commonly dissolve to release Al^{3+}, with Mn^{2+}, Zn^{2+}, and (less commonly) Ni^{2+}, Cu^{2+}, Cd^{2+}, Pb^{2+}, and the metalloid As also being mobilized where mineralogical sources for these are present. Above the water line, dissolution is often incomplete, and the products of sulfide oxidation accumulate as efflorescent hydroxysulfate minerals. Later dissolution of these will release acidity. The resultant water is "acid mine drainage" (albeit "acidic" is more correct). In addition to low pH and elevated concentrations of iron and (possibly) other metals, acid mine drainage is invariably rich in sulfate (Younger et al. 2002).

The total acidity in mine drainage has two components: "proton acidity" due to the presence of high concentrations of hydrogen ions (H^+) that manifest in a low pH (below 6 would typically be regarded as "acidic" in this context) and "metal acidity" due to the presence of the metals listed above that tend to react with any available alkalinity to form hydroxide minerals, releasing further protons in the process. In many mine waters, the total acidity is exceeded by the total alkalinity, which in the relevant pH range is predominantly accounted for by dissolved bicarbonate (HCO_3^-). Such mine waters are termed "net-alkaline." Where the total acidity exceeds the total alkalinity, the mine water is termed "net-acidic." This distinction is important: many net-acidic mine waters actually have a near-neutral pH (>6) where they first flow out at surface, but after prolonged oxidation and hydrolysis of their metal acidity, pH drops to strongly acidic levels (< 4.5). Misidentification of net-acidic waters as net-alkaline on the basis of pH alone can be a costly mistake.

The principal concern with acid mine drainage is ecological, as it often devastates aquatic life in receiving watercourses. In engineering terms, the high acidity poses heightened risks of corrosion of steel and other materials, thus demanding careful galvanic protection. The high sulfate concentrations pose a risk of rapid weathering of concretes based on ordinary Portland cement. Sulfate-resistant cements must be specified for structures likely to contact acid mine drainage. Acidic attack can weaken many rocks and engineering soils. Passive and active treatment methods are routinely used to treat acid mine drainage (Fig. 1).

Acid Mine Drainage, Fig. 1 A typical acid mine drainage outflow – Bardon Mill Colliery, Northumberland, UK

Cross-References

- Acidity (pH)
- Contamination
- Hydrogeology

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Acidity (pH)

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Definition

Acidity (pH) is numerically equal to the negative decimal logarithm of the activity of (a_{H+}) or the concentration [H^+] of hydrogen ions (in gram ions per liter).

This concept was introduced in 1909 by the Danish chemist Sørensen. pH reflects the first letters of Latin words *potentia hydrogeni* – the power of hydrogen, or *pondus hydrogenii* – weight of hydrogen.

For low mineralized water, the difference between activity and concentration of hydrogen ions is not geochemically significant, but for high mineralized water the identification of activity and concentration is essential.

The introduction of pH as an indicator of acid-base properties of aqueous solutions was founded on the ability of water to dissociate into ions according to the scheme $H_2O = H^+ + OH^-$. In connection with this reaction and using the concept of ionic product of water, $K_W = a_{H+} + a_{OH-}$ (where K_W - ionic product of water, a_{H+}+ and a_{OH-} activities of H^+ and OH^- respectively). K_W at 22 °C is equal 10^{-14}. If the water does not contain other ions, the H^+ and OH^- concentrations are equal according to the electroneutrality of ion activities and at 22 °C it has the value of 10^{-7}. In that condition, pH = pOH = 7 (the neutral reaction medium). If $a_{H+} > a_{OH-}$ the solution is acidic (pH < 7), if $a_{H+} < a_{OH-}$ the solution is alkaline (pH > 7).

The pH value is an important characteristic of all aqueous solutions and natural water bodies (rivers, lakes, seas, oceans). The pH value along with the reduction-oxidation (redox) potential determines the possible concentration in aqueous solutions of different chemical elements, their migration forms, and possible processes of changes of concentrations and properties of compounds. It also has effects on soils and ecosystems both terrestrial and aquatic (Kraynov et al. 2004).

Cross-References

- Acid Mine Drainage
- Diagenesis
- Dissolution
- Volcanic Environments

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Aeolian Processes

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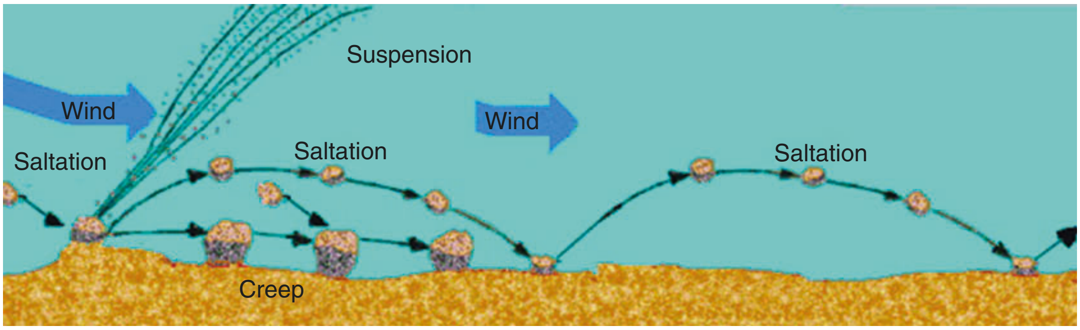
Definition

Processes related to wind in the atmosphere. For engineering geology, those processes which interact with the geosphere.

Wind erodes, transports, and deposits materials especially in arid or semiarid areas with sparse vegetation cover and little soil moisture, particularly where the substrate consists of unconsolidated sediments. Turbulent wind removes loose, fine-grained particles and entrains them as dust (deflation), or wears down surfaces by inter-particle grinding and blasting or onto rock surfaces (abrasion) thereby creating more transportable material. Areas of long-term sediment deflation result in a rock surface (desert pavement). Deflation can form basin shaped depressions, from centimetres to kilometres (blowouts) in size.

Particles are transported in three different ways:

- Upwelling currents of air support small-suspended particles (less than 2 mm in diameter) and can hold them in suspension indefinitely as haze or dust depending on how much is entrained.
- Sand-sized particles can bounce some 1 cm above the ground surface at about half to one third of the wind speed (saltation). Saltating particles can impact other grains that also saltate.
- Larger grains, too heavy to saltate may be pushed, rolled, or slide (creeping) along surfaces.



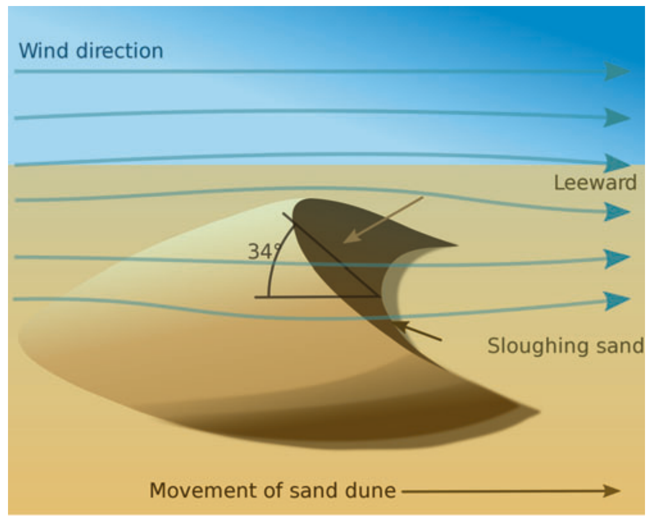
Aeolian Processes, Fig. 1 Mobilization of particles during wind erosion. Creative Commons Attribution 3.0 Unported License: © Po Ke Jung

Aeolian turbidity currents arise when rain passes into arid areas causing cooler denser air to sink towards the ground. When this reaches the ground, it is deflected forward as wind and suspends, mainly silt-sized debris, as dust storms. Suspension persists until the wind energy decreases and cannot support the weight of particles which are then deposited. Deposition is local if the particles are entrained near the ground and wind energy is low. But dust can be transported for long distances in strong winds before deposition takes place. If upwelling is strong enough to carry particles high into the atmosphere, these can be distributed on the continental or global scale.

Dust may be deposited sparsely into local soils, but where frequent winds carrying large amounts of dust meet a barrier, such as a mountain range, thick silt deposits (loess) accumulate. These are highly porous and have problematic engineering properties including compaction and collapse when moistened or affected by earthquakes.

Wind across a loose, dry, surface moves and deposits particles locally. Wind over a sand grade surface may cause saltation forming troughs and crests with long axes perpendicular to the wind direction at distances (wavelengths) reflecting the average length of particle bounces. The resulting ripples have the coarser material at the crests and finer material in the troughs (Gillette and Passi 1988).

Larger scale movements build dunes by saltation and creep. Grains move up a slope towards a crest, accumulate there, and when the critical angle of repose is exceeded, fall down the far side (This angle is the steepest angle of dip to the horizontal plane to which a material can accrete without failing). This causes a profile with a shallow back slope, often covered with smaller ripples, and a steep fore-slope (slip face) (Nishimori and Ouchi 1993). This repeated process causes slow advance of the dune until it is stabilized by changing climate and/or armouring (natural vegetation; engineered surfaces). Dunes may sometimes grow to a few 100 m in height (Lancaster 1984). The characteristics of depositional structures are environmental indicators in the geological record.



Aeolian Processes, Fig. 2 Process of dune formation. Creative Commons Attribution-Share Alike 3.0 Unported License. © dune.jpg: RaySys

Wind has similar effects, at a smaller and more local scale, on dry uncohesive soils exposed by human activity such as: sites stripped for quarrying, construction, and engineering works; mine and quarry tips and tailings lagoons; and ground cleared for agriculture (leading to soil loss and deterioration) (Kabir and Madugu 2010). Dust emissions can be reduced by keeping exposed surfaces moist or stabilizing them and enclosing plant and equipment (Figs. 1 and 2).

Aeolian processes can be relevant to engineering geology and engineering geologists in several ways:

- The difficult site investigation and sampling conditions where dry poorly consolidated sand or silt deposits occur at the ground surface.
- The need for careful support of excavations and trenches and design of appropriate foundations in poorly consolidated aeolian deposits.

- The potential for collapse of loess when affected by earthquakes or excessive moisture.
- The avoidance of dust emissions from construction, mining, and quarrying sites.
- The analysis for potentially hazardous minerals and elements in dust.
- The protection of developments from incursions of aeolian deposits.

Cross-References

▶ Desert Environments
▶ Erosion
▶ Loess
▶ Physical Weathering
▶ Sand
▶ Silt

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Aerial Photography

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Definition

An image of the ground produced on light-sensitive media (including digital sensors or a film of light-sensitive emulsion) that has been taken from an elevated position, unsupported by a ground-based structure. Aerial photographs may be vertical ($<3°$ from the camera to the focal point of the image) or oblique.

Synonyms

Air photographs; Airborne imagery; Orthophotos

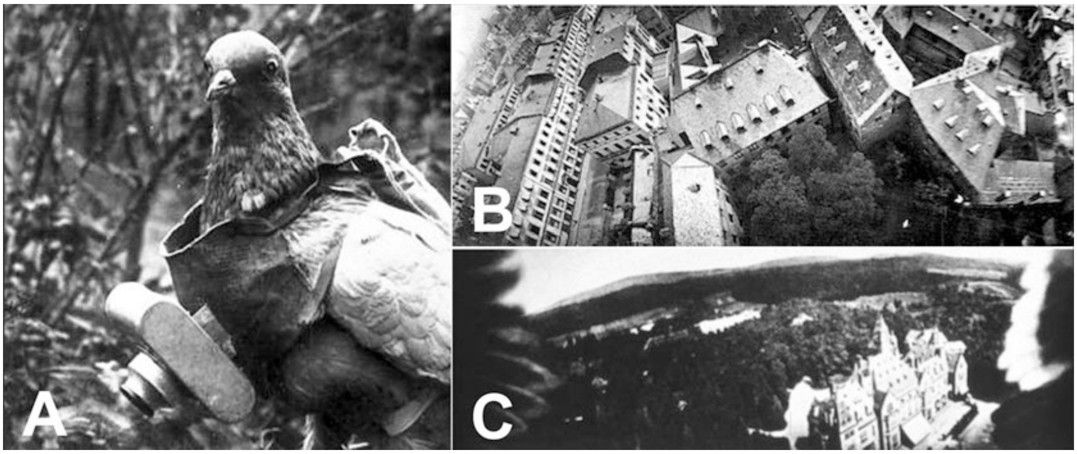
Early Years

Aerial Photography made its debut late in the nineteenth century. While French balloons had been used for military observations as early as 1794, the secret of capturing and holding or “fixing” images on film would not be invented for another 40 years. The first camera was invented in 1845 by Francis Ronalds who also invented the electric telegraph and was arguably the world’s first electrical engineer.

By 1855, French balloonist and photographer Gaspar Felix Tournachon, also known as “Nadar,” patented the idea of using photographs to survey and map from the air. Three years later, he successfully produced an air photograph from a tethered balloon 80 m above the French village of Petit-Becetre (PAPA International 2017). Surviving early aerial photographs include the City of Boston captured a few years later by James Wallace Black, and images of Paris by Nadar and later by Triboulet. As the camera became portable (no longer requiring a darkroom in the sky) and film transfer more reliable, the quality of image capture improved remarkably (Fig. 1).

With box cameras gaining popularity, aerial photography platforms extended dramatically to kites (initially using timed explosives to trigger the camera), rockets and famously, pigeons. Julius Neubronner conceived of and patented the idea of strapping small aluminum cameras to homing pigeons in 1908. This invention became the

Aerial Photography, Fig. 1 Oblique air photograph of the pyramids of Giza taken by Eduard Spelterini in 1904 from a balloon 600 m above the ground (Spelterini 1928) (Photograph is public domain)



Aerial Photography, Fig. 2 Neubronner's German box camera with remote trigger strapped to a homing pigeon. Note the wingtips in the third image (Copyright) (Public domain photographs)

short-lived Bavarian Pigeon Corps which gathered reconnaissance images for Germany, but were soon replaced by the technological advancements in flight and the stability of airplanes (Fig. 2).

By 1909, Wilbur Wright, the elder half of the famous duo, took a passenger on a flight in Italy who captured the first motion pictures and photographs of a military field near Rome (PAPA International 2017) and soon airplanes and dirigibles were being used as aerial photography platforms.

Advancing Technologies During WWI

During the early growth of aerial photography, ongoing technological advances were required to accommodate the needs of a nascent field. These included improvements to camera stability and shutter speed. US inventor Sherman Fairchild created a camera with the shutter located inside the lens, accomplishing both (stability and shutter speed) and fundamentally changed camera design from that point forward. Frederick Charles Victor Laws, flying dirigibles for what would become the British RAF, experimented with overlapping photographs and found that stereoscopic vision was possible with a 60% overlap in images. This became a fundamental principle of aerial photograph interpretation that allowed for the recreation of a three-dimensional scene (or depth perception). In addition to better interpretations and cartographic precision, when coupled with lens information stereoscopic images allowed the calculation of elevation of objects on the ground. Other technologies included increased focal distances and floor-mounted pilot-triggered cameras (in airplanes). Some estimates provide that England took about 500,000 air photographs during WWI, and the Germans claim similar statistics (Northstar Imaging 2017).

Post WWI Applications

Following WWI, commercial application of aerial photography began in earnest. In the UK, Aerofilms Ltd. was established by Francis Wills and Claude Graham White to begin commercial surveys and photogrammetry of Europe, Africa, and Asia using aircraft from the London Flying School. In the USA, Sherman Fairchild started his own aircraft firm and developed specialized planes for high altitude aerial surveys (from 23,000 and later 30,000 ft.). One of his early contracts was to study soil erosion in New Mexico.

In Canada, national air surveys for forestry and mapping began as early as 1920 using dirigibles donated by Britain, and by 1925, the National Air Photograph Library was created to be responsible for all nonmilitary images. Similar patterns emerged in other countries such as Australia that began to accumulate air photographs in 1928. In Canada, by 1956, the role of aerial photography shifted from the Royal Canadian Air Force to commercial enterprises.

Aerial photography evolved in WWII to more completely cover high speed, high altitude stereoscopic aerial photography. At its peak, more than 50,000 images a day were captured by British pilots, and similar numbers were acquired by other countries.

Following WWII, high altitude stereoscopic air photographs were being acquired by national surveys or by commercial means around the world. Long range bombers were adapted to reconnaissance air craft at the onset of the Cold

War and in the 1960s satellites began capturing images of the Earth's surface. In 1972, the USA started the Landsat program to capture imagery of the Earth from space.

Types of Air Photographs

Vertical

Vertical air photographs are those taken looking straight down. They are typically used for mapping and photointerpretation (Fig. 3).

Oblique

Refers to photographs taken at an angle to the Earth surface.

Orthorectified (Orthophotos)

Vertical air photographs taken looking straight down, apparently from an infinite distance and where perspective and terrain corrections have been geometrically rectified such that the image may be viewed as a map.

Film Types

Film types most commonly used in aerial photography are black and white, color, and near infrared. Black and white images were common in the 1970s but largely overtaken by color images. Near infrared aerial photography is taken where the film or sensor is sensitive to the near infrared spectrum (700–900 nm). It is frequently used to identify vegetative health and to reduce the effects of haze. Infrared photography is often called false-color photography.

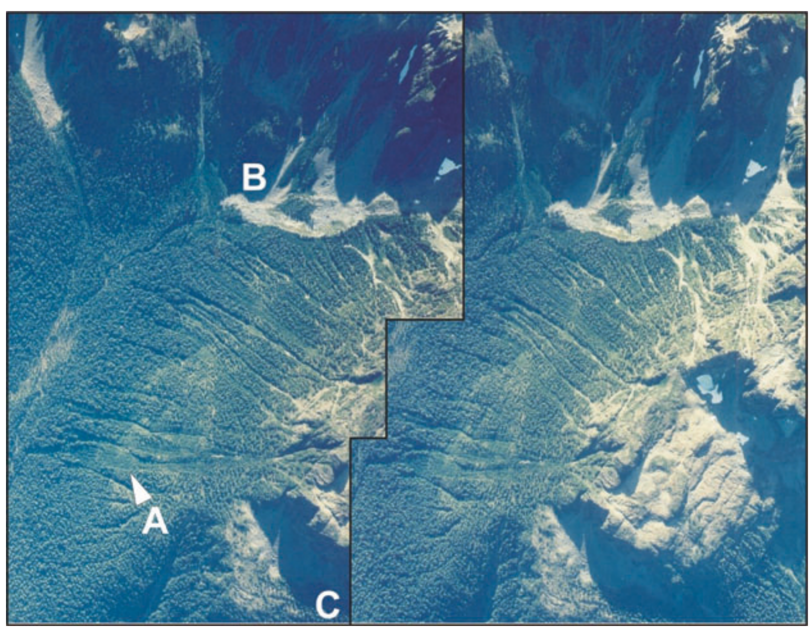
Application to Engineering Geology

The ability to view the earth from a distance, to observe landforms, patterns, sources of change (the propagation of processes through a geomorphological system for example), and evidence for geological history are largely attributable to the invention, development of, and improvements to aerial photography. Air photographs provided foundational knowledge upon which modern engineering geology is based. They contributed to many aspects of the field, from basic cartographic mapping to the slow unveiling of geological solutions that changed our perception of fundamental principles (such as the scablands, example provided below).

Topography

Engineering geology relies substantially on understanding the topographic ground surface. Before aerial photography important ground features were sketched or surveyed. Post WWII however, the widespread use of stereo air photographs allowed for the creation of nationwide accurate topographic maps. Using a stereoplotter, a mapper would "float" a red dot along the surface of the ground as viewed through two air photographs. The red dot could then be moved along x and y axes, following the ground contour while remaining at the same elevation. Tracing this dot led to the creation of contour lines, and ultimately, topographic maps. All of North America was mapped in this fashion, the USA having complete topographic coverage at 1:24,000 and Canada at 1:50,000.

Today, topography can be created from overlapping images (both oblique and vertical) using the same principles



Aerial Photography, Fig. 3 Vertical air photograph pair used for Hazard interpretations in Engineering Geology. (**a**) indicates one of the larger snow avalanche paths, (**b**) a rock avalanche path, and (**c**) a rock fall. Transmission line routes through remote areas, roads, pipelines, or logging routinely consider hazards identifiable on air photographs partially or fully automated by computers. The forefront of this technology allows the user to take multiple overlapping images from a regular camera or even a phone, and create a three-dimensional model of the image (see "Structure from Motion" below).

Direct Mapping

Geological data that impacts engineering design is routinely derived from air photographs. Structural features such as joints, faults, folds, bedding planes, and other lineaments are often discernable from detailed and even regional imagery. Constraint maps might include information about wetlands, karst activity, depth to bedrock and the various types of ground instability (e.g., Fig. 3), and important resources such as aggregate are also readily found on air photographs. Direct mapping is used to support and extend the information found in borehole investigations for foundations or for linear infrastructure.

Resources and Land-Use Change

It would be hard to overestimate the contribution of aerial photography to the understanding and development of national resources. In Canada, for example, where population densities are low, the historical record of timber resources, agricultural land, and changes in land-use were accurately captured by repeated air photograph surveys. Much of the country is covered at intervals of 10 years or less beginning in the mid-1950s. Similarly, changes in coastlines, the widespread existence of permafrost, the ongoing movements of rivers and the development of transportation corridors, pipeline routes, and transmission lines all benefited enormously from the air photograph record. Geomorphological and terrain maps historically relied on air photographs to guide and direct engineering efforts with respect to hazards.

Unravelling the Scablands

Finally, air photographs give engineering geologists new insights into old problems. The scablands are an excellent example: In the 1920s geologists had firmly adopted the theory of Uniformitarianism, the notion that the surface of the Earth's crust has developed because of uniform and continuous processes throughout geological history. Harlen Bretz (1923) examined the massive rock-cut channels and huge ripples in east-central Washington, and proposed something different. In what he named the channeled scablands, Bretz proposed a post-glacial flood of such magnitude that it was not accepted by his colleagues. Not only did Bretz's theories contradict uniformitarianism, but others argued that no sufficient source of water was available. In 1956, Bretz published results from a second field trip, this time including air photographs showing massive ripple beds tens of meters high that began to turn the tide of criticism that had marred his work to date. Although it would be another 20 years before his ideas were fully accepted, air photographs were fundamental in the visualization of the processes, and to find adequate sources of water to create the landforms.

New Technology

Google Earth Image Draping

Google Earth® was released to the public in 2005 and fundamentally changed the way we view the world. Google Earth drapes aerial photographs and satellite images over a digital elevation model (DEM) of the world. It allows the viewer to "fly" through the world at varying scales and to simulate a 3D environment by tilting and panning the images in-flight. Among the many improvements, the ability to shift from vertical to oblique views of any image is a step change in technology. Ongoing agreements with satellite image and air photograph providers as well as the ongoing acquisition of new data mean that Google Earth contains the most accessible, widely used remote imagery database in the world. Air photographs and satellite images are blended seamlessly for the viewer, and newer images are using technology similar to Structure from Motion (below) to create photorealistic DEMs (Fig. 4).

Softcopy Mapping

Air photographs today are captured digitally and many old air photographs have been converted to digital format. Not surprisingly, heads-up 3D mapping systems have replaced traditional air photograph interpretation for many users. In this computer supported environment, air photographs are viewed on-screen using active polarized lenses to ensure that each eye sees a similar but slightly different image, maintaining the parallax effect. The advantages are several including on-the-fly histogram changes that allow the viewer greater detail in over or underexposed portions of the scene (like turning a light up or down), rapid changes in map scale, increased detail, and perhaps most importantly, direct transfer of line work into a spatial model and pre-assigned datum through a GIS platform.

Air Photographs and LiDAR

Light Detection and Ranging, first introduced in the 1980s, creates a digital surface model by shooting lasers at a target surface, and calculating distance from the returned signals. The advantage of LiDAR is a data-rich point cloud that produces a detailed digital elevation model of the ground surface (a bare Earth model) with sub-meter accuracy. Airborne LiDAR (taken from above) is therefore best for relatively flat surfaces, and ground-based LiDAR is best for inclined surfaces. LiDAR is now routinely used to supplement aerial

Aerial Photography, Fig. 4 Oblique Google Earth image of Portland Maine, USA. The DEM is computer generated based on multiple images. Viewers can "fly" around individual trees, beneath the bridge, and through streets (Image copyright Google Earth 2016)

photography and increase both accuracy and content of the interpretations. Because of softcopy mapping, the images are frequently interpreted in the same platforms.

UAVs and Satellites

Digital capture has further revolutionized aerial photography. Landsat is the longest running commercial satellite image capture program beginning in 1972 and continuing today; however, there are many concurrent programs gathering images daily and satellite imagery is largely supplanting traditional air photograph programs.

At the same time, digital technology and portable cameras have created a new niche market whereby unmanned aerial vehicles (UAV) are gathering air photographs for universities, industries, governments, hobbyists, and enthusiasts alike. Despite potential licensing and privacy issues, the digital processing of location information and automatic generation of surface models means that UAV technology is affordable and useable by a tech-savvy consumer.

Structure from Motion

Structure from Motion (SfM) is the phenomenon whereby humans perceive three dimensions from two-dimensional image sequences through motion. Parallax, the difference in the apparent position of an object viewed from slightly different lines of sight (like two eyes), is perceived (in SfM) as objects move by a viewer. In the same manner, computers analyze multiple images of an object in motion relative to the camera, and identify changes in key features. Measuring the angular relationships between all the key features yields distance information and ultimately allows a computer to generate a surface model (DEM).

Structure from Motion in Engineering Geology is particularly useful for measuring structures, replacing hand measurements from a few places, with thousands of measurements taken from the automated or semi-automated classification of a SfM generated point cloud that forms the DEM (Westoby et al. 2012). In principle, SfM works for everything from images taken with a phone, to images taken from space, but as a rule, higher quality images produce better results.

One of the exciting outcomes of SfM is the ability to go back through historical air photographs to computationally produce and compare DEMs from different periods. In this manner, engineering geologists may be able to observe evolution of landforms over the air photograph record.

Summary

Beginning in the nineteenth century, aerial photography changed the way engineering geologists, and the public, viewed the world. Aerial photographs enable us to understand Earth materials, mechanics, landforms, and processes in an unprecedented way, fully contextualized and at multiple scales (Mollard 2013). At the dawn of the twenty-first century, traditional aerial photography is becoming supplanted by satellite imagery, UAVs, and handheld cameras where multiple images are combined in sophisticated programs. The benefits to engineering geology, however, remain.

Cross-References

▶ Insar
▶ Lidar

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Aeromagnetic Survey

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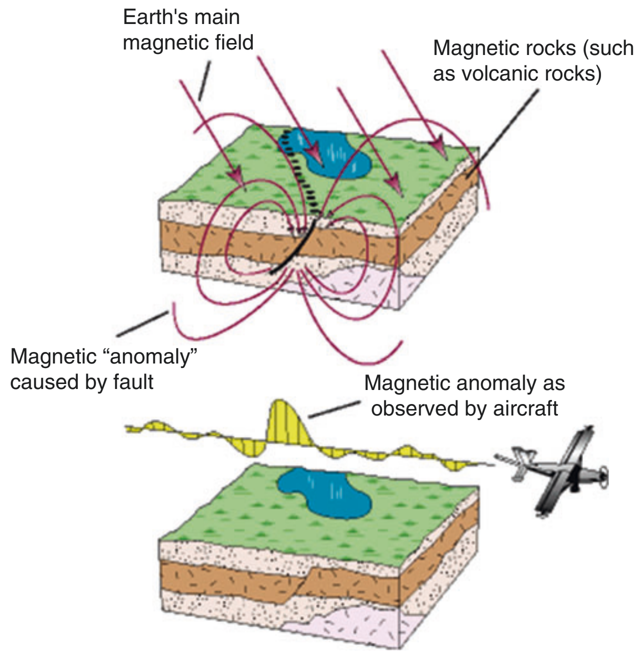
Definition

An **aeromagnetic survey** (AMS) is an airborne geophysical survey performed using a magnetometer aboard or towed behind an aircraft. A magnetometer is an instrument used to measure the magnetic field. Aeromagnetic surveys are probably one of the most common types of airborne geophysical surveys. The applications of AMS in engineering geology include, but are not limited to, near-surface geological mapping, structural geology mapping, aiding three-



Aeromagnetic Survey, Fig. 1 Schematic illustration of an aeromagnetic survey. The low-altitude flying airplane flies back and forth in a grid-like pattern to measure the magnetic anomalies caused by changes in the magnetic field by different rocks and geological structures (Blakely et al. 1999)

dimensional (3D) geological subsurface model construction, groundwater study, environmental study, and geologic hazards assessment.

In an aeromagnetic survey, an airplane, flying at a low altitude, carrying a magnetic sensor, flies back and forth in a grid-like pattern over an area, recording disturbances in the magnetic field (Fig. 1). Height and grid line spacing determine the resolution of the data. Geologic processes often bring together rocks with slightly different magnetic properties, and these variations cause very small magnetic fields above the Earth’s surface. The differences in the magnetic field are called “anomalies” (Blakely et al. 1999).

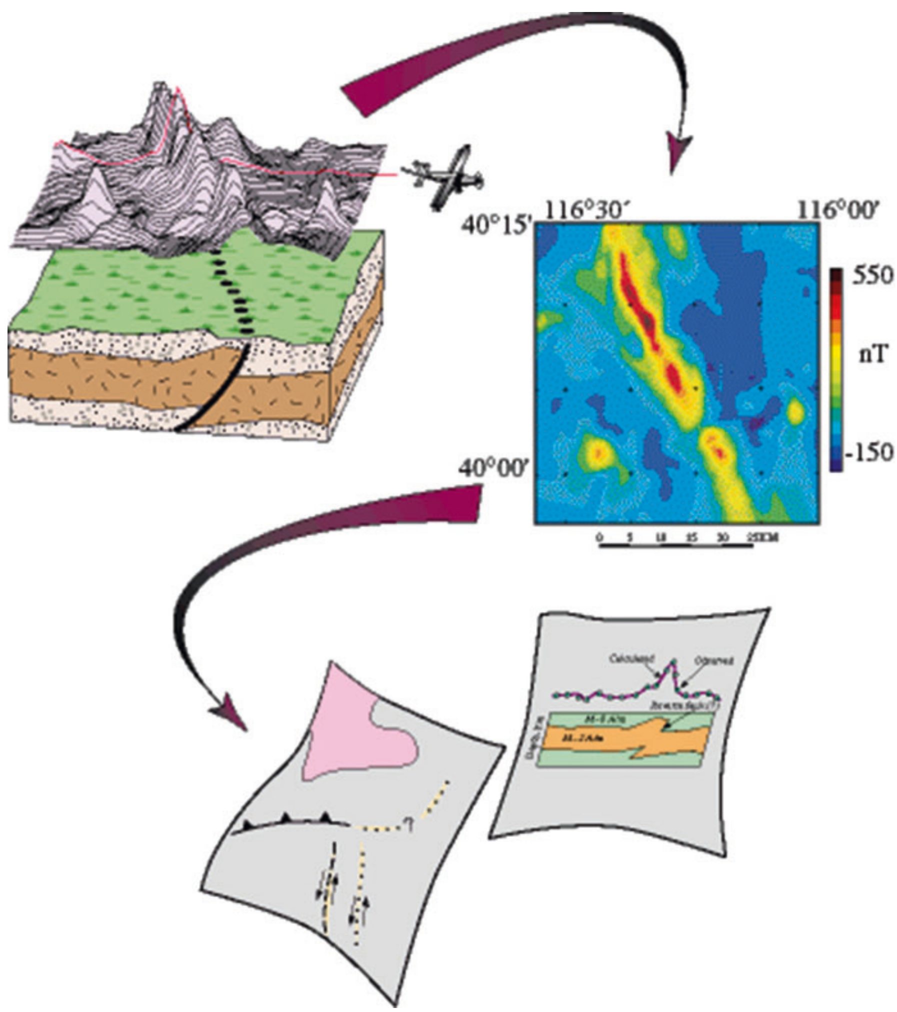
Introduction

Rocks or soils containing iron and nickel can have strong magnetization and, as a result, can produce significant local magnetic fields. The magnetic minerals contain various combinations of induced and remanent magnetization. At exploration depths, the Earth’s primary magnetic field is perturbed by the presence of magnetic iron oxide (magnetite, the most strongly magnetic and the most common magnetic mineral), iron-titanium oxides (titanomagnetite, titanomaghemite, and titanohematite), and iron sulfides (pyrrhotite and greigite) (Reynolds et al. 1990). The remanent magnetization in the Earth’s magnetic field occurred during the mineral formation process, whereas the induced magnetization was created by the presence of the Earth's magnetic field. The magnitudes of both induced and remanent magnetizations depend on the quantity, composition, and size of the magnetic mineral grains. The goal of the magnetic method is to map changes in the magnetization that are, in turn, related to the distribution of magnetic minerals (Hoover et al. 1992).

The magnetometer was invented in 1832 and was designed and constructed to measure the intensity of the Earth's magnetic force (Gauss 1832). However, development of magnetometers used in exploration, i.e., usable for taking a large number of readings over a given area of interest in a reasonably short period of time, dates only from the invention of the electronic magnetometer during World War II (Reeves 2005). Aeromagnetic surveys were performed, using a magnetic anomaly detector attached to an aircraft, in World War II to detect submarines.

The aeromagnetic survey technology was progressively refined with time. In the late 1950s, the proton precession magnetometer was invented but, despite ongoing refinement of the fluxgate instrument, eventually was replaced in routine survey operations (Reeves 2005). The US Geological Survey (USGS) pioneered the first airborne magnetic survey in 1944, during which 10,000 line miles of magnetic data were collected over Naval Petroleum Reserve 4 in the northernmost part of Alaska (Hildenbrand and Raines 1987). In the following years, airborne geophysics evolved into a major component of earth science. Today, aircrafts are capable of acquiring a wide variety of geophysical data (e.g., gravity, magnetic, electromagnetic, radiometric, spectral, and thermal), which are critical to solving national resource, environmental, and geologic hazards problems.

After pioneering the first airborne magnetic survey in 1944, the USGS collected piecemeal aeromagnetic data for most of the USA, including offshore areas on both coasts. The USGS's digital and analog archives comprise more than 1,000 surveys, covering approximately 8,000,000 line km of data, flown at various flight heights and line spacings (Hanna 1987).



Aeromagnetic Survey, Fig. 2 Schematic illustration of steps of an aeromagnetic survey and products (Blakely et al. 1999)

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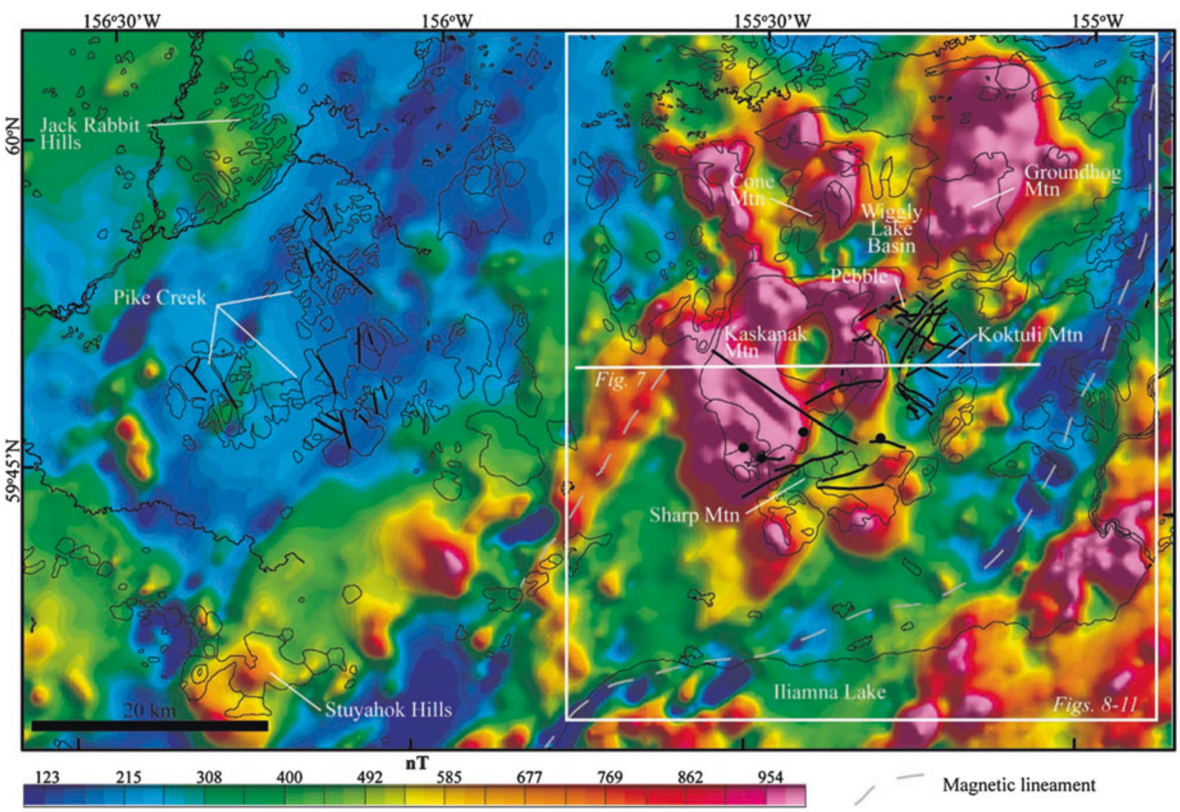
Aeromagnetic Survey Method

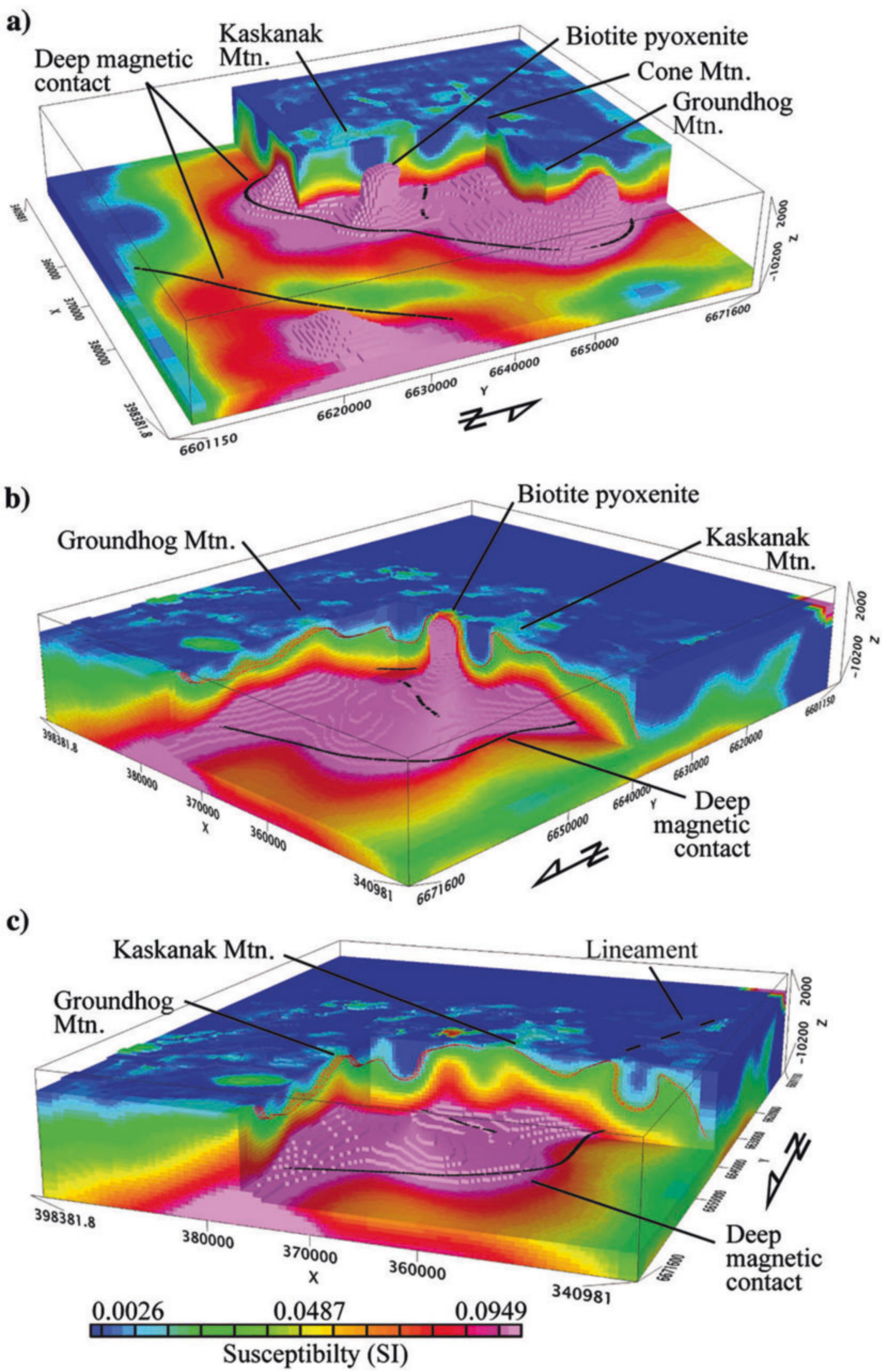
Magnetic measurements are usually made from low-flying airplanes flying along closely spaced, parallel flight lines. Additional flight lines are flown in the perpendicular direction to assist in data processing. These large volumes of measurements are processed into a digital aeromagnetic map. Assisted by computer programs, the geophysicist builds a geologic interpretation from the digital aeromagnetic data, incorporating geological mapping and other geophysical information (gravity, seismic reflection) where available (Fig. 2). Interpretations often involve both map-based information (e.g., a fault map) and three-dimensional information (e.g., a geologic cross section and 3D geological model) (Blakely et al. 1999).

The workflow of the aeromagnetic survey method includes the aeromagnetic survey design, data acquisition, data processing, and interpretation. There are many parameters to consider in a typical aeromagnetic survey design. These parameters include the line spacing of flying, flying heights, the flight line direction with the intention of maximizing the magnetic signature, and features of the survey aircraft. Flight line spacing is determined by the degree of detail required in the final mapping or the size of exploration target and the funding available for the survey. The strength of a magnetic field decreases approximately as the inverse of the square of the distance from the magnetic source. Therefore, to record small variations in the fields, aircraft must fly close to the ground (Horsfall 1997).

As the aircraft flies, the magnetometer measures and records the total intensity of the magnetic field at the sensor. Aeromagnetic data can be presented as contour plots or thematic maps (e.g., Fig. 3). Intensity of the aeromagnetic anomalies is expressed in these plots, or maps, as contour lines or different colors. The shape, depth, and properties of the rock bodies causing the aeromagnetic anomalies can be interpreted by a trained geophysicist. The magnetic anomaly map also allows a visualization of the geological structure of the upper crust in the subsurface, particularly the spatial geometry of bodies of rock and the presence of faults and folds because different rock types differ in their content of magnetic minerals even if the bedrock is obscured by surficial materials, such as sand, soil, or water.



Aeromagnetic Survey, Fig. 3 The magnetic anomaly map of the Pebble district and Pike Creek–Stuyahok Hills area, in southwest Alaska. Both areas show contrasting magnetic signatures. *Dashed lines* represent major magnetic lineaments discussed in the text. *Black dots* show the location of middle Cretaceous porphyry-style ores (Anderson et al. 2014)



Aeromagnetic Survey, Fig. 4 The result of 3D magnetic inversions. The model shows that relatively highly magnetic material occurs below Kaskanak Mountain, Alaska, and extends continuously to the north of Groundhog Mountain (Anderson et al. 2014)

Selected Case Studies

Aeromagnetic surveys, in conjunction with other geophysical methods, are used to help in geological mapping, structural geology mapping, environmental and groundwater studies, 3D geological modeling, mineral exploration, and petroleum exploration. This section focuses on case studies of the aeromagnetic applications in engineering geology and its closely related fields.

Hood (1965) presented the measurement of the first vertical derivative of the total field in aeromagnetic surveys by using two sensitive magnetometer heads, separated by a constant vertical distance. The difference in outputs revealed that steeply dipping geological contacts in high-magnetic latitudes are outlined by the resultant zero-gradient contour. It also demonstrated that it is possible to obtain the depth of a subsurface contact from an aeromagnetic survey. Measurements of the vertical gradient during aeromagnetic surveys would, therefore, be of great value in subsequent geological mapping of the areas surveyed.

Blakely et al. (2000) presented the results of a high-resolution aeromagnetic survey of the Amargosa Desert, and surrounding areas, an area of approximately 7,700 km^2, extending from Beatty, Nevada, to south of Shoshone, California, that includes parts of the Nevada Test Site and Death Valley National Park. Aeromagnetic flight lines were oriented east–west, spaced 400 m apart, and flown at an altitude of 150 m above terrain or as low as permitted by safety considerations. This survey provided insights into the buried geology of this structurally complex region.

Ranganai and Ebinger (2008) integrated aeromagnetic (AM) and Landsat Thematic Mapper (TM) data from the south-central Zimbabwe Craton to map the regional structural geology and to develop strategic models for groundwater exploration in hard-rock areas. The derived maps reveal several previously undetected lineaments corresponding to dikes, faults, shear zones, and/or tectonically related joints, striking predominantly NNE, NNW, and WNW. The open groundwater conduits and recharge area were inferred from the AM and TM, which are of hydrological significance (Ranganai and Ebinger 2008).

Anderson et al. (2014) demonstrated that aeromagnetic data can be used to understand the 3D distribution of plutonic rocks near the Pebble porphyry copper deposit in southwestern Alaska, USA (Fig. 4). In this study, magnetic inversion was constrained by a near-surface, 3D geological model that is attributed with measured magnetic susceptibilities from various rock types in the region. It was concluded that aeromagnetic data were an effective tool for mapping middle Cretaceous igneous rocks in southwest Alaska and should provide valuable insights during exploration for similar age porphyry copper deposits in the region.

Summary and Conclusions

An aeromagnetic survey is one of the most common airborne geophysical survey methods. AMS infers the underlain geology by measuring and interpreting magnetic anomalies caused by magnetic minerals. There are many applications of AMS in the areas of petroleum and mineral explorations. The applications of AMS in engineering geology include, but are not limited to, near-surface geological mapping, structural geology mapping, aiding 3D geological modeling, groundwater study, environmental study, and geologic hazards assessment.

Cross-References

▶ Aerial Photography
▶ Insar
▶ Lidar

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Aggregate

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Definition

Granular material such as sand, gravel, or crushed stone used for a variety of purposes, but most commonly in relation to construction activities (Bobrowsky 1998).

Context

Natural aggregates (sand and gravel) occur in various deposits, reflecting different processes of erosion, transportation, abrasion, and deposition often representative of local/regional lithologies and all textural classes. The most common aggregate rich environments are fluvial and glacial deposits, but coastal, marine, desert, and other environments can also be exploited. In a fluvial environment, fine-grained aggregates are more common in meandering river and distal high energy streams conditions, whereas coarser aggregates occur in proximal high energy or braided stream conditions. Glacial environments are by nature more variable in sand and gravel composition but somewhat more predictable when relying on facies analysis and sequence stratigraphy (e.g., glaciofluvial deposits).

Crushed stone aggregate is typically derived at a local scale when natural aggregate is unavailable and is largely dependent on the bedrock lithology of the region. Most crushed aggregate is produced for concrete and road construction purposes. Most igneous rocks (like dolerite) and sedimentary rocks such as limestone and dolomite are excellent sources for crushed aggregate. Metamorphic rocks especially those with high cleavage and schistosity are a poor source for aggregate (Langer 1993).

Depending on the final purpose and use, raw aggregate is processed in a number of ways including washing, removal of detrital items, sorting, screening, sieving, etc. The rock type, shape, and texture of aggregate strongly influence the range of uses for the materials. For concrete and bituminous needs, key properties of aggregate include hardness, strength, chemical properties, size gradation, particle shape, contaminant absence, specific gravity, and so on.

Aggregate is primarily used in construction activities but include a number of uses such as concrete, cement and blocks, road asphalt, construction fill, road subgrade, bricks, pipes, roof shingles, railroad ballast, glass, abrasives, filtration beds, fertilizer, lime, metallurgic fluxstone, and so on. The properties of aggregate are critically restrictive for certain uses and are subject to a suite of tests and technical specifications before adoption. In some cases, slag and clinkers are used as aggregate substitutes (Langer 1993).

Natural aggregate is typically mined through open pit operations or by dredging in water-related environments. Crushed stone aggregate mining is commonly achieved by open, pit, or bank quarrying.

Cross-References

▶ Abrasion
▶ Aggregate Tests
▶ Alkali-Silica Reaction
▶ Armor Stone
▶ Bedrock
▶ Blasting
▶ Cap Rock
▶ Chemical Weathering
▶ Coastal Environments
▶ Concrete
▶ Crushed Rock
▶ Dissolution
▶ Durability
▶ Erosion
▶ Fluvial Environments
▶ Gabions
▶ Glacier Environments
▶ Gradation/Grading
▶ Limestone
▶ Mining
▶ Petrographic Analysis
▶ Physical Weathering
▶ Rock Field Tests
▶ Rock Laboratory Tests
▶ Rock Properties
▶ Sand

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Aggregate Tests

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Definition

Tests performed on granular material such as sand, gravel, or crushed stone to determine their composition, characteristics, properties, and suitability for specific uses.

Context

Natural and crushed stone aggregate serves a wide list of needs and uses around the world (Bobrowsky 1998). Most aggregate is for construction purposes, and of this, most is used in Portland-cement concrete and bituminous mixes. Specific uses require specific requirements and attributes for the materials in use. Cement aggregate needs are more rigid and less flexible than, for instance, bituminous needs. To ensure the most appropriate aggregate is used for a specific need, a suite of tests are available to assess the composition, characteristics, properties, and suitability of the aggregate materials. The most common tests include:

Abrasion test – determines the hardness properties of aggregate, most commonly relies on the Los Angeles abrasion test to establish the percentage of wear resulting from the rubbing action of steel balls (abrasive charge) on the aggregate samples. Cast iron spherical balls (48 mm in diameter, ~400 g) are placed in a drum with 5–10 kg of aggregate. After 500 to 1000 revolutions (30 rpm), the sample is sieved, weighed, and compared to the total sample weight to provide a Los Angeles Abrasion index.

Bitumen Adhesion test – a number of tests are available to determine the reliability of adhesion of a bitumen binder to aggregate when water is present. Adhesion and binder stripping problems tend to arise when the bitumen mixture is permeable to water or the aggregate is exposed to wet and cold.

Crushing test – provides an indication of aggregate resistance to crushing under an applied crushing load. The test requires samples of the aggregate to be subjected to standard load crushing conditions. Multiple layers of presieved materials are tamped 25 times before a 40-ton load is applied at a rate of 4 tons per minute. The crushed aggregate is then sieved, weighed, and compared to the original total weight to provide an aggregate crushing value.

Impact test – determines the resistance of aggregates to impact forces. Multiple layers of sieved samples are tamped 25 times before a 14 kg hammer is dropped for a total of 15 blows. The resultant sample is sieved and the weight is compared to the original sample to generate an Impact value.

Shape test – this test provides an indication of the extent of detrimental flaky and elongated materials in the aggregate samples. A Flakiness Gauge is used to define the percentage of particles whose smallest dimension is less than 6/10ths the mean size (by weight). An Elongation Gauge is used to define the percentage of particles whose longest dimension is 1.8 times the mean dimension (by weight).

Soundness test – establishes the resistance of aggregate materials to prolonged weathering action. Sorted aggregate samples are subjected to 5 cycles of wetting (saturated solution of sodium sulfate or magnesium sulfate) and drying (105–110 °C). The weight loss in the sample provides a proxy indication of the disintegration potential of the materials.

Specific gravity/water absorption tests – involve two measures of specific gravity: apparent specific gravity and bulk specific gravity; the former determines SG of aggregate minus voids and the latter SG of aggregate sample including the voids. The water absorption test is simply the difference between the two measures of specific gravity.

Extensive testing can involve any combination of the following assessments: Aggregate Crushing Value, Bulk Density, Chloride sulfate content, Clay and Fine Silt, Color, Flakiness Index, Los Angeles Value, Mean Least Dimension, Organic Impurities, Particle Density/Water Absorption, Particle Shape, Particle Size Distribution, Petrographic Examination, Polished Aggregate Friction Value, Resistance to Stripping, Resistance to Wear, Sieve Analysis, Soundness, Unconfined Compressive Strength, Weak Particles, and Wet/Dry Strength Variation (Barksdale 2013).

Cross-References

▶ Abrasion
▶ Aggregate
▶ Alkali-Silica Reaction
▶ Bedrock
▶ Chemical Weathering
▶ Concrete
▶ Crushed Rock
▶ Dissolution
▶ Durability
▶ Gradation/Grading
▶ Limestone
▶ Petrographic Analysis
▶ Physical Weathering

▶ Rock Field Tests
▶ Rock Laboratory Tests
▶ Rock Properties
▶ Sand

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Alkali-Silica Reaction

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Definition

The reaction between alkali in cement and noncrystalline or porous silica in aggregate, in the presence of moisture, that may cause expansion and cracking of concrete.

Synonyms

"Concrete cancer"

Characteristics

Alkali silica reaction (ASR) is an acid-base reaction between calcium hydroxide (portlandite, $Ca(OH)_2$) and silicic acid (H_4SiO_4) which can be represented as:

$$\begin{aligned} Ca(OH)_2 + H_4SiO_4 &\rightarrow Ca^{2+} + H_2SiO_4{}^{2-} + 2\,H_2O \\ &\rightarrow CaH_2SiO_4 \cdot 2\,H_2O \end{aligned}$$

The alkaline solution reacts with amorphous silica to produce a viscous alkali silicate gel. As the reaction proceeds Ca^{2+} ions are dissolved into the pore water. These react with the gel to form solid calcium silicate hydrate whereas the alkaline solution converts the remaining siliceous minerals into bulky alkali silicate gel. The increase in volume causes expansion pressure, cracking, and spalling that weakens the concrete and sometimes causes failure (Fig. 1).

ASR can be reduced or prevented by:

- Limiting silicate content of the aggregate by avoiding crushed rock or natural gravels containing amorphous or porous silica
- Limiting the alkali metal content of the cement or preventing alkalis from external sources coming into contact
- Adding fine grained siliceous materials as the cement is setting to promote a controlled reaction neutralizing excessive alkalinity

A wide variety of rocks show alkali silica reactivity, depending on the nature of contained silica, including:

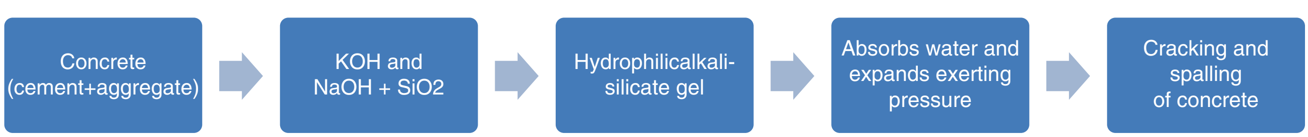
- Acidic and intermediate volcanic rocks (obsidian, rhyolite, dacite, andesite), porphyries and tuffs, granites, and granodiorites
- Shale, slate, sandstone, siltstone, quartzite; siliceous carbonate rocks, graywackes, argillites, chert, and flint
- Phyllites and granitic and grano-dioritic gneisses

Suspect forms of silica are amorphous glasses and opal, porous tridymite and cristobalite and microcrystalline chalcedony. It is therefore important to carry out petrographical examination of potential aggregates and to undertake tests for potential alkali-silica reactivity (Farney and Kirkhoff 2007; Pani et al. 2012).

Examples of tests are:

Chemical methods – crushed samples are reacted with 1 N sodium hydroxide at 80 °C. After 24 h, the amount of silica dissolved from the aggregate and reduction in alkalinity of the solution are measured and plotted against a reference curve to establish whether these fall within one of three ranges: innocuous, deleterious, or potentially deleterious. The test identifies highly reactive aggregates fairly well, but not slowly reactive aggregates, and is helpful rather than reliable.

Bar methods – mortar bars are immersed in NaOH solutions for 14 days, or longer, and changes in length are measured. This is useful for aggregates that react slowly or



Alkali-Silica Reaction, Fig. 1 Sequence of events in alkali silicate reaction expand late in the reaction. However, test conditions do not correspond to those of concrete in service and tend to overestimate aggregate reactivity. The test indicates aggregates that are acceptable but not necessarily those that should be rejected.

Cross-References

- ▶ Aggregate Tests
- ▶ Aggregate
- ▶ Cement
- ▶ Igneous Rocks
- ▶ Rock Laboratory Tests
- ▶ Sedimentary Rocks

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Alteration

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Definition

Alteration is any mineralogic change to a preexisting rock through chemical reaction caused by hot circulating hydrothermal fluids.

Introduction

Hydrothermal fluids, owing to temperature and pressure gradient, travel within a rock's primary or secondary porosity. They react with country rock, alter original mineralogy, and produce new minerals. Hydrothermal fluids can be magmatic, meteoric, marine, or sedimentary (connate) in origin. They carry mobile elements, large ion lithophile elements (Li, Be, B, Rd, Cs), alkalies, alkali earths, and volatiles (Guilbert and Park 1986).

Alteration Processes

The fluids responsible for inducing alteration of minerals may eventually deposit ore minerals as a result of thermal and chemical changes. Therefore, mapping alteration halos is key to discovering hydrothermal mineral deposits that may or may not outcrop on the surface. Alteration is common with porphyry, skarn, and orogenic/magmatic vein-hosted, low-temperature (epithermal), volcanic massive sulfide deposits. Alteration associated with magmatic and sedimentary-hosted deposits does exist but is not very conspicuous (Guilbert and Park 1986). According to Guilbert and Park (1986), common alteration reactions include hydrolysis (a reaction between silicate minerals and either pure water or aqueous solution), hydration (addition of water to produce a new mineral)–dehydration, carbonitization (addition of CO_2 to form carbonate rocks)–decarbonitization (removal of CO_2 from minerals), alkali/alkali-earth replacement (addition of alkali or alkaline Earth metals), silication (replacement or breakdown of silicate minerals by reaction with free silica), silicification (hydrothermal alteration in which quartz, opal, chalcedony, jasper, or other forms of the amorphous silica content of the rock increase), and oxidation (addition of oxygen)–reduction (removal of oxygen). Depending on the chemistry of hydrothermal fluids and the wall rock, various assemblages of alteration mineral products may result. The most common assemblages include potassic (e.g., K feldspar, biotite), propylitic (e.g., chlorite, epidote, calcite), phyllic (e.g., sericite), and argillic (kaolinite, montmorillonite) minerals.

Examples of Alteration Reactions

$3KAlSI_3O_8$ (K feldspar) + 2H = $KAl_3Si_3O_{10}(OH)_2$ (sericite) $+SiO_2$+2 K – hydrolysis reaction

$KAlSi_3O_8$ (K feldspar) + 6.5 Mg +$10H_2O$ = $Mg_{6.5}(Si_3Al)O_{10}(OH)_8$ (chlorite)+ K+12H – hydration reaction

Alteration indices are used to discriminate altered rocks from their unaltered counterparts and to quantify the degree of alteration. The common alteration indices include the Hashimoto, Ishikawa, ACNK, silicification, and chlorite-carbonate-pyrite indices (Harris et al. 2000; Doyle 2001; Van Ruitenbeek et al. 2005). These indices are calculated in terms of enrichment or depletion in mobile elements as shown below:

$(MgO+K_2O/MgO+K_2O+CaO+Na_2O)*100$ – Hashimoto index

$(K_2O+MgO/K_2O+MgO+Na_2O+CaO)*100$ – Ishikawa index

$(Al_2O_3/Na_2O+CaO+K_2O)*100$ – ACNK index

$(MgO+FeO/MgO+FeO+Na_2O+K_2O)*100$ – chlorite-carbonate-pyrite index

$(SiO_2/SiO_2+Al_2O_3)*100$ – silicification index

Cross-References

- ▶ Alkali-Silica Reaction
- ▶ Chemical Weathering
- ▶ Diagenesis
- ▶ Dissolution
- ▶ Fluidization
- ▶ Geochemistry
- ▶ Hydrogeology
- ▶ Hydrothermal Alteration
- ▶ Igneous Rocks
- ▶ Karst
- ▶ Limestone
- ▶ Metamorphic Rocks
- ▶ Mineralization
- ▶ Rock Properties

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Angle of Internal Friction

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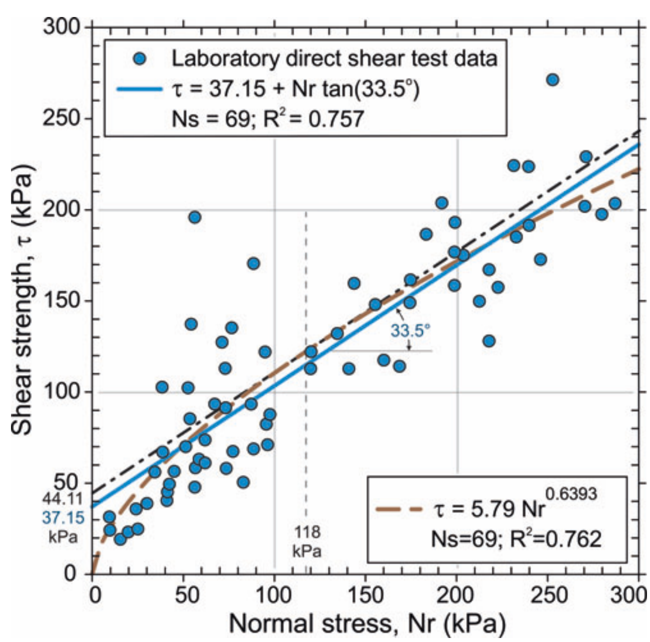
Definition

The angle of internal friction is a physical property of Earth materials or the slope of a linear representation of the *shear strength* of Earth materials.

Earth materials that are unconsolidated and uncemented typically are called *soil* by engineers and geologists and may be called *sediment* by geologists. Soil consists of grains of minerals or rock fragments in a range of sizes (mm to m) from very fine to very coarse (clay, silt, sand, gravel, cobble, and boulder-size). Grains that are chemically and mechanically separate from each other form a mass that can be excavated with relative ease, and the excavated material can be placed in a pile that attains a conical shape with slopes that are at the *angle of repose* (Fig. 1). The angle of repose is a representation of the angle of internal friction; however, it tends to be governed by grain shape such that the slopes of most piles of

Angle of Internal Friction, Fig. 1 Conical pile of crushed Oligocene dolostone at a rock-products quarry in northern Florida, USA (Photo by Jeffrey R Keaton, 24 July 2008)



Angle of Internal Friction, Fig. 2 Graphical representation of Eqs. 1 and 2 (Laboratory data used by Keaton and Ponnaboyina (2014)) loose, dry grains of natural soil are in the range of 28° to 34°. A pile of angular gravel-size grains can attain stable slope angles up to 45°.

Shear strength (τ) of most soil is a function of the confining stress or normal stress (*Nr*), such that it is lower at low normal stress and higher at high normal stress. Samples of alluvial silty medium to coarse sand subjected to direct shear testing might have a linear regression peak shear strength represented by Eq. 1. Eq. 1 describes an angle of internal friction (ϕ) of 33.5° and a *cohesion intercept* of 37.15 kPa. A silty medium to coarse sand with nonplastic silt would be cohesionless. A two-parameter power function regression (Eq. 2) of the same direct shear test data shows a variable angle of internal friction and forces the cohesion intercept to zero (Fig. 2), which is appropriate for sandy soil.

$$\tau = 37.15 + 0.662\,Nr = 37.15 + Nr\tan\left(33.5^{\circ}\right) \quad (1)$$

$$\tau = 5.79\,Nr^{0.639} \quad (2)$$

The friction angle (ϕ) for the power function regression equation matches the linear regression at a normal stress value of approximately 118 kPa; however, the cohesion intercept for the tangent to the power function regression at this normal stress is 44.11 kPa. Earth materials are known to exhibit nonlinear strength and deformation behavior; this example demonstrates the nonlinear strength aspect. The shape of the coarse sand grains creates an equivalent *roughness* in the sample and is responsible for much of the nonlinear character in its shear strength.

The angle of internal friction is determined in a laboratory environment using a direct shear test or triaxial compression test.

Cross-References

- ▶ Angle of Repose
- ▶ Mohr Circle
- ▶ Mohr-Coulomb Failure Envelope
- ▶ Shear Strength
- ▶ Soil Mechanics
- ▶ Soil Properties

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Angle of Repose

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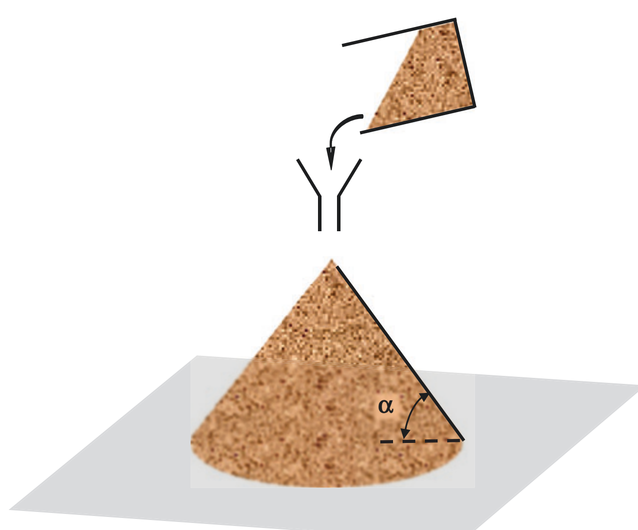
Synonyms

Critical angle of repose; Dynamic angle of repose

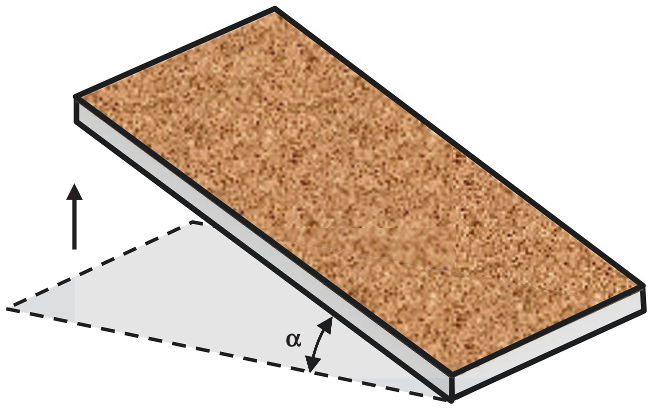
Definition

Theoretically, the maximum angle at which grains in a heap of sediment can remain in place without becoming unstable. The angle of repose of granular materials is relevant in various applications of science and engineering, such as sedimentology, geomorphology, hydraulic, and chemical engineering. It can be used in the descriptions of initial motion of sediment grains and sediment transport process, and in the investigations of bank stability, riprap protection, and reservoir sediment removal.

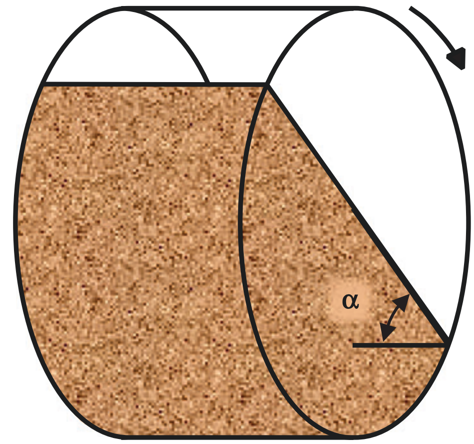
The angle of repose is measured using different methods (Carrigy 1970; Francis 1986), of which some are illustrated in Figs. 1, 2, 3, and 4. For example, it can be determined by pouring sand grains to form a conical pile (Fig. 1). Two different slopes can be differentiated during the pile formation. As grains are gradually added to a heap, they can pile up to an upper angle of repose. Once masses slump, a new



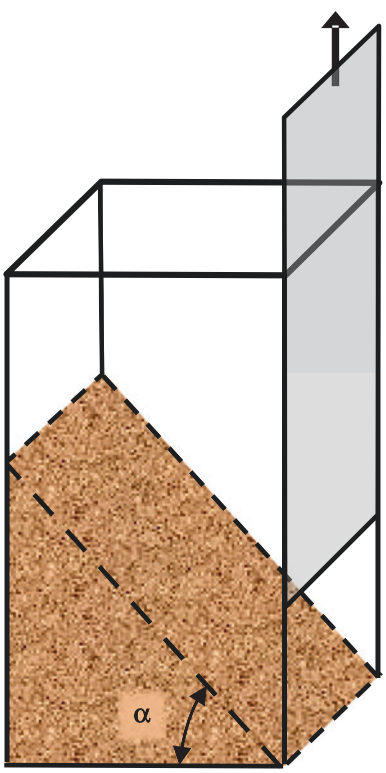
Angle of Repose, Fig. 1 Conical heap (or pile). Particles are slowly poured through a funnel at a fixed height, and a pile is thus formed. The side slope of the pile is measured as the angle of repose



Angle of Repose, Fig. 2 Tilting box. A box of particles with a plane surface is slowly tilted up until some particles are about to move downward along the slope. This critical slope is measured as the angle of repose



Angle of Repose, Fig. 4 Rotating drum. A drum is partially filled with particles and slowly rotated. This method is used to measure the upper, lower and dynamic angle of repose



Angle of Repose, Fig. 3 Removal of side support. With a box of particles, an avalanche will occur when the side wall is removed, forming the angle of repose

surface will form at a lower angle of repose. As a result, the angle of repose varies repeatedly during the growth of the pile. The upper angle is associated with the onset of slope instability, and the lower angle is associated with the cessation of slope instability. The upper angle of repose may also be measured as a critical angle using a tilting box for which some grains start to roll down along the inclined surface (Fig. 2). In comparison, the lower angle of repose is achieved at the end of an avalanche, which can be generated by the removal of support for loose material (Fig. 3). The angle of repose can also be measured by draining grains through a bottom opening of a container by building up a cone over a fixed base.

A rotating drum can be used to measure three different angles of repose (Fig. 4). When the drum rotates, a series of variations can be observed in the slope of the free surface of grains. At a very low rotating speed, grains move together with the drum, demonstrating a rigid body motion until the slope reaches its upper angle. Then, an increase in the slope angle triggers an avalanche, transporting grains down the slope. At the end of the avalanche, a new slope forms at a lower angle. If the rotating speed is increased, both upper and lower angles disappear and the slope angle approaches a constant as grains keep rolling down the slope. This indicates the beginning of the rolling stage. The corresponding slope angle is referred to as the dynamic angle of repose. At this stage, grains move continuously from the upper to lower end of the slope, yielding a surface shear layer of grains that flow down the plane inclined at a fixed angle.

Theoretically, the angle of repose can be considered the maximum angle at which grains can remain in place without becoming unstable. Unfortunately, confusions often exist in the differentiation among the different angles of repose and thus the use of the term of angle of repose in the literature.

For example, Simons and Senturk (1992) stated that the angle of repose is the angle of slope formed by particulate material under the critical equilibrium condition of incipient sliding. Soulsby (1997) applied the term of the angle of final repose for the angle of the lee slopes of dunes and the angle of slope of the conical scour around a circular vertical pile, which is observed at the end of avalanching. Garcia (2008) considered the angle of repose as a slope angle beyond which spontaneous failure of the slope occurs. An early differentiation between the upper and lower angles of slope was made by Bagnold (1966), who called the upper angle the apparent limiting static friction angle of initial yield and the lower angle the residual angle. Allen (1969) described the upper angle as the angle of initial yield and the lower angle as the residual angle after shearing. Carrigy (1970) noted that there is no agreement reached as to what angle should be measured. Francis (1986) indicated that some confusion exists in the precise meaning of the term "angle of repose," and a single angle of repose is inadequate to explain all observable characteristics of many scree slopes.

Cross-References

▶ Aeolian Processes
▶ Angle of Internal Friction
▶ Geohazards
▶ Landslide
▶ Mining
▶ Noncohesive Soils
▶ Physical Weathering
▶ Sand
▶ Sediments
▶ Shear Stress
▶ Silt
▶ Strength

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A

Aquifer

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Definition

Aquifer (from Latin *aqua* water and *ferre* to bear, to carry) is a layer or a layered sequence of rock or sediment, comprising one or more geological formations that can store and transmit significant quantities of water under an ordinary hydraulic gradient.

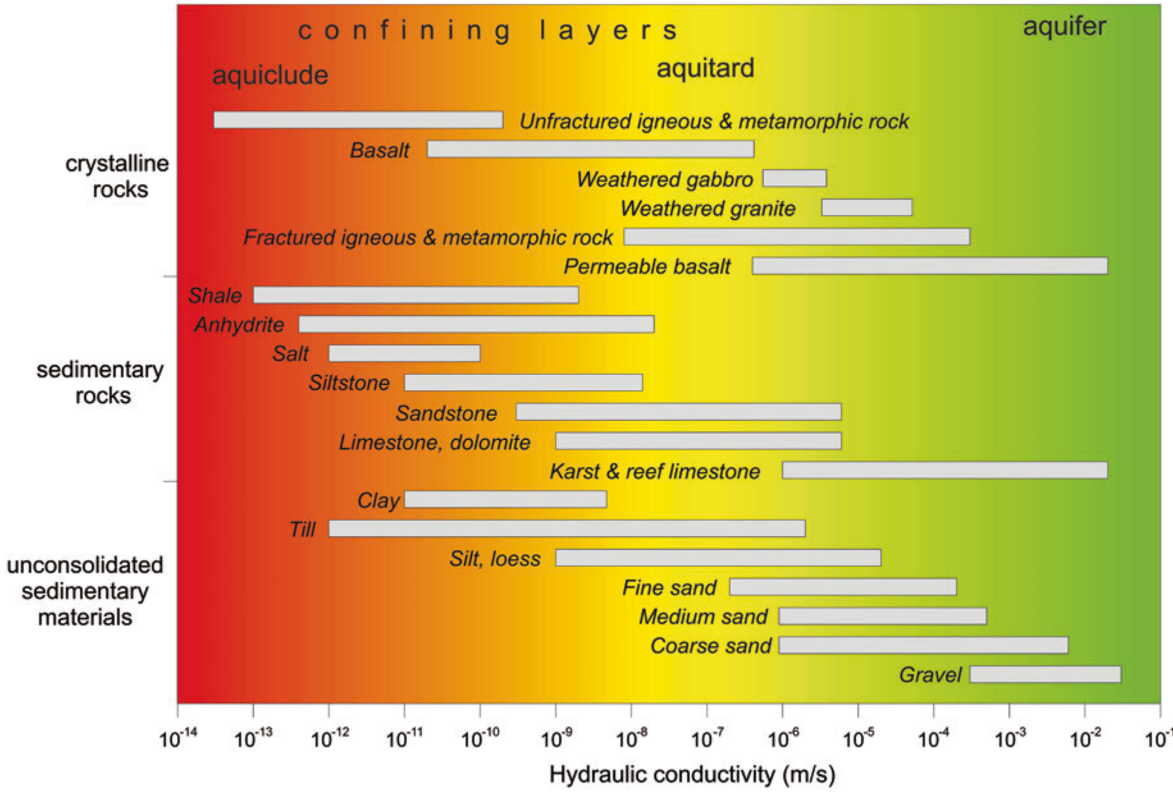
Aquifer also includes the unsaturated part of the permeable material, that is, the part above the water table, as well as the saturated part. The sole saturated part of an aquifer, or the part from the aquifer bottom to the water table is referred to as the "effective" aquifer Price (1996).

Characteristics

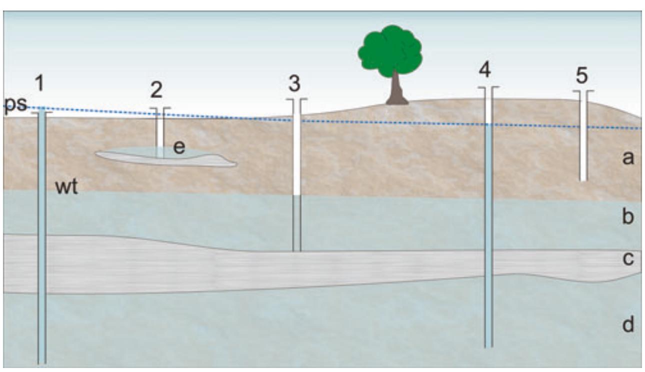
The most important characteristics of aquifers are the storing capacity or capability to store water in the rock voids (pores and fractures) and the hydraulic conductivity, which is the capability of water to move through the solid matrix. The hydraulic conductivity derives from the interconnected pores of unconsolidated sediments (sand and gravel) or by the fracture network of consolidated sedimentary (sandstone, limestone), igneous, and metamorphic rocks. Some materials, such as clay, can have a high porosity, but the scarce interconnection between pores makes such clay-rich rocks quite impermeable, thus creating a confining layer.

Hydraulic conductivity values for different kinds of rocks and sediments are summarized in Fig. 1. But, field values strongly depend on the fine grain component, interbedding within finer layer in sedimentary aquifers, grade and extent of fracturing of rocks.

Regarding the confining layer (those layers with lower hydraulic conductivity), aquifers can be classified as either **unconfined** or **confined** (Fig. 2):



Aquifer, Fig. 1 Range of hydraulic conductivity for different kinds of rocks and their potential hydraulic function: aquifer (green), aquitard (yellow), aquiclude (red). The latter are generally referred as confining layers. Range of hydraulic conductivity after Domenico and Schwartz (1997)



Aquifer, Fig. 2 Unconfined, confined, and perched aquifers. (**a**) Unsaturated part of the unconfined aquifer; (**b**) saturated part of the unconfined aquifer; (**c**) confining layer or aquiclude; (**d**) confined aquifer; (**e**) perched aquifer; ps potentiometric surface of confined aquifer; wt water table of the unconfined aquifer. Well 1 enters the confined aquifer. At this point the head exceeds the ground level, providing a free flowing (or artesian) well. Well 2 enters a small perched aquifer; because of its limited extent, it could result as transient perched aquifer that exists only after a period of infiltration. Well 3 enters the unconfined aquifer and intercepts the water table; Well 4 enters the confined aquifer. At this point the hydraulic head does not exceed the ground, thus the artesian scenario does not occur. Well 5 enters the unsaturated part of the unconfined aquifer. The well is dry, but seasonal recharge can raise the wt up the well, providing the same scenario of well 3

– Unconfined aquifer (also known as *phreatic* aquifer, or water-table aquifer) is one where the water table occurs within the aquifer layer. In this type of aquifer, the upper limit of saturation (the water table) is at atmospheric pressure, and at any depth below the water table, the pressure is greater than the atmospheric pressure and at any point above the water table (capillary zone), the pressure is less than atmospheric pressure. The hydraulic heads measured in wells in an unconfined aquifer define a potentiometric surface that coincides with the water table, the upper limit of saturation. In plan view, the water table surface is a contour map showing a horizontal distribution of heads in the aquifer. Groundwater pathways are perpendicular to the contour lines (Fig. 2).
– In a confined aquifer, the entire thickness of the aquifer layer is saturated and there is a confining layer at the top of the aquifer. At any point in confined aquifers, the water pressure is greater than atmospheric pressure, consequently the water level in a monitoring well in a confined aquifer rises above the top of the aquifer (Fig. 2). The hydraulic heads measured in wells in a confined aquifer define the potentiometric surface, an imaginary surface that does not coincide with the physical top of the aquifer.

Natural occurring scenarios can be more complex, comprising interbedded systems of permeable (aquifers) and (leaky) confining layers to form a multilayered aquifer system (see also ► Aquitard) Celico (1986).

In some heterogeneous settings, lenses/layers of less conductive materials can occur above the water table. Infiltrating water can be held up by those layers that form the base of saturated perched zones, known as perched aquifers (Fig. 2). If the lens is extensive, the body of perched water may be thick enough to allow a water supply well to be tapped without drilling deeper to the regional water table.

Usually aquifers are recharged by meteoric water that is rainfall infiltrating into the ground in the recharge areas within the normal hydrological cycle. Less common aquifers can bear connate water that represents water trapped in the pores of a rock during formation of the rock.

Coastal aquifers can be hydraulically connected with seawater. When fresh groundwater approaches the coastline, its flow can be hampered by sea water. The fresh water/seawater contact is made up of an interface (or more exactly a progressive mixing zone) whose position is determined by the difference in hydraulic load between the water table and the middle sea level. This interface is a natural and dynamic balance, over-pumping of freshwater can lead to a progressive intrusion of seawater into freshwater aquifers causing salinization of potable freshwater supplies. The thickness of freshwater topping the seawater can be computed by the Ghyben-Herzberg equation.

Cross-References

- ► Aquitard
- ► Artesian
- ► Clay
- ► Fluid Withdrawal
- ► Groundwater
- ► Hydraulic Action
- ► Hydrogeology
- ► Liquefaction
- ► Piezometer
- ► Saturation
- ► Voids
- ► Wells

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Aquitard

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Synonyms

Confining layer; Leaky confining layer

Definition

An aquitard is a leaky confining bed that transmits water at a very slow rate to or from an adjacent aquifer.

Characteristics

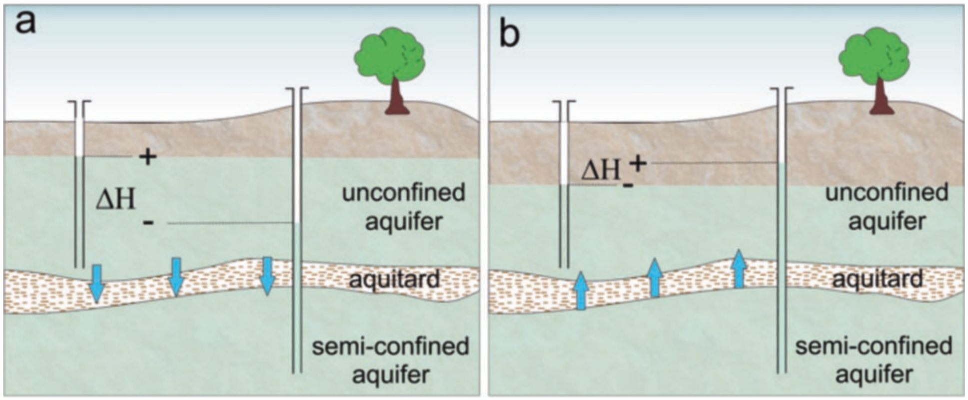
Due to the reduced hydraulic conductivity, aquitards permit water to move through at very slow rate as compared to the adjacent aquifers. In particular they permit the vertical (upward or downward) flow of water between underlying and overlying aquifers, depending on the hydraulic gradient. Aquitards differ from aquicludes in that the latter prevent water transmission and can act as a barrier to regional groundwater flow.

Aquitards may transmit quantities of water that are significant in terms of regional groundwater flow, but from which negligible supplies of groundwater can be obtained. Examples of aquitards include fluvial, glaciofluvial, and lacustrine deposits, or poorly fractured sedimentary and crystalline rock.

Water flow through aquitards depends on the hydraulic conductivity and thickness of the aquitard as well as the head difference between the adjacent aquifers. As a result of pumping or seasonal recharge, the hydraulic heads and therefore the groundwater motion can change during these periods (Fig. 1). When the aquifer underlying the aquitard is capable of exchanging water through the aquitard, it is known as a semi-confined aquifer.

Cross-References

- ► Aquifer
- ► Artesian
- ► Clay
- ► Fluid Withdrawal
- ► Groundwater



Aquitard, Fig. 1 Semi-confined aquifer and groundwater motion through aquitard. In (**a**) the head of unconfined aquifer exceeds the head of the aquifer underlying the aquitard. Groundwater moves from unconfined aquifer to semi-confined aquifer through the aquitard. In (**b**) the hydraulic head of the semi-confined aquifer exceeds the head of unconfined aquifer, therefore groundwater moves upwards into the unconfined aquifer. For small ΔH (difference of head), the aquitard can act as a confining layer. Actions that increase ΔH (e.g., seasonal recharge, pumping) lead to higher exchanges through the aquitard. This means that pumping one layer of this leaky system has measurable effects in layers other than the pumped layer. The resulting drawdown in each layer is a function of several parameters, which depend on the hydraulic characteristics of the aquifer layers and those of the aquitards

► Hydraulic Action
► Hydrogeology
► Liquefaction
► Piezometer
► Saturation
► Voids
► Wells

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Definition

Armour stone is a general term used to refer to a range of natural (and sometimes artificial) stone applications used for wave protection of shorelines and *erosion* protection of streambanks from the eroding action of waves and flowing water as well as in *retaining walls* and slope buttressing related to construction.

Some applications use “armour stone” to refer to boulder-size blocks of durable natural rock material. Applications of armour stone commonly are in the form of revetments but can

Armor Stone, Fig. 1 Breakwater armoured by blocks of Jurassic metavolcanic rock quarried and brought in by barge to protect a marina at Port of Long Beach, California, USA (Photo by Jeffrey R Keaton, September 6, 2016)

be of a variety of shapes and positions relative to the shorelines or *channel banks*, such as used for breakwaters (Fig. 1), groynes, and blankets (CCAA 2008). The armour stone can be blocks and fragments that range in sizes, usually to a specified *gradation* that are dumped into place or they can be uniform blocks that are carefully stacked (NRCS 2007). Armour stone applications are designed for minimal maintenance; consequently, the *durability* of the stone fragments has high importance.

Armour stone material is selected for its size, mass, and durability, and sometimes for its shape, as is the case for stacked blocks. Armour stone is also called “quarry stone” because the sizes required must be extracted by *blasting* rock formations. Defects in the *rock mass*, such as bedding, joints, faults, and dykes, must be characterized for evaluating the likely range of sizes of durable rock material that might be produced from a prospective quarry. Sandstone formations with shale partings tend to be less desirable for use as armour stone than thick-bedded sandstone formations. Certain applications of armour stone, such as around bridge piers in river channels where it may be called "riprap," may be exposed to forces of turbulent clear-water flow with little suspended sediment. Other applications may be in a coastal environment and exposed to high-energy waves on beaches composed of gravel and cobbles. The high-energy beach environment exposes armour stone blocks to *abrasion* and wear by *attrition*. Tests for durability of armour stone material range from simple tests, such as wetting-drying, freezing-thawing, sodium sulfate soundness, and slake durability, to more elaborate tests developed for concrete aggregate, such as Los Angeles abrasion that involves pounding by steel balls in a rotating drum.

Armour stone is popularly used in landscape design as retaining walls and buttressing of slopes where erosion protection from waves or flowing water may not be primary.

Cross-References

▶ Aggregate
▶ Aggregate Tests
▶ Boulders
▶ Coastal Environments
▶ Current Action
▶ Durability
▶ Erosion
▶ Fluvial Environments
▶ Gradation/Grading
▶ Hydraulic Action
▶ Levees
▶ Marine Environments
▶ Nearshore Structures
▶ Retaining Structures
▶ Stabilization

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Artesian

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Definition

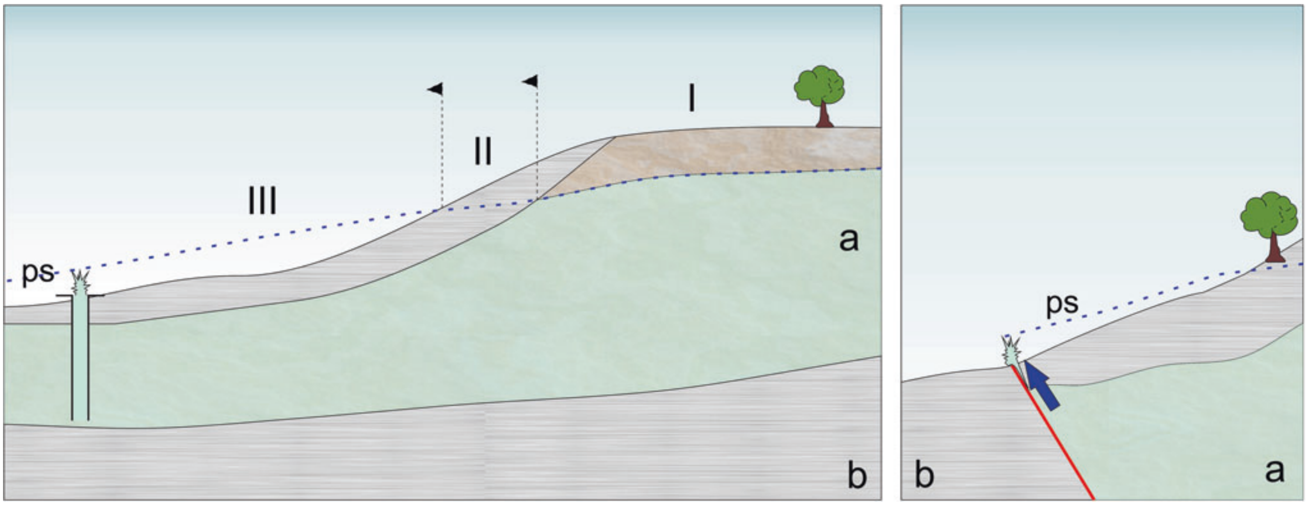
When the water level from a well within in a confined aquifer rises above the level of the ground surface and flows freely without pumping, it is called "artesian" (Fig. 1). The aquifer in such cases is called an **artesian** aquifer and the well is called an artesian well. The word "artesian" comes from "Artois", a region of northern France, where free-flowing wells have been known since the Middle Ages. Artesian wells are most common at the base of slopes in hilly terrain, where high heads within the uplands can induce strong upward hydraulic gradients (Price 1996).

Some authors use the term "artesian" for all confined aquifers, where the potentiometric surface exceeds the saturation level, whether or not the potentiometric surface reaches the ground level.

Characteristics

Instead of artesian aquifers or wells, one should refer to artesian conditions. As represented in Fig. 2, an aquifer can display different features (unconfined, semi-confined, confined, artesian) in different points as determined by confining layers and the topographic surface. The artesian scenario usually depends on the occurrence of confining layers,

Artesian, Fig. 1 A freely flowing monitoring well. At this point, the potentiometric surface of the confined aquifer beneath is higher than the topographic surface



Artesian, Fig. 2 (Left) In section I, aquifer a is in an unconfined condition: the potentiometric surface ps coincides with the water table. In section II the aquifer is confined by the impermeable formation b; the ps does not coincide with the saturate surface. In section III the ps runs higher than the topographic surface, bearing the artesian conditions. A well drilled in this section is freely flowing. (Right) Artesian spring. The confining layer is cut by a fault that provides a permeable path along which groundwater can discharge from a confined aquifer

morphological conditions, and other factors that vary with time such as the recharge/discharge (pumping) pattern that controls the hydraulic head (Fig. 2). For these reasons at any given place, the artesian features may temporarily disappear.

Transient artesian conditions related to earthquakes have been reported in some wells (e.g., Wang and Manga 2014). Earthquakes may affect the state of the subsurface (stress–strain of the aquifer) and therefore can affect the potentiometric surface. As well, for the same reasons, water flow from artesian wells may stop due to earthquakes (Chenglong et al. 2011). Changes in the hydraulic head related to earthquakes may or may not be recovered over time.

In some cases, the high hydraulic head that leads to artesian conditions is a result of a more complex fluid system that includes the groundwater and gases (e.g., CO_2, CH_4).

The Australian “Great Artesian Basin” is the largest artesian basin in the world. It includes over 1,700,000 km^2 and provides the only source of fresh water through much of inland Australia.

Cross-References

- ▶ Aquifer
- ▶ Aquitard
- ▶ Clay
- ▶ Fluid Withdrawal
- ▶ Groundwater
- ▶ Hydraulic Action
- ▶ Hydrogeology
- ▶ Liquefaction
- ▶ Piezometer
- ▶ Saturation
- ▶ Voids
- ▶ Wells

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Artificial Ground

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Synonyms

Artificially modified ground; Anthropogenic (human-made) ground; Technogenic ground

Definition

Areas in which anthropogenic activities have directly modified the landscape by deposition and associated excavation.

Introduction

Artificial Ground generally refers to landforms and sediments deposited or excavated on or within the shallow ground surface, but such modifications can also be created at deeper levels in the subsurface where they cross-cut existing natural geological strata. This novel sedimentary environment includes areas of the excavation, transport and deposition of natural geological materials, and the deposition of novel materials related to activities such as urban development, mineral exploitation, waste management, and land reclamation. Anthropogenically induced deposition of natural sediments, such as behind dams, or erosion, such as through deforestation, is not considered as Artificial Ground.

Humans have already modified more than 50% of the ice-free land surface of the world but the distribution is uneven. Urban areas, which form only about 1–3% of the land surface, have extensive anthropogenic deposits associated with the development of the city landscape and infrastructure. Typically, the need to rebuild cities is facilitated by building upon the debris of older constructions, artificially increasing land levels and creating environments that potentially favor the preservation of earlier urban strata. Rural areas normally have much less Artificial Ground cover, with much of the remaining land surface associated with agriculture or forestry, with subordinate development of rural settlements, transportation networks, and mineral extraction sites. The agricultural disturbance of soil horizons through ploughing and land improvement such as applications of fertilizers and land drainage is commonly excluded from most concepts of Artificial Ground, as is the extensive trawling and dredging followed by resettling of sea-bed sediments on continental shelves.

The modifications associated with Artificial Ground were initiated in many parts of the world thousands of years ago. However, first through the Industrial Revolution, starting in the late eighteenth century in the UK and subsequently the global Great Acceleration of the mid-twentieth century, the rate of accumulation, the extent of reach of such landscape modification, and diversity of composition of the deposits have increased dramatically. This contribution considers how this anthropogenic landscape is described and classified by geologists, how such modified ground is characterized through site investigation, the nature of hazards and resources associated with Artificial Ground, and how engineering investigation, design, and construction can be used to manage risk.

Classifications

In an engineering geology context, “highly variable artificial deposits” are defined as engineering soils. Such deposits can be markedly heterogeneous in composition, depth, and geotechnical properties, ranging from fine to coarse soils, from very soft to hard cohesive soils, and very loose to very dense noncohesive soils. Though commonly ungraded (e.g., landfills), there are circumstances where such deposits may be uniformly graded as part of the production process (e.g., pulverized fuel ash waste, road sub-base), producing homogeneous deposits. Fill is a general term commonly used in site investigations to describe artificial deposits. It can be used to denote material used to infill a void or cavity or in the case of constructional fill material that is added on to the natural ground surface. Several schemes have evolved to provide a means of classifying “fill” into units with distinct characteristics, as carried out for natural rocks and superficial deposits. Examples are described below.

The European Waste Catalogue provides a hierarchical inventory of 20 categories of anthropogenic materials that can be considered wastes, based upon the material and the source industry. Each category has a two-digit code between 01 and 20. This list of wastes was established by Commission Decision 2000/532/EC (Table 1) and provides an established means of categorizing distinct lithological components of artificial deposits.

A classification of anthropogenic strata discriminating materials and boundaries primarily in terms of the time and manner of deposition was proposed by Nirei et al. (2012). This Geo-Stratigraphical Unit Investigation Method recognizes “*chrono-layers*” of materials laid down in a single event and “*material layers*” laid down in one or more depositional events, “*bundles*” of several adjacent chrono-layers, and “*associations*” representing the whole assemblage of units at a site. Their boundary between anthropogenically modified layers and underlying natural geological deposits is referred to, in Japan, as the Jinji unconformity or discontinuity. The scheme provides a hierarchical approach designed to be used to describe anthropogenic successions in detailed site investigations, and is not suitable for the wider classification of Artificial Ground carried out by geologists (Ford et al. 2014).

There are numerous geological approaches to the classification of artificially modified ground. Technogenic deposits (Peloggia et al. 2014) occur as subaerial, subaqueous, and subterranean environments, with distinctions in genesis, composition, and morphology into deposit types, which include both entirely artificial deposits and natural deposits that have been anthropogenically modified or influenced. The scheme of Peloggia et al. (2014) recognizes technogenic grounds that are classified into four main categories (Table 2): aggraded ground, degraded ground, modified ground, and mixed ground, each of which is subdivided into specific genetic types. Aggraded ground includes built-up deposits (made ground and infilled ground), induced alluvial and colluvial deposits, and remobilized deposits. Degraded ground includes induced erosion, such as eroded, slipped, or sunken

Artificial Ground, Table 1 Main categories of wastes recognized in the European Waste Catalogue (Official Journal of the European Communities 2000)

Code	Waste type
01	Wastes resulting from exploration, mining, dressing and further treatment of minerals and quarry
02	Wastes from agricultural, horticultural, hunting, fishing and aquacultural primary production, food preparation and processing
03	Wastes from wood processing and the production of paper, cardboard, pulp, panels and furniture
04	Wastes from the leather, fur and textile industries
05	Wastes from petroleum refining, natural gas purification and pyrolytic treatment of coal
06	Wastes from inorganic chemical processes
07	Wastes from organic chemical processes
08	Wastes from the manufacture, formulation, supply and use (MFSU) of coatings (paints, varnishes and vitreous enamels), adhesives, sealants and printing inks
09	Wastes from the photographic industry
10	Inorganic wastes from thermal processes
11	Inorganic metal-containing wastes from metal treatment and the coating of metals, and non-ferrous hydrometallurgy
12	Wastes from shaping and surface treatment of metals and plastics
13	Oil wastes (except edible oils, 05 and 12)
14	Wastes from organic substances used as solvents (except 07 and 08)
15	Waste packaging; absorbents, wiping cloths, filter materials and protective clothing not otherwise specified
16	Wastes not otherwise specified in the list
17	Construction and demolition wastes (including road construction)
18	Wastes from human or animal health care and/or related research (except kitchen and restaurant wastes not arising from immediate health care)
19	Wastes from waste treatment facilities, off-site waste water treatment plants and the water industry
20	Municipal wastes and similar commercial, industrial and institutional wastes including separately collected fractions

ground, as well as excavated ground. Modified ground comprises chemically and mechanically modified soils.

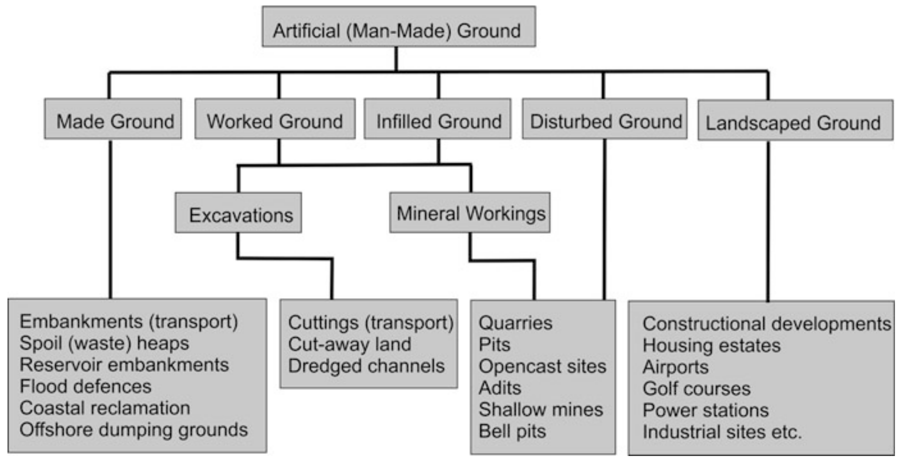
Howard (2014) proposed a twofold classification system of *anthrostratigraphic units* (ASU) and *technostratigraphic units* (TSU). ASUs comprise a stratiform or irregular body of anthropogenic origin distinguished on the basis of lithological characteristics and/or bounding disconformities. The basic ASU is the *anthrostratum*, defined as a mostly stratiform body of artificially mixed earth (rock, sediment, soil, etc.) and artifactual (brick, concrete, etc.) materials, which may be grouped to form an *anthroformation* if regionally mappable. TSUs comprise a stratiform body of anthropogenic origin defined on the basis of human artifacts. The basic TSU is the *technozone*, defined by the commercial ranges of certain artifacts. The technozones are comparable to the ranges shown for "technofossils" (archaeological remains of the future) as illustrated by Ford et al. (2014). Howard (2014) recognized three distinct technozones, equivalent to biotic biozones: Late Pre-Industrial Technozone (1850–1900), Early Industrial Technozone (1900–1950), and Late Industrial Technozone (1950–Present).

Artificial Ground, Table 2 Classification of technogenic ground by Peloggia et al. (2014)

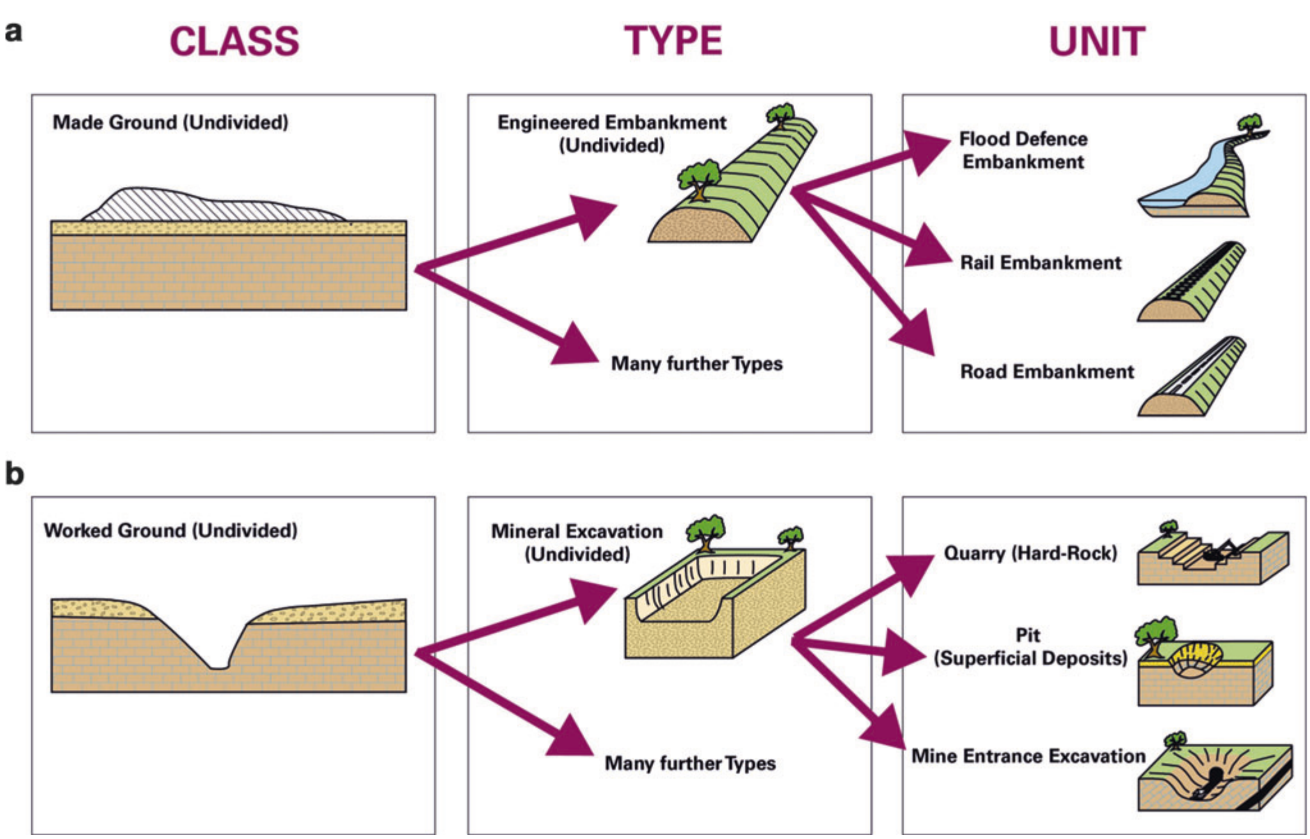
Class	Types	Technogenic layer or feature
Aggraded technogenic ground	Made ground	Technogenic built-up deposits
	Infilled ground	Technogenic built-up deposits covering worked ground
	Technogenic sedimentary or wash ground	Induced alluvium-like sedimentary technogenic deposits
	Colluvial technogenic slope ground	Induced colluvium-like technogenic deposits
	Displaced ground	Remobilized technogenic deposit
Degraded technogenic ground	Eroded ground	Erosion scars due to induced processes
	Slipped or scarred ground through landslides	Slope mass movement scars due to induced processes
	Sunken or disturbed ground	Subsidence sinkholes due to induced processes
	Excavated or worked ground	Excavation surfaces
Modified technogenic ground	Chemically modified ground	Contaminated soil horizons
	Mechanically modified ground	Compacted or revolved soil horizons
Mixed technogenic ground	Complex ground	Complex technogenic profiles
	Layered ground	Composed technogenic profiles

A stratigraphical classification scheme that has been developed for geological maps and models of artificially modified ground in the UK is largely based on morphogenetic attributes (McMillan and Powell 1999; Rosenbaum et al. 2003; Ford et al. 2010). The scheme defines five main classes, each of which may in turn be subdivided with increasing levels of detail into types and units reflecting different genetic forms (Figs. 1, 2, and 3):

- Worked ground: ground artificially cut away or excavated (quarries, pits, rail and road cuttings, dredged channels, etc.).
- Made ground: artificially deposited ground (engineered fill, flood defenses, spoil heaps, coastal reclamation fill; offshore dumping grounds; constructional fill or landraise, etc.).
- Infilled ground: ground artificially cut away and later infilled (back-filled workings such as pits, quarries, opencast sites, landfill sites).



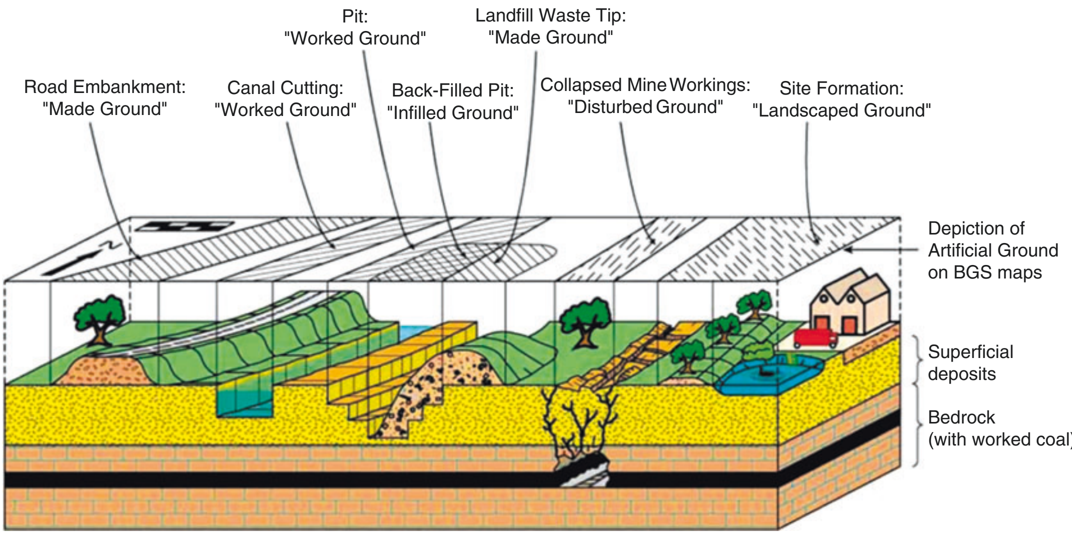
Artificial Ground, Fig. 1 Types of Artificial Ground (Modified from McMillan and Powell 1999)



Artificial Ground, Fig. 2 Selected examples of the enhanced classification scheme for Artificial Ground (From Ford et al. 2010)

- Landscaped ground: extensively remodeled ground, where it is impractical to delineate areas of worked ground and made ground within it (housing estates or golf courses).
- Disturbed ground: mineral workings where excavations and associated subsidence are complexly associated with each other (collapsed bell pits, shallow mine workings, etc.).

This scheme evolved to help recognition of potential geohazards, e.g., contamination, subsidence, instability, with an emphasis on industrial workings and deposits. This is particularly so in schemes such as that proposed by Norbury (2010), which distinguished three categories of human-made soils for engineering geological purposes: (1) Fine or coarse soils composed of natural materials that have been laid down (redeposited) anthropogenically to form new structures like embankments; (2) those fine or coarse manufactured or processed materials laid down anthropogenically that can be described and tested geotechnically as they are physically or chemically similar to natural soils; and (3) those fine or coarse



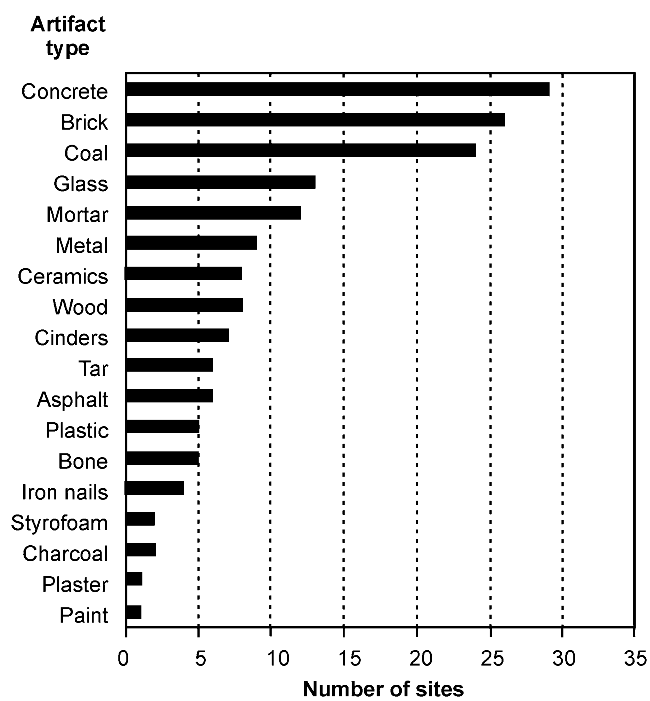
Artificial Ground, Fig. 3 Schematic view showing examples of the main classes of Artificial Ground and their representation on topographic maps (From Ford et al. 2010)

manufactured materials laid down anthropogenically that cannot be easily geotechnically described (e.g., municipal and commercial refuse in landfill, fly-tipped material, and demolition waste). Human intervention and disturbance in the subsurface could also be considered a form of Artificial Ground (Ford et al. 2014; Zalasiewicz et al. 2014).

History of Development, Types, and Distribution of Deposits/Critical Materials

Foundations of urban settlements: Landscaping and earthworks for the construction of buildings are included as Artificial Ground, as is building rubble, whereas "extant" buildings are generally excluded (Ford et al. 2014). Urban areas, where ground has been modified for residential, commercial, and cultural purposes ("Brownfield sites"), historically preserve artificial deposits comprising building materials such as bricks and mortar, crushed rock, dimension stone, and roofing slates (Ford et al. 2014). The modern building materials of choice include concrete, concrete-masonry units (breeze or cinder-block), reinforcing steel (rebar), and glass. Modern towns and cities are associated with development of facilities, many extending to depths of ~20 m in areas of office spaces, museums, underground car parks, petrol station storage tanks, warehousing, etc. (Evans et al. 2009). This subsurface zone of human interaction includes complex networks of public utility pipelines/cables supplying electricity, gas, water, sewerage extraction, and telecommunication facilities (Zalasiewicz et al. 2014).

Redevelopment of urban areas results in demolition waste in the UK typically comprising ~57% by weight masonry, 37% concrete, and 2% timber, with the remainder of gypsum and plaster products, ferrous metals, and small amounts of nonferrous, metals, glass, asbestos products, etc., whereas construction waste may contain 45% soil (Douglas and Lawson 2001). Artifacts recorded at former demolition sites in Detroit, USA (Howard 2014), are typically waste building materials and coal-related wastes (Fig. 4). This Detroit study showed that urban demolition waste can be readily



Artificial Ground, Fig. 4 Type and abundance of artifact types found in urban soils within demolition sites in Detroit, USA ($n = 68$) (From Howard 2014)

A

Artificial Ground, Table 3 Properties of artificial deposits at former demolition sites compared with natural superficial deposits in Detroit, USA (From Howard 2014)

Site	Artifact content (%)	Texture			Exchangeable bases (cmol/kg)				BS (%)	Reaction to acid	pH	OM (%)
		Sand (%)	Silt (%)	Clay (%)	Ca	Mg	K	Na				
Anthropogenic deposits												
1	30–50	66	14	20	17.1	0.24	1.07	0.04	100	Strong	7.97	6.3
2	30–50	54	27	19	16.2	0.22	2.63	0.03	98	Strong	8.08	2.6
3	30–50	46	33	21	18.9	0.20	1.64	0.05	100	Strong	7.77	3.8
4	10–30	34	36	30	22.3	0.18	2.28	0.15	98	Strong	8.04	1.9
5	10–30	39	40	21	19.0	0.18	1.19	0.06	97	Strong	8.28	1.7
Native Quaternary sediments (natural superficial deposits)												
1	0	91	8	1	11.7	0.52	0.01	0.02	74	Strong	7.94	2.2
2	0	97	2	1	0.51	0.12	0.04	0.01	39	None	6.5	0.1
3	0	98	1	1	0.54	0.10	0.06	0.01	67	Weak	6.9	0.1
4	0	89	10	1	0.67	0.11	0.04	0.01	9	None	5.2	0.1
5	0	93	6	1	0.19	0.12	0.03	0.02	100	Strong	5.4	0.3

BS base saturation; *OM* organic matter content

distinguished from natural surficial deposits. A higher clay and organic matter content within the artificial deposits reflects the mixing of soils and sediments during fill emplacement (Table 3). Higher values of pH, base saturation, and exchangeable Ca and Mg probably reflect the presence of calcareous materials such as mortar and concrete. Exchangeable K is higher in the artificial deposits because of the presence of unweathered parent materials, in contrast to natural soils in which fixation by weathered mica has reduced plant-available K to low levels (Howard 2014). These urban strata are also distinguished by elevated levels of lead and other common toxic metals and metalloids (Howard 2014).

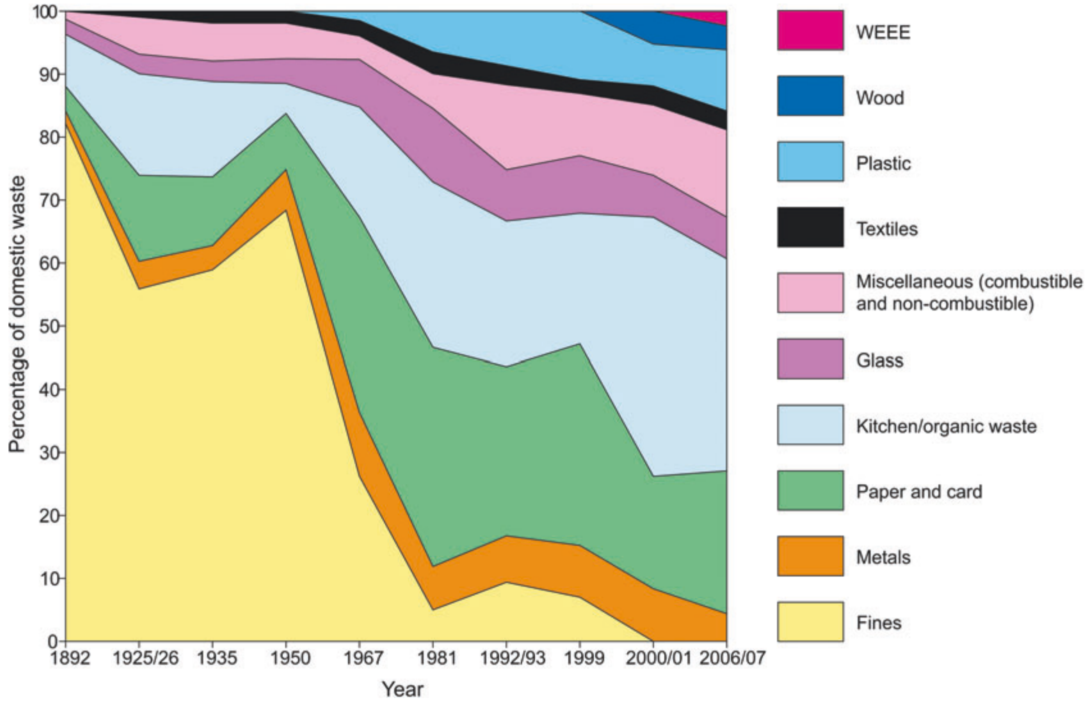
Municipal and commercial wastes: As towns and cities grew, refuse began to be concentrated at sites outside them, to minimize proximity to pollution and vermin. Through much of their history they have been uncontrolled tips of mixed inert to biodegradable and potentially toxic wastes. However, as populations have soared over recent decades, the increasing risk of contamination of the air and water led to tighter controls on waste management, with an emerging trend of sealing the landfill material to prevent seepage of leachate into groundwater. Landfilling is initially relatively cheap but is environmentally and socially expensive in the longer term.

Prior to the Industrial Revolution, in the UK, volumes of waste were small and dominated by ash, wood, bone, and vegetable and human wastes (Fig. 5), with a low proportion of metal due to its reuse and recycling. By the early twentieth century, most houses in the UK were fuelled by coal fires and significant volumes of ash went to landfill. However, following the implementation of the Clean Air Act in 1956, the use of coal fires in homes, and the proportion of ash and cinder in landfill waste, began to decline (Ford et al. 2014). In the 1960s, with increased production of plastics replacing traditional materials and with increased waste production, the volume and composition of wastes changed radically in the UK (Ford et al. 2014). Subsequent legislation in the UK, including the Control of Pollution Act in 1974, and increased reuse and recycling resulted in greater segregation of waste materials and corresponding changes in waste volume and composition, a change that has occurred throughout developed nations. These changes over time are important in evaluating past sites.

Landfill sites may contain a wide variety of components, including some, or all of the following: paper, cardboard, glass, plastics, metals, wood, kitchen waste, garden waste, textiles, paints, inks, adhesives and resins, solvents, detergents, batteries, construction debris, topsoil, etc. This list is not exhaustive. The nature of potential contaminants associated with these deposits is outlined later (see section "Hazards and Risk Management Associated with Areas of Artificial Ground").

Transportation routes: In roads, railways, tunnels, and most airports, the amount of materials moved to form the infrastructure (i.e., earthworks) are typically far greater than the built structures, such as the road surfaces, rail lines. In order to maintain even gradients, many transportation routes involve local shifting of materials, with excavation of cuttings and redeposition of the extracted geological materials as embankments or through construction of cut and cover tunnels, requiring limited costly importation or exportation of deposits (Douglas and Lawson 2001). Earth movement required for road foundations and new urban construction is an average of 0.75 m depth (Douglas and Lawson 2001).

Modern roads commonly have a surface of asphalt, concrete, or road metal ("tar macadam"/blacktop) about 5 cm thick, resting upon a compacted base of sand, gravel, or crushed rock aggregate (ballast). During the era of steam locomotives, railway lands, both depots/carriage works and track beds, included much timber, coal debris, oils, and solvents and so were prone to spontaneous combustion (Waters et al. 1996).



Artificial Ground, Fig. 5 The changing composition of household Municipal Solid Waste in Great Britain. *WEEE* waste electrical and electronic equipment (Sourced, unmodified, from Ford et al. 2014)

The rail tracks continue to be made of steel, now mainly resting on concrete sleepers rather than traditional wooden ones, resting in turn on a ballast base at least 0.3 m thick.

Road construction in hilly areas can initiate debris slides and flows, can cause erosion (particularly through gully initiation), or act as a focus for sediment accumulation (Tarolli and Sofia 2016). In flat landscapes, elevated road or rail embankments can significantly impact on natural flood patterns (Tarolli and Sofia 2016).

Mineral extraction: It is estimated globally that 57 gigatonnes (Douglas and Lawson 2001) of industrial minerals, including sand, gravel, clay, metal ore, coal, etc. and including spoil and tailings and waste were quarried or mined annually at the turn of the millennium. These minerals are mainly extracted by either underground mining or surface (opencast or strip) mining. Underground mining tends to selectively extract only the workable mineral and is typically associated with low volumes of waste materials, although colliery spoil is an extensive type of made ground, commonly forming elevated heaps in many coalfields of Europe and the eastern USA. Surface mining, which has become volumetrically more important over recent decades, can result in stripping of overburden to depths of tens of meters to access the resource and creates significantly more waste (Ford et al. 2014), though much of the overburden is commonly reinstated into the excavated void (Douglas and Lawson 2001) as infilled ground. This is particularly the case for large coal opencast sites. Large aggregate pits located along floodplains, for example, in the UK lack sufficient waste materials to allow reinstatement, and high water tables make them unsuitable as landfill locations, so commonly remain as flooded workings following completion, although there can be opportunities for restoration of margins to support some wildlife.

Subsidence of up to 50 mm/year and lowering of the water table can accompany mineral extraction, particularly by underground mining (Tarolli and Sofia 2016). Landslides, debris flows, and rockfalls are enhanced, too, in areas of extensive surface mining, both within the spoil debris and within adjacent natural deposits (Tarolli and Sofia 2016).

Industrial development: Many heavy industries have generated waste products both as artificial deposits, and also air and waterborne contamination. Iron works, foundries, and other metal smelters produce large quantities of slag, sand, ash, and spent refractory materials with high concentrations of heavy metals. Coal-burning power stations generate large quantities of pulverized fuel ash (pfa). Asbestos wastes generated either by the production of asbestos, or in wastes that contain those products, are a potentially significant hazard. Similarly radioactive wastes generated from power, military, medicinal, or scientific usage also produce highly hazardous wastes.

The chemical industry produces diverse acids, adhesives, cleaning agents, cosmetics, dyes, explosives, fertilizers, food additives, industrial gases, paints, pharmaceuticals, pesticides, petro-chemicals, plastics, textiles, and so on, each with distinct waste products. A significant recent development is of large underground storage facilities. These have been developed for liquefied petroleum gas and compressed air energy storage, whereas the subsurface is increasingly used as a source of geothermal energy, and to provide long-term burial of nuclear and other hazardous wastes (Evans et al. 2009; Zalasiewicz et al. 2014).

Water supplies and sewerage: In order to provide adequate water supplies and power for modern urban conurbations and to supply agricultural schemes, river systems have been converted into reservoirs through construction of dams and barrages. Urban centers include extensive utility pipelines supplying clean water and removing sewage. Many sewage works are located on gravel beds on floodplains, with such treatment works generating solid sewage sludge. In many agricultural areas, more than 80% of catchment areas can be drained by surface ditches or subsurface pipes (Tarolli and Sofia 2016).

Flood defenses and coastal reclamation schemes: Engineered structures are commonly constructed to provide coastal defenses to control erosion and to protect human habitations from tides and waves. These can take the form of seawalls and levees to protect from wave-induced erosion or high tides. Similarly, the construction of groynes, breakwaters, and artificial headlands inhibits lateral transport of sediments and maintains shingle and sand beaches, which in turn protect from storm waves and tidal surges.

Extensive areas of Artificial Ground result from land reclamation from lakes and seas. The generation of extensive agricultural land (e.g., polderization in the Netherlands) typically creates little artificial deposits. In contrast, reclamation increases living space, facilitates construction of industrial sites or to locate major airports (e.g., the Palm Jumeirah, Dubai, and Chek Lap Kok island, Hong Kong), and involves the transport and deposition of vast quantities of rock, sediment, and soil.

Warfare: Precautions against impacts of warfare can include construction of defensive structures. The results of warfare include building debris associated with bomb damage, which is often aggregated to form extensive and thick deposits (e.g., the Teufelsberg, Berlin). High explosives first initiated significant landscape modification during WWI, creating extensive cratering of the Western Front from persistent detonations, leaving little or no original soil surface remaining undisturbed and undetonated ordnance as a significant hazard (Zalasiewicz and Zalasiewicz 2015).

Characterization of Artificial Ground

Based on history/desk study: To provide planners and developers with an indication of the categories of Artificial Ground (described above) that may be present at a site and the broad makeup of human-made deposits, desk studies should be undertaken so that planners and developers are aware of the risks of difficult engineering ground conditions and the potential presence of toxic residues and explosive gases, and can ensure that site investigations are designed to assess these problems.

Current and former land use can often be determined through sequential study of detailed scales of topographical and geological maps and aerial photographs. These indicate areas of made, worked, and infilled ground as geomorphological features that represent a topographic modification to the preexisting land surface. The descriptions of certain land-use types (though not comprehensive) are commonly provided on topographic maps, or carried out as part of regional land-use studies. An understanding of the current or former land use may provide an indication of the broad nature of the associated deposits. In areas of mineral workings, mine plans may identify the location of potential shallow open workings and mine entrances, and opencast completion plans may show the extent and depth of workings prior to subsequent backfill (Waters et al. 1996).

The nature, composition, and thickness of made ground can be assessed from a study of historical borehole and trial pit data. However, such data are used with the caveat that the site investigation data would have been collected prior to the intended development of the site and would not necessarily reflect what is currently present. Potential surface elevation changes subsequent to development may be estimated through interpolation of start height data published on the historical site investigation record, compared with elevations from a modern digital terrain model (DTM). The DTM can also provide an accurate determination of the location and vertical scale of geomorphological features which can be attributed to specific land uses, such as railway embankment or quarry excavation.

Ground investigation techniques: Made ground is heterogeneous and can rapidly vary in composition, both laterally and vertically. In order to determine the nature of fill material, it is necessary for an appropriate site investigation to be carried out. The investigation of Artificial Ground is a specialist task and must be carried out in compliance with current best practice (e.g., BS 5930 2015; BS 10175 2011) and with regard to the safety of site personnel and the public.

Ground investigations should involve trial pits and boreholes. Trial pits are particularly useful as they allow large sections of the fill to be inspected. Standpipe piezometers sealed into boreholes should be employed to obtain information about water levels within the fill. The ground investigation should assess geotechnical aspects of Artificial Ground and should consider investigation of other relevant factors which could, in some situations, include chemical attack (on buried pipes and concrete), gas generation, combustibility, and toxicity. Investigation of contaminated land to detect and evaluate the concentrations of types and levels of pollution generally involves the following (Nirei et al. 2012):

(a) Drilling or trial pitting, often on a regularly spaced grid pattern, and evaluation of the extent of contamination on the basis of predictive models using probability or geostatistical methods; and (b) taking and analyzing samples at prescribed depths. However, it may be beneficial to consider such drilling and sampling programs to be designed to take proper account of the sequence of deposition of materials as well as their physical characteristics (Nirei et al. 2012), using one of the schemes outlined in "classification schemes." The identification of units within Artificial Ground can establish how pollutants may have entered, migrated through, and accumulated within these deposits (Nirei et al. 2012).

With variable fill, small-scale laboratory tests may be of limited use whereas a program of field tests can provide much important information. The most useful tests on deep fills may be simply to monitor settlement rates of the fill by precise leveling. Stable benchmarks need to be established away from the filled ground, so it is important that the boundaries of the filled area are established by borings and pitting.

Geophysical surveying may be used to aid the design of the borehole or trench site investigation and as a means of interpolating between, and extrapolating from, available borehole/trial pit data (but not to replace such direct investigation techniques) to map out the lateral extent and potentially thickness of artificial deposits.

Electrical resistivity/conductivity techniques, surveyed on a grid basis, have proved effective in locating anomalous zones associated with infilled voids or changes in groundwater composition. Electromagnetic (EM) data can show high electrical conductivity in the near subsurface relating to contaminant plumes associated with leakage from landfill sites, waste dumps, mine waste tips, and industrial pollution. It can also provide information on foundations, archaeological structures, and presence of certain artifacts. Urban demolition wastes may be distinguished from natural soils by elevated electrical conductivity values (>150 μS/cm) (Table 4), with differences to native soils attributed partly to the presence of artifacts, which are virtually ubiquitous in soils in urban areas (Howard 2014). Self (spontaneous)-potential (SP) methods are useful in recognizing groundwater flow-paths in artificial deposits, including seeps in earth dams.

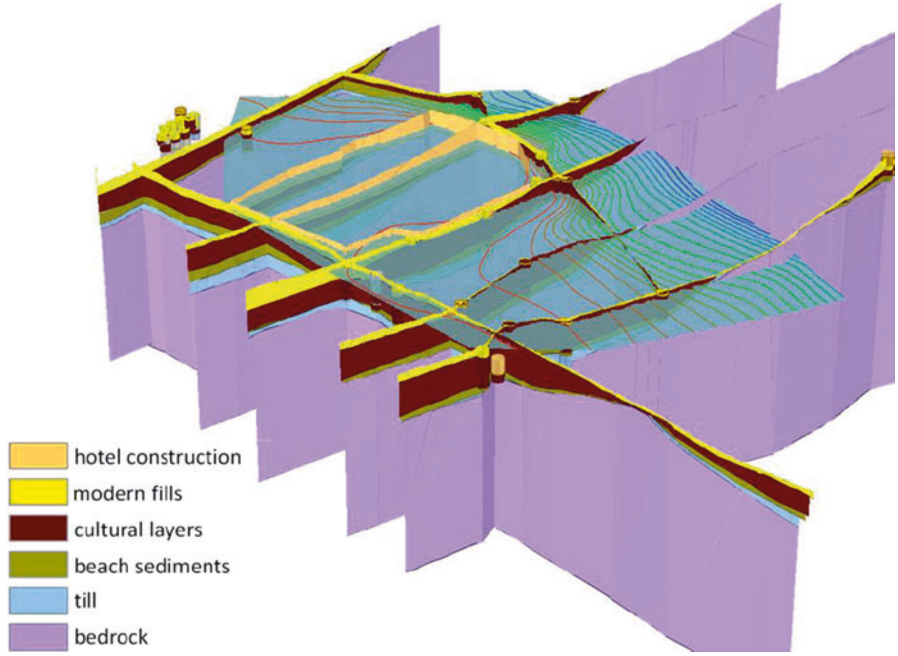
Gravity surveys can detect large near-surface cavities or infilled voids in which the fill has markedly different density to the surrounding deposits. Ground probing (penetrating) radar has particular use in recognizing archaeological structures and areas of disturbed ground and may be useful in association with magnetic (Er) surveys in detecting shaft and adit mine entrances and mine workings. Anthropogenic soils have elevated values of magnetic susceptibility relative to natural soils due to the presence of magnetite or maghemite-bearing artifacts such as brick and other fired ceramics, coal-related ash and cinders, and metallurgical, coking, and cement dusts (Howard 2014). A magnetic survey would also be able to identify ferrous metallic debris and recognize old utility cables. A seismic reflection study could be useful in identifying the depth to the base of thick artificial deposits, if used in conjunction with borehole drilling.

Engineering properties: Basic engineering properties such as particle size analysis, voids ratio and porosity, bulk density, dry density and relative density (RD), moisture content, and Atterberg limits on clay-rich fill can provide important information on the likely physical behavior of artificial deposits during any future development on, or reuse of, the fill. However, given the often marked, and potentially unpredictable, heterogeneity of such deposits, it may be of little use in providing representative site-wide data.

Simple field loading tests can be useful in some situations since they provide direct evidence of performance. Lightweight structures with strip footings generally stress the ground significantly only to depths of 1.5–2.5 m. Consequently, it is relatively simple to test load the fill to reproduce the actual stress level and distribution with depth. Such tests, however, reflect only the near-surface properties of the fill. In some fills (e.g., soil fills without any large hard fragments),

Artificial Ground, Table 4 Electrical conductivities of urban soils, natural soils, and parent materials in Detroit, USA (From Howard 2014)

Sample material	Artifacts	Electrical conductivity (μS/cm)		Number (*n*)
		Average	Range	
Natural superficial deposits				
Diamicton	None	787.2	626–948	2
Sand	None	38.3	19–77	4
Native (natural) soils				
Topsoil	None	123.8	70–157	6
Outer urban residential demolition sites				
Topsoil	Common	294.8	82–629	16
Inner urban residential demolition sites				
Topsoil	Abundant	320.2	72–1,034	27
Industrial demolition sites				
Topsoil	Abundant	305.4	284–327	2



Artificial Ground, Fig. 6 Three-dimensional ground model showing natural and anthropogenic deposit types along constructed cross sections with contoured elevation of groundwater (From De Beer et al. 2012). Boreholes that are used to determine the model are shown as vertical cylinders, colored to represent the associated type of deposit shown in the key

cone penetration tests (CPT's) can give an indication of the condition of the fill at depth.

Where it is intended to develop an area in the vicinity of a completed landfill site, they should be investigated by drilling with measurement of gas composition, the temperature gradients with depth, the direction and rate of groundwater flow, and the permeability of subsurface materials.

Chemical tests: The main basic tests of relevance to artificial deposits include pH value, sulfate content, organic content, and chloride content. Groundwater that is excessively acid or alkaline can have detrimental effects on buried concrete and metals, sulfate-rich waters can attack concrete and cementitious materials, and chloride-rich solutions can cause corrosion of iron and steel. High organic content can reduce bearing capacity and increase compressibility and swelling/shrinkage potential as well as contribute to groundwater acidification.

Ground models: Ground models can provide insights to urban planners, heritage authorities, and the general public into geological, archaeological, and other anthropogenic deposits and structures that are present above and below the ground surface. A good example of a detailed 3D ground model associated with complex Artificial Ground resulting from historical archaeological activities and modern development and urbanization is the World Heritage Site of Bryggen in Bergen, Norway, described by De Beer et al. (2012). The 3D framework model (Fig. 6), constructed from available borehole records, allows the compilation and presentation of complex modeling and monitoring data (including archaeological, geological, geochemical, geophysical, geotechnical, and hydrological) from different sources. The model can be presented as correlated and synthetic cross sections, synthetic horizontal slices, spatial distribution "envelopes" of anthropogenic units, and contoured surfaces as well as 3D visualizations (De Beer et al. 2012).

Hazards and Risk Management Associated with Areas of Artificial Ground

Unless the dimensions and material nature are known to a high degree of certainty, all sites of artificial deposits should be considered as suspect because of the likelihood of extreme variability in the composition and compaction of the fill materials (some of which may be hazardous to health or harmful to the environment or building).

Artificial deposits generated by the disposal of domestic and industrial waste in licensed and unlicensed tips are potentially the most problematic to construction and land use (Waters et al. 1996). The material may include a very wide range of inorganic, organic, inert, reactive, combustible, harmless, and toxic substances and may have no record of what has been deposited. These materials may have been tipped without regard to compaction, containment, or their potential to interact with each other and the environment. It is difficult to predict the engineering ground conditions of such sites as they will be highly variable across their area and with depth.

Not all artificial deposits will present problems, though many can represent a potential hazard in three main ways (Waters et al. 1996):

Unstable foundation conditions: Modern, properly engineered fills, such as highway embankments, controlled opencast infill, or deposits rich in inert building stone or waste deposits associated with extraction of building stone, may have excellent engineering properties. However, the often uncompacted or poorly compacted nature of anthropogenic deposits can give rise to unstable foundation conditions. In places the deposits may be very weak or cavernous and cause excessive and uneven settlement. Organic material within the artificial deposits may rot, causing cavitation and settlement below surface structures. When spoil is dumped on a slope, the buried soil/organic layer may be weak and therefore might form a potential failure surface. Poorly managed groundwater flow can produce catastrophic failure of poorly compacted embankments and spoil heaps.

Geotechnical properties of domestic and industrial waste will vary with time as material decays, and the nature of old domestic and/or industrial waste sites cannot necessarily be used as a guide to the future behavior of recent sites because of the changing nature of the waste materials through recent history (Waters et al. 1996). Modern waste management practices should result in much better ground conditions than in the past and, following thorough site investigation, the use of suitable foundations and sealing and/or ventilation of basements will enable sites to be satisfactory for many end uses.

A number of key hazards identified by Waters et al. (1996) include: (1) Made ground may rest on soft alluvial clays or peats which may, themselves, undergo excessive differential consolidation settlement when loaded in addition to that resulting from the compression of the fill itself. Therefore, the stability of the ground underlying the fill may need to be examined; (2) if structures are built on piles passing through the fill into underlying strata, negative skin friction caused by the fill settling under its own weight may be a major consideration in foundation design; (3) where the fill is deep, self-weight may cause long-term settlement. With granular fills and poorly compacted unsaturated fills of all types, the major compression occurs almost immediately and consequently most of the settlement due to self-weight occurs as the fill is placed. Nevertheless, significant further movements ("creep" settlement) do occur under conditions of constant effective stress and moisture content. With many fills, the rate of creep compression decreases fairly rapidly with time; (4) compression of fills by building loads will be very variable depending on the nature of the fill, its particle size distribution, compactness, the existing stress level, the stress increment, and the moisture content. Assuming the stress increments due to building loads do not bring the fill close to bearing capacity failure, settlements can be most simply calculated using the constrained modulus, defined as $\Delta\sigma v/\Delta\varepsilon v$, where $\Delta\sigma v$ is an increment of vertical stress, and $\Delta\varepsilon v$ is the increase in vertical strain produced by $\Delta\sigma v$. Movements which occur during building construction are likely to be much less of a problem than those which occur after completion of the structure, so the long-term creep component is therefore of particular significance; (5) loose unsaturated fill materials are usually liable to collapse settlement following inundation with water. If this occurs after construction, a serious settlement problem may arise. This may be a major cause of settlement problems in building development on restored opencast mining sites. Problems can also be caused by water penetrating into the fill from the surface through deep trench excavations for drains associated with building development; (6) when fine material is placed underwater, as in sludge lagoons, a soft cohesive fill is formed which is characterized by low permeability. Settlement is controlled by a consolidation process in which excess porewater pressures dissipate slowly as water is squeezed out of the voids in the fill. This type of fill may be susceptible to liquefaction, and often a firm but thin crust, overlying very soft material, may form over the surface of the lagoon deposit; (7) excessive differential settlements, leading to distortion and damage to buildings, are to be expected in highly variable poorer types of fill, or where the depth of fill changes rapidly, such as where the deposits backfill a pit or quarry and this may be compounded in cases where cavitation results from chemical or bacterial breakdown of the fill material (e.g., bricks or concrete may degrade in the presence of water, temperature fluctuations and sulfate or chloride-rich groundwaters, whereas iron-based metals corrode in the presence of oxygen and chloride ions); (8) at sites in areas of demolished industrial buildings and housing, load-bearing walls may be present at, or close to, the surface of the fill. New foundations built across such walls and the surrounding fill are liable to severe differential settlements. Such sites may also contain basements, cellars, tunnels, and service ducts which may be only partially filled with rubble and often remain as complete voids.

Contamination: The delineation of contaminated land is difficult, partly because there is little consensus what constitutes contaminated land and also the concern that identification of such areas may affect property values. Toxic residues, either as a primary component of the human-made deposit or generated secondarily by chemical or biological reactions within the deposits, can migrate both through the deposit and into adjacent permeable strata.

Contaminants, most notably from landfill sites but also from materials spread on industrial sites, may include a wide range of heavy metals, sulfates, sulfides, acids, alkalis, hydrocarbons, general organics, phenols, dioxins, and polychlorinated biphenyls (PCBs). Contamination from metalliferous mining and industrial sites, including chemical works, could be of concern. Wastes associated with the iron and steel industry may contain high levels of heavy metals, such as chromium, lead, nickel and zinc, sulfates and sulfides, acids and alkalis, asbestos, solvents, phenols, and hydrocarbons. Land used in old gasworks for coal carbonization, purification, tar storage, dumping of "spent oxide," coke storage, tar refining, and asphalt production can be associated with high concentrations of sulfates, phenols, coal tars and other aromatic hydrocarbons, oils, free and complexed cyanides, sulfur, and sulfides. Heavy metals may include chromium, copper, lead, nickel, and zinc, and other hazardous substances which may be found at former gasworks include asbestos, benzene, toluene, and xylene. In railway lands associated with depots and engineering works, contaminants include sulfates, heavy metals, oils, solvents and paints, asbestos, and PCBs, and the fill may be susceptible to spontaneous combustion due to the common presence of timbers and coal debris.

Pyrite (iron sulfide) present in colliery spoil is prone to oxidize and produce sulfate-rich, acidic groundwater leachates (acid mine drainage) causing corrosion problems for concrete present in foundations or buried services. The leachates may also affect the quality of surface water. Sewage works may contain sludges and solid wastes with high concentrations of heavy metals, filtered from the sewage, including cadmium, copper, lead, nickel, and zinc, and acids used in water treatment.

Note that contamination can arise in areas where little, or no, artificial deposits are present. Also, the identification of a previous, potentially contaminative land-use at a site, does not, necessarily, imply that the area is contaminated. Furthermore, even if the former land use did lead to contamination, it is possible that measures were taken to decontaminate the site, but records of these may be elusive or lost.

Gases – radon, carbon dioxide, and methane: Toxic or explosive gases, particularly methane and carbon dioxide, can be generated within waste tips, landfill sites, and in disused mine workings. Radon is produced from rock types including certain granites, uraniferous mineral deposits, uraniferous black shales, and phosphatic sedimentary rocks. All such gases can sometimes migrate through adjacent permeable strata and accumulate within buildings or excavations, either nearby or some distance away.

The accumulation of methane in closed spaces in buildings, sub-floor spaces, or basements may reach explosive concentrations (5–15% in air). Colliery spoil heaps contain coal and carbonaceous material which, if present in high enough proportions (in excess of 20% coal by weight), may be combustible. Filter beds and settling tanks may contain high levels of organic matter. When disused and buried, these organic deposits may slowly decompose under saturated conditions to produce nitrogen-rich leachate, methane, carbon dioxide, and hydrogen sulfide. Foundry sand often contains organic materials capable of generating methane (Hooker and Bannon 1993).

Artificial Ground as a Resource

Reworking wastes: Mine waste represents the materials that were considered worthless at the time of production, but which may contain mineral and energy resources that sometimes subsequently become valuable. Following the definitions of Lottermoser (2011): *Reuse* of mine wastes and mine waters involves the new use or application of the waste in its original form without any reprocessing. *Recycling* of mine wastes involves extraction of new resources, or converts the waste into a new product or application with some physical, thermal, biological, or chemical reprocessing, rendering the remaining waste material suitable for a new benign use or disposal. *Reprocessing* uses mine waste to produce a product, such as recovered minerals and metals. *Treatment* of mine waste is intended to reduce the waste's toxicity or volume. *Rehabilitation* and *reclamation* of mine wastes refer to measures that alleviate environmental impacts during post-mining waste storage.

Various options have been proposed for mine wastes reworking as a resource (Table 5), though only a few of these options are commonly employed (Lottermoser 2011). Mineral wastes also have potential uses as a source of engineered fill, with benign waste materials for landscaping, to fill open voids (especially sandy tailings mixed with cement to produce a grout), as well as acting as a hydrogeological barrier capping and allowing revegetation of waste repositories. Coarse-grained mining wastes, especially barren waste rocks from coal and metal mining, are as building and construction materials used as bulk fill for land affected by subsidence or as aggregates in embankment, dam, road, pavement, foundation, and building construction (Lottermoser 2011). Reddened and oxidized burnt shale present in colliery spoil may also be suitable for use in road construction. Pulverized fuel ash can be used as a reactant in concrete and breeze block production. Slags produced during metalliferous smelting may be suitable for the production of concrete and cement, as fill, ballast, abrasive, and road aggregate. Such blast-furnace slags, along with mineral extraction wastes, may contain metals which are now economically worth recovering. Oil shale waste may yield liquid hydrocarbons on distillation.

Voids for waste disposal: Areas of disused quarries or pits or disused railway cuttings, particularly if located within impermeable bedrock or superficial deposits, may form a valuable repository for waste disposal.

Archaeological/geological sites: Some areas of Artificial Ground may be associated with ancient monuments or include deposits of archaeological or geological significance. For example, mine waste tips may contain scientifically interesting minerals. Disused quarries can be restored to allow

Artificial Ground, Table 5 Options for the reworking of mine, processing, and metallurgical wastes (From Lottermoser 2011). © 2011 by the Mineralogical Society of America

Waste type		Reuse and recycling option
Mining wastes	Waste rocks	Resource of minerals and metals
		Backfill for open voids
		Landscaping material
		Capping material for waste repositories
		Substrate for revegetation at mine sites
		Aggregate in embankment, road, pavement, foundation, and building construction
		Asphalt component
		Feedstock for cement and concrete
		Sulfidic waste rock as soil additive to neutralize infertile alkaline agricultural soils
	Mine waters	Dust suppression and mineral processing applications
		Recovery of metals from acidic mine drainage (AMD) waters
		Drinking water
		Industrial and agricultural use
		Coolant or heating agent
		Generation of electricity using fuel cell technology
		Engineered solar ponds for electricity generation, heating, or desalination and distillation of water
	Mine drainage sludges	Extraction of hydrous ferric oxides for paint pigments
		Extraction of Mn for pottery glaze
		Flocculant/adsorbant to remove phosphate from sewage and agricultural effluents
Processing wastes	Tailings	Reprocessing to extract minerals and metals
		Waste reduction through targeted extraction of valuable minerals during processing
		Sand-rich tailings mixed with cement used as backfill in underground mines
		Clay-rich tailings as an amendment to sandy soils and for the manufacturing of bricks, cement, floor tiles, sanitary ware, and porcelains
		Mn-rich tailings used in agro-forestry, building and construction materials, coatings, cast resin products, glass, ceramics, and glazes
		Bauxite tailings as sources of alum
		Cu-rich tailings as extenders for paints
		Fe-rich tailings mixed with fly ash and sewage sludge as lightweight ceramics
		Energy recovery from compost–coal tailings mixtures
		Phlogopite-rich tailings for sewage treatment
		Phosphate-rich tailings for the extraction of phosphoric acid
		Ultramafic tailings for the production of glass and rock wool
		Carbon dioxide sequestration in ultramafic tailings and waste rocks
Metallurgical wastes	Bauxite red mud	Treatment of agricultural and industrial effluents
		Raw material for glass, tiles, cements, ceramics, aggregate, and bricks
		Treatment of AMD waters
		Carbon dioxide sequestration
	Historical base metal smelting slags	Production of concrete and cement
		Use as fill, ballast, abrasive, and aggregate
		Extraction of metals (e.g., Cu, Pb, Zn, Ag, Au)
	Phospho-gypsum	Soil amendment
		Building and construction material
		Extraction of elements and compounds (e.g., U, Y, REE, and calcium sulfate)

continued access as a geological site of interest for research or educational purposes.

Recreational sites: Large aggregate pits located along floodplains, flooded following completion, provide potential amenities for wildlife and recreation. Similarly, old quarries may be left or restored as wildlife havens.

Engineering Design and Construction

Ground improvement techniques to enhance the load-carrying characteristics of artificial deposits prior to development may be a viable solution. This typically involves increasing the density of the fill, although other methods such as grouting may be applicable in some situations. Treatment techniques may include excavation and refilling in thin layers with controlled compaction, preloading with a surcharge, dynamic consolidation, and, for granular soil fill, vibro-compaction. Where the fill is deep, costs of all methods will be closely related to the depth to which the fill is to be treated. Any ground improvement technique considered for use on a filled site should be examined for its cost effectiveness, and in the light of other problems existing at the site, as there may be situations in which a particular improvement method may be beneficial in more than one respect. For example, compaction of colliery spoil will not only improve its load carrying characteristics but may also reduce or eliminate the risk of combustion. In contrast, where gases are being generated during organic decay, ground treatment solutions involving vibro-techniques could lead to the formation of paths through which methane gas could enter building foundations.

For relatively large structures built on shallow artificial deposits, poor load carrying characteristics can be circumvented by using piled foundations and a suspended floor. The piles should be designed for negative skin friction caused by settlement of the fill. Particular care is needed in fills in which methane gas is being generated as piles could form pathways for the escape of the gas into foundations. In such cases, venting should be carried out prior to development. Where small structures have to be built on deep fill, piling to a firm sub-stratum is not likely to be an economic solution. Reinforced concrete rafts with edge beams have been used, but very substantial foundations may be required where large differential movements are possible. In the foundation design, it is also important to distinguish between settlement due to the weight of the building and that due to other causes such as self-weight of the fill. With small structures on deep fills, the latter will almost invariably predominate and so "bearing capacity" can be a misleading concept. Foundation design should therefore be based on an assessment of the magnitude of movements of the fill subsequent to construction on it. Problems associated with low-rise buildings on filled ground can be reduced by avoiding building across the edges of filled areas, where the structure would be founded partly on fill and partly on undisturbed ground. Construction should be restricted to small units and long terraces of housing should not be built on existing filled ground. The relative movements between the building and the services entering it also merit careful consideration, for example, the use of short pipe lengths with flexible connections.

Migration of landfill, or other, gases may be limited by lining of the site with an impervious membrane and introduction of courses of inert, permeable, granular fill, and vent stacks to prevent excessive build-up of methane from the waste material. Retrospective sealing of old landfill sites may be carried out by grouting fissures present in the adjacent bedrock with bentonite/filler grout.

Summary

The importance of humanly modified Artificial Ground as both a novel geological landform and sedimentary deposit and as a geoengineering unit has only been realized in recent decades. This realization coincides with a dramatic increase in the rate of generation of Artificial Ground across the Earth's surface, and increasingly so in the subsurface. Despite the inherent difficulties in analysis of this typically complex environment, its importance as the foundation strata of many/most surficial developments (especially in urban areas) has resulted in the development of improved methodologies for the categorization and mapping/modeling of Artificial Ground and its characterization through engineering investigations. Artificial Ground can be associated with risks related to unstable foundation conditions, contamination and the presence of toxic or explosive gases, but is also increasingly being seen as a potential resource. Management of risks through ground improvement techniques and due consideration being given during the design and construction stages of development can in many cases cost-effectively mitigate the recognized hazards.

Cross-References

▶ Aggregate
▶ Borehole Investigations
▶ Brownfield Sites
▶ Classification of Soils
▶ Cohesive Soils
▶ Concrete
▶ Contamination
▶ Cut and Cover
▶ Cut and Fill
▶ Dams
▶ Drilling Hazards
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▶ Mining
▶ Nearshore Structures
▶ Tailings
▶ Waste Management

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Atterberg Limits

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Synonyms

Liquid limit; Plastic limit; Plasticity characteristics; Plasticity index

Definition

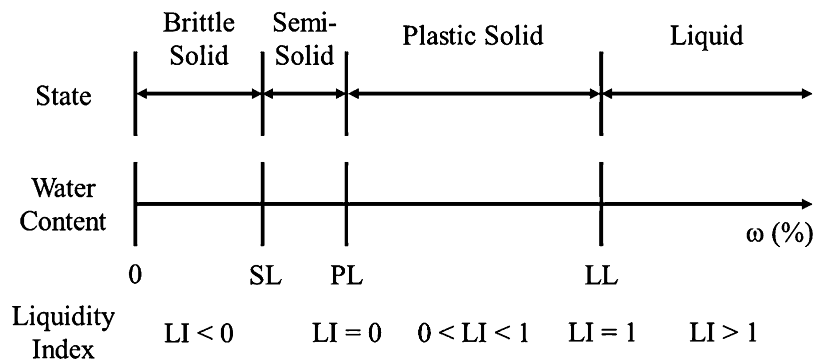
Atterberg limits are water contents at which marked changes occur in the engineering behavior of fine-grained soils. Fine-grained soils, consisting of particles smaller than 0.074 mm (#200 sieve), include silt and clay. Water content is the ratio of the weight of water to the weight of solids in a soil mass, expressed as a percentage.

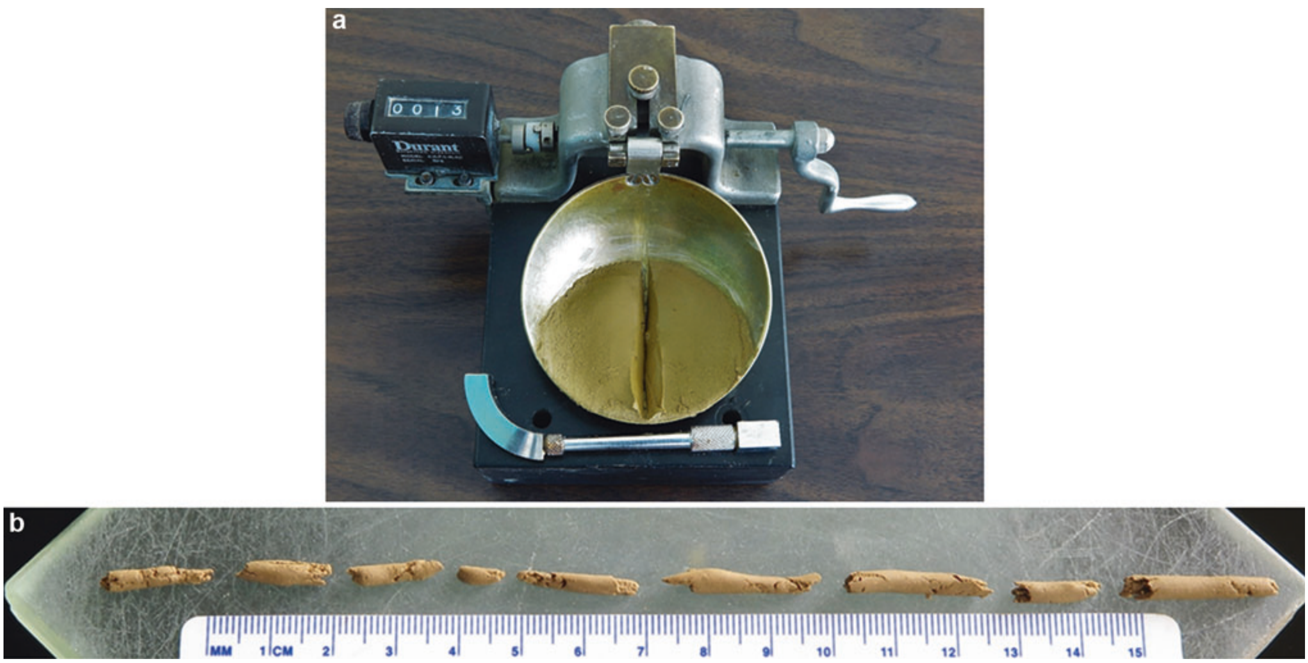
Introduction

Atterberg limits were developed by Albert Atterberg, a Swedish soil scientist (1911). Based on the behavior of fine-grained soils with changing water content, Atterberg defined seven limits (Holtz et al. 2011). Casagrande (1932) standardized Atterberg limits for engineering classification of fine-grained soils. The Atterberg limits used in engineering practice include liquid limit (LL), plastic limit (PL), and, less frequently, shrinkage limit (SL). Liquid limit is the lowest water content at which a soil-water mixture behaves as a viscous liquid, plastic limit is the lowest water content at which a soil-water mixture behaves as a plastic material, and shrinkage limit is the lowest water content beyond which no further change in volume occurs as the soil-water mixture dries. Plasticity index (PI) is the numerical difference between liquid limit and plastic limit. PI indicates the range of water contents over which a soil behaves as a plastic material. On a continuum of soil-water mixture (Fig. 1), as the water content increases, the soil behavior changes from a brittle solid to a semi-solid, to a plastic solid, to a viscous liquid, and finally to a true liquid (Holtz et al. 2011). Although Atterberg limits are water contents marking the boundaries between varying engineering behaviors of fine-grained soils, Atterberg limits, by convention, are reported without the percentage sign (Casagrande 1948) (Fig. 1).

Atterberg limits are very important index properties of fine-grained soils. They are used for classification of fine-grained soils (Casagrande 1948) and have been correlated

Atterberg Limits, Fig. 1 Changes in the engineering behavior of fine-grained soils with increasing or decreasing water content





Atterberg Limits, Fig. 2 (**a**) Liquid limit test apparatus showing the standard groove closed for 13 mm length; (**b**) plastic limit test showing the soil thread breaking into small segments at a water content equal to the plastic limit

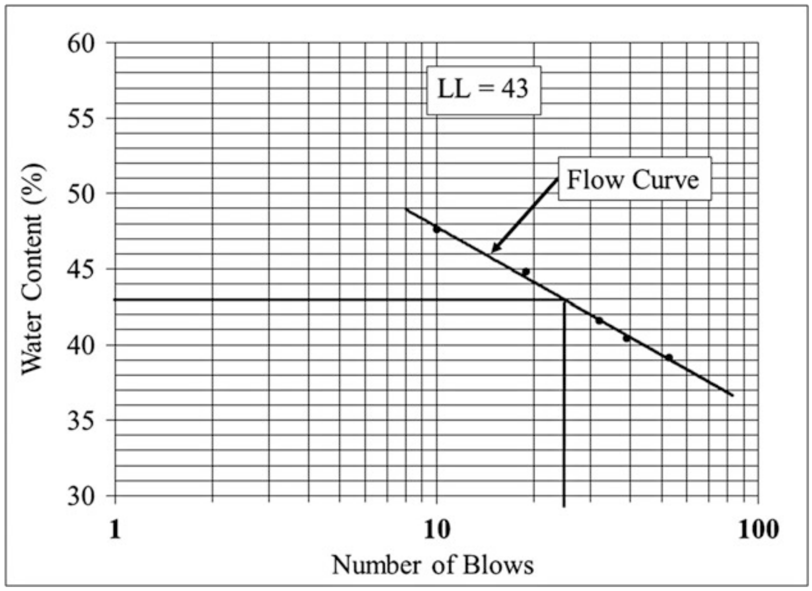
empirically with many other engineering properties of soils such as clay mineralogy (Mitchell and Soga 2005), shrink-swell behavior (Gibbs 1969; Mitchell and Gardner 1975; Martin-Nieto 2007), compression index (Terzaghi and Peck 1967), and shear strength parameters (Holtz et al. 2011). Both Atterberg limits and other engineering properties of fine-grained soils are strongly influenced by the amount and types of clay minerals present in a soil.

Higher values of LL and PI indicate that the soil has: (i) a high percentage of clay and active clay minerals (clay minerals that are sensitive to moisture changes), (ii) has a high resiliency, making it difficult to compact, (iii) has a low load-carrying (bearing) capacity, and (iv) is more susceptible to volume changes upon moisture fluctuations, making it an undesirable foundation material.

Determining Atterberg Limits

Liquid Limit

In order to standardize the test procedure for Atterberg limits, Casagrande (1932) defined liquid limit as the water content at which a groove cut in a soil pat, by a standard grooving tool, will require 25 blows to close for 13 mm when the LL-apparatus cup drops 10 mm on a hard rubber base (Fig. 2). The standardized test requires testing five to six samples so that approximately half require fewer than 25 blows to close the groove for 13 mm and half need more than 25 blows and plotting water contents (determined by oven-drying the tested samples for 24 h at 105 °C) versus logarithm of the corresponding number of blows (Fig. 3). Where the resulting curve, known as the flow curve, crosses



Atterberg Limits, Fig. 3 Plot of liquid limit test results. The liquid limit corresponds to the water content where the vertical line, representing 25 blows, intersects the flow curve

25 blows, the corresponding water content defines the liquid limit. Details of liquid limit apparatus, grooving tool specifications, sample preparation, and test procedure can be found in American Society for Testing and Materials (ASTM) method D 4318 (ASTM 2010). The liquid limit values can range from zero to 1000, with most soils having LL values less than 100 (Holtz et al. 2011) (Figs. 2 and 3).

Plastic Limit

Plastic limit is the water content at which a thread of soil, rolled gently on a frosted glass plate to 3 mm diameter, crumbles into segments 3 mm–10 mm long (Fig. 2). If the thread can be rolled to a diameter smaller than 3 mm, the soil water content is more than the PL and it should be balled up and rolled again. If the thread starts crumbling before it is 3 mm in diameter, the soil is drier than the PL and the procedure should be repeated after adding more water to it. Since the PL test is somewhat arbitrary, at least three trials are performed and the average value is reported. ASTM method D 4318 (ASTM 2010) provides details of the test procedure for the PL test. The PL can range from zero to 100, with most soils having values less than 40 (Holtz et al. 2011).

Both the liquid limit and plastic limit tests are performed on material passing # 40 sieve (<0.425 mm).

Plasticity Index

Plasticity index (PI) is the numerical difference between LL and PL. It is one of the most important index properties of fine-grained soils.

Liquidity Index

Liquidity index (LI) compares the natural water content of a fine-grained soil with its Atterberg limits and helps predict if the natural soil will behave as a brittle solid, plastic material, or viscous liquid when disturbed. Liquidity index is defined as:

$$LI = (w_n - PL)/PI \tag{1}$$

where w_n is the natural water content of the soil. If LI is greater than 1, the soil will behave as a viscous liquid when disturbed; if LI is between 0 and 1, the soil will behave plastically, that is, deform without fracturing under the application of stresses; and if LI is less than zero (has a negative value), the soil will behave as a brittle material when stressed (Holtz et al. 2011).

Summary

Atterberg limits are water contents at which significant changes occur in the engineering behavior of silt and clay. Important Atterberg limits include liquid limit, plastic limit, and plasticity index. Atterberg limits are used for classification of fine-grained soils and show significant correlations with many other properties of such soils. A comparison of Atterberg limits of a fine-grained soil with its natural water content, referred to as the liquidity index, indicates whether the soil would behave as a brittle material, as a plastic material, or as viscous liquid, when sheared.

Cross-References

▶ Clay
▶ Cohesive Soils
▶ Expansive Soils
▶ Liquid Limit

► Silt
► Soil Properties

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Avalanche

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Definition

Rapid flow of snow (snow avalanche), debris (debris avalanche), or rock (rockslide avalanche) downslope when the force of the material exceeds its strength. Avalanches accelerate rapidly and increase in their mass and volume under the influence of gravity.

Context

Snow avalanches occur spontaneously when the snow load increases beyond its strength, coupled with a trigger such as melting, rain, earthquake, landslide, and more commonly in recent years human activity including skiers, hikers, snowmobiles, etc.

Snow avalanches pose a considerable threat to recreation, property, resources, energy/communication corridors, and human lives across the world in all sloped and cold climate environments that are prone to snow accumulation and fluctuating temperatures. Globally, an estimated 250 individuals die in avalanches per year, including about 14 deaths per year in Canada, 28 deaths per year in the USA and 103 deaths per year in the European Alps. The number of fatalities has recently risen with the growing expansion of voluntary recreational activities (snowmobiles, cross-country skiing, etc.) and involuntary activities (transportation and settlements) into previously inaccessible areas (Statham et al. 2018).

One of the most catastrophic avalanche-related events in history occurred on the slope of Mt. Huascaran, Peru. On the 31 May 1970, a M7.9 earthquake near Ancash, Peru, triggered the release of an estimated 80 million m^3 of snow on the slopes of Mt. Huascaran which reached speeds of 335 km/h as it absorbed rock and debris before burying the towns of Yungay and Ranrahirca some 18 km away killing more than 20,000 individuals.

Avalanche types include slab, powder snow, and wet snow avalanches. Snowpack structure differences contribute to variations in avalanche patterns such as dry loose, storm slab, wind slab, and persistent slab. Issues to consider in the understanding and management of avalanches include terrain type, likelihood of avalanche(s) (function of sensitivity to triggers and spatial distribution), and destructive avalanche size. Table 1 provides an example of avalanche size classification.

Avalanche, Table 1 Canadian snow avalanche size classes by destructive potential (after McClung and Schaerer 2006)

Size	Destructive potential	Typical mass (t)	Typical path length (m)	Typical impact pressure (kPa)
1	Relatively harmless to people	<10	10	1
2	Could bury, injure, or kill a person	10^2	100	10
3	Could bury a car, destroy a small building, or break a few trees	10^3	1000	100
4	Could destroy a railway car, large truck, several buildings, or a forest with an area up 4 ha	10^4	2000	500
5	Largest snow avalanches known; could destroy a village or a forest of 40 ha	10^5	3000	1000

Avalanche monitoring involves direct quantitative measurements of wind speed, temperature, and new snow accumulation in starting zones as well as remote radar and satellite monitoring of larger areas.

One of the biggest issues currently facing engineering geologists relates to the avoidance of avalanche damage and impacts to infrastructure and communities in mountainous, snow covered regions. Road, rail, and telecommunication networks and new communities are designed and sited to reduce the risks associated with destructive avalanches. If full avoidance is not possible, other mitigation techniques include temporary (soft) measures (forecasting, road closure) or permanent (hard) approaches such as tunnels, reforestation, snow bridges, diversion walls, and snow sheds.

Cross-References

► Climate Change
► Earthquake Intensity
► Earthquake Magnitude
► Engineering Geomorphology
► Ground Shaking
► Hazard
► Hazard Assessment
► Instrumentation
► Land Use
► Mass Movement
► Mountain Environments
► Risk Assessment
► Risk Mapping
► Stabilization
► Tunnels

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B

Barton-Bandis Criterion

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Definition

A series of rock-joint behavior routines which, briefly stated, allow the *shear strength and normal stiffness* of rock joints to be estimated, graphed, and numerically modelled, for instance, in the computer code UDEC-BB. Coupled behavior with deformation and changes in conductivity is also included (Barton 2016).

A key aspect of the criterion is the quantitative characterization of the joint, joints, or joint sets in question, in order to provide three simple items of input data. These concern the joint-surface roughness (*JRC: joint roughness coefficient*), the joint-wall compressive strength (*JCS: joint compressive strength*), and an empirically derived estimate of the *residual friction angle* (φ_r). These three parameters have typical ranges of values from: JRC = 0 to 20 (smooth-planar to very rough-undulating), JCS = 10 to 200 MPa (weak-weathered to strong, unweathered) and $\varphi_r = 20°$ to $35°$ (strongly weathered to fresh-unweathered). Each of these parameters can be obtained from simple, inexpensive index tests or can be estimated by those with experience.

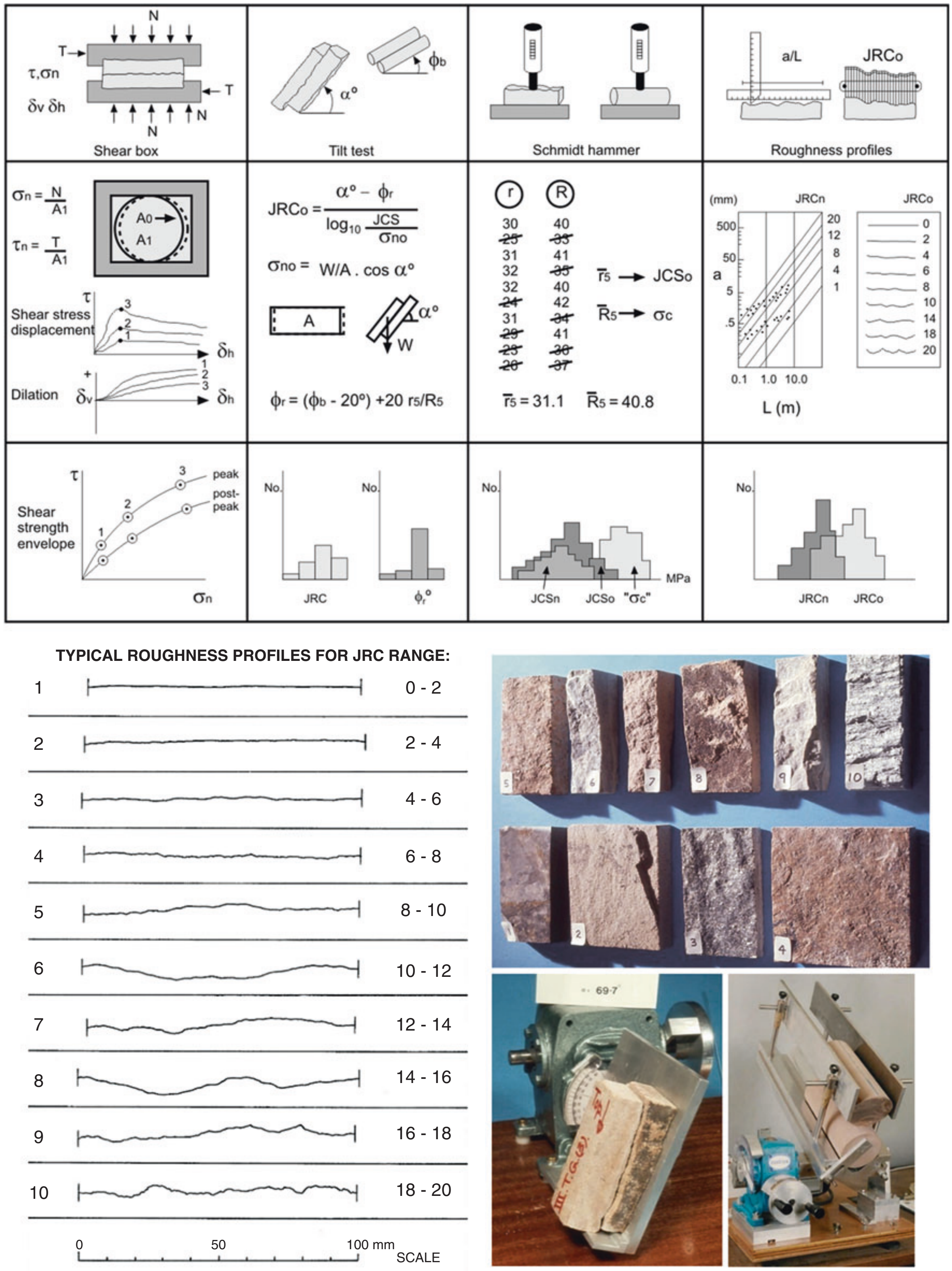
The three parameters JRC, JCS, and φ_r form the basis of the nonlinear peak shear-strength equation of Barton (1973) and Barton and Choubey (1977). This is a *curved shear strength envelope* without cohesion (c). It will be contrasted to the linear Mohr-Coulomb "c and φ" (with apparent cohesion) criterion later. To be strictly correct the original Barton equation utilized the basic friction angle φ_b of flat, unweathered rock surfaces (in 1973), whereas φ_r was substituted for φ_b following 130 direct shear tests on fresh and partly weathered rock joints (in 1977).

As well as peak and residual shear strength envelopes for laboratory-scale joint samples, Barton's cooperation with Bandis (from 1978) resulted in corrections (reductions) of JRC and JCS to allow for the *scale effect* and reduced strength as rock-block size is increased (Barton and Bandis 1982). The laboratory-scale parameters, written as JRC_0 and JCS_0 for laboratory-size samples of length L_0 (typically 50–250 mm), are written as JRC_n and JCS_n for *in situ* rock block lengths of L_n (typically 250–2500 mm, or even larger in massive rock).

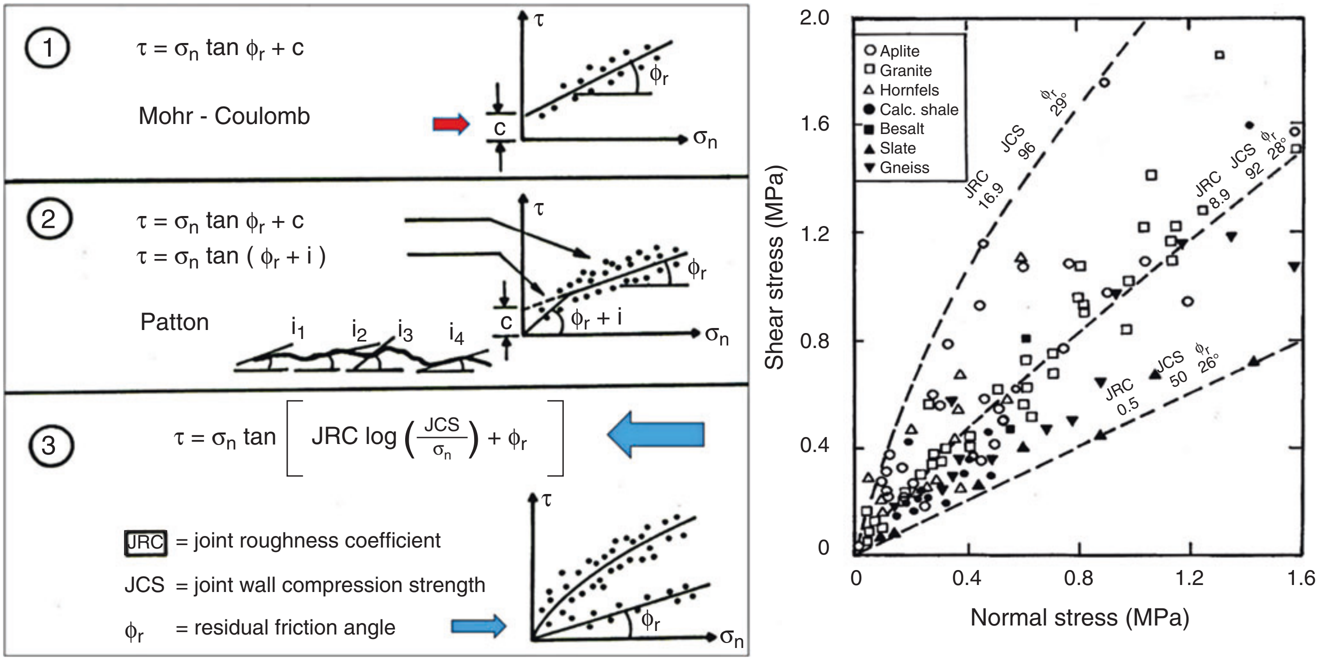
Bandis is also responsible for utilizing JRC and JCS in empirical equations to describe *normal closure and normal stiffness*. Normal stiffness (Kn) has units of MPa/mm and might range from 20 to 200 MPa/mm. The Barton-Bandis (B-B) criterion includes the related modelling of *physical joint aperture E* (typically varying from 1 mm down to 50 μm, or 0.05 mm) as a result of the normal loading (or unloading). B-B also includes the theoretically equivalent smooth-wall *hydraulic aperture e* (typically 1 mm down to 5 μm, or 0.005 mm). Usually $E > e$, and the two are empirically inter-related, using the small-scale joint roughness JRC_0.

Finally the stiffness in the direction of shearing has also to be addressed. It is called *peak shear stiffness (Ks)*. It has typical values of 0.1 MPa to 10 MPa/mm, that is, 1/10th to 1/100th of normal stiffness. The concept of *mobilized roughness* ($JRC_{mobilized}$) developed by Barton (1982) allows both the peak shear-stiffness and the *peak dilation angle* (giving an effective aperture increase with shearing) to be calculated. The full suite of Barton-Bandis joint behavior figures includes *shear stress-displacement-dilation, stress-closure, and the change of estimated conductivity* in each case. Examples of these are given in the following diagrams illustrating joint index testing (Figs. 1, 2, 3, 4, 5, 6, 7, and 8) (Barton and Bandis 2017).

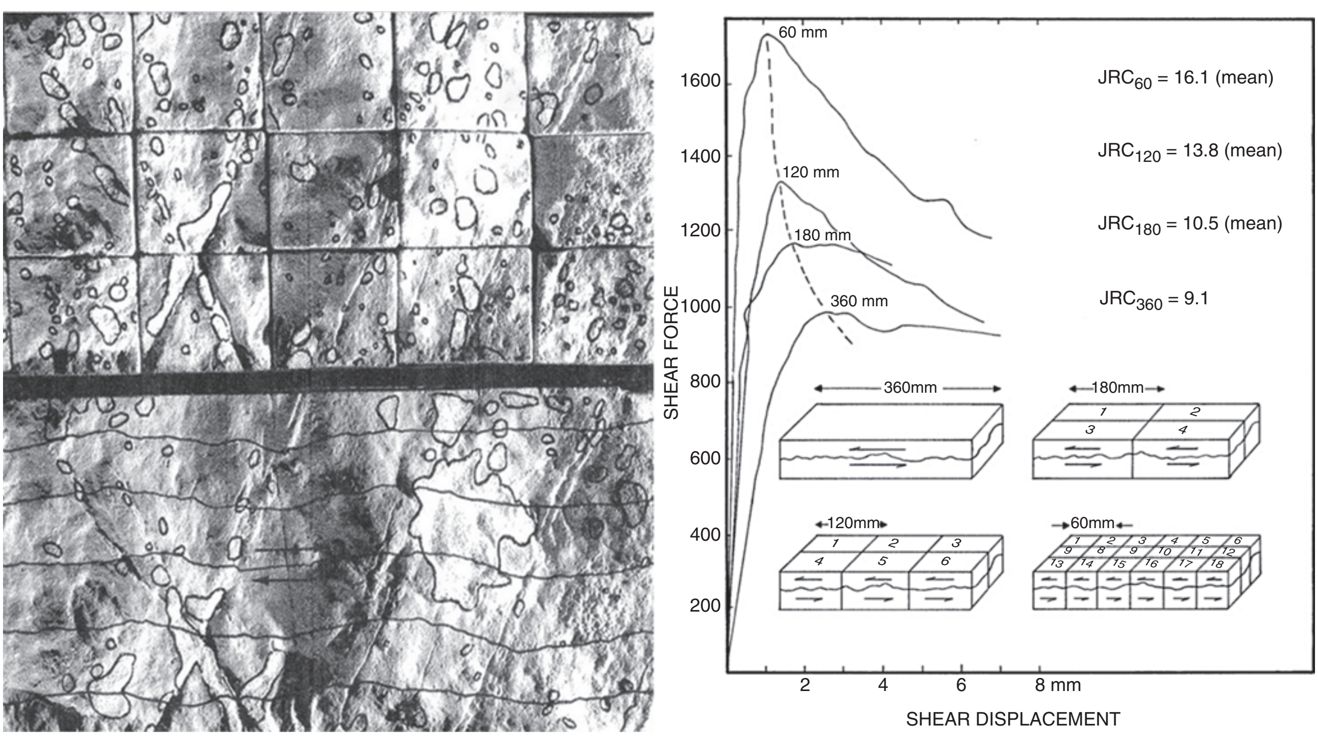
P. T. Bobrowsky, B. Marker (eds.), *Encyclopedia of Engineering Geology*,
https://doi.org/10.1007/978-3-319-73568-9



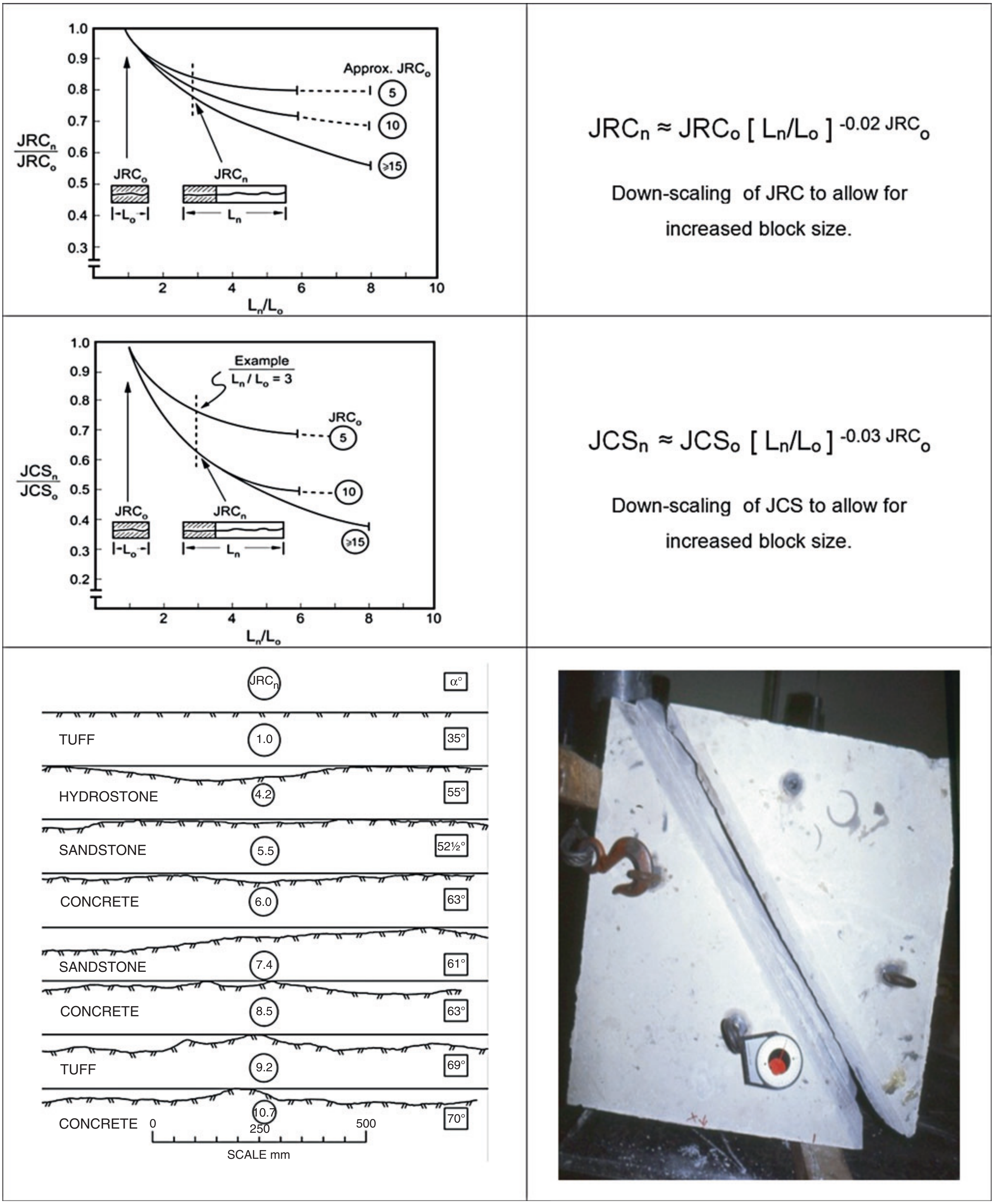
Barton-Bandis Criterion, Fig. 1 Top: Four columns of diagrams showing *1*. direct shear tests principles (Note: apply shear force T “in-line” to avoid creating a moment), *2*. tilt test principles for measuring JRC_0 with jointed-block samples, and ϕ_b with drill-core *3*. Schmidt hammer test principles for measuring JCS, and *4*. roughness recording with profile gauge, and a/L (amplitude/length) method for estimating JRC_n at larger scale (Barton 1999). Bottom: Example roughness profiles and the ten samples with JRC ranges, tilt tests for JRC and φ_b. (Barton and Choubey 1977)



Barton-Bandis Criterion, Fig. 2 Left: Three shear strength criteria compared: *1*. Linear Mohr-Coulomb (with an assumed cohesion intercept c), *2*. Bi-linear Patton (φ + i) and *3*. Continuously curved Barton formula, termed Barton-Bandis when scale-effects are included. Right: The peak shear strengths of 130 joint samples, and examples of the maximum, mean and minimum strength envelopes, with JRC, JCS and φ_r values. (Barton and Choubey 1977)

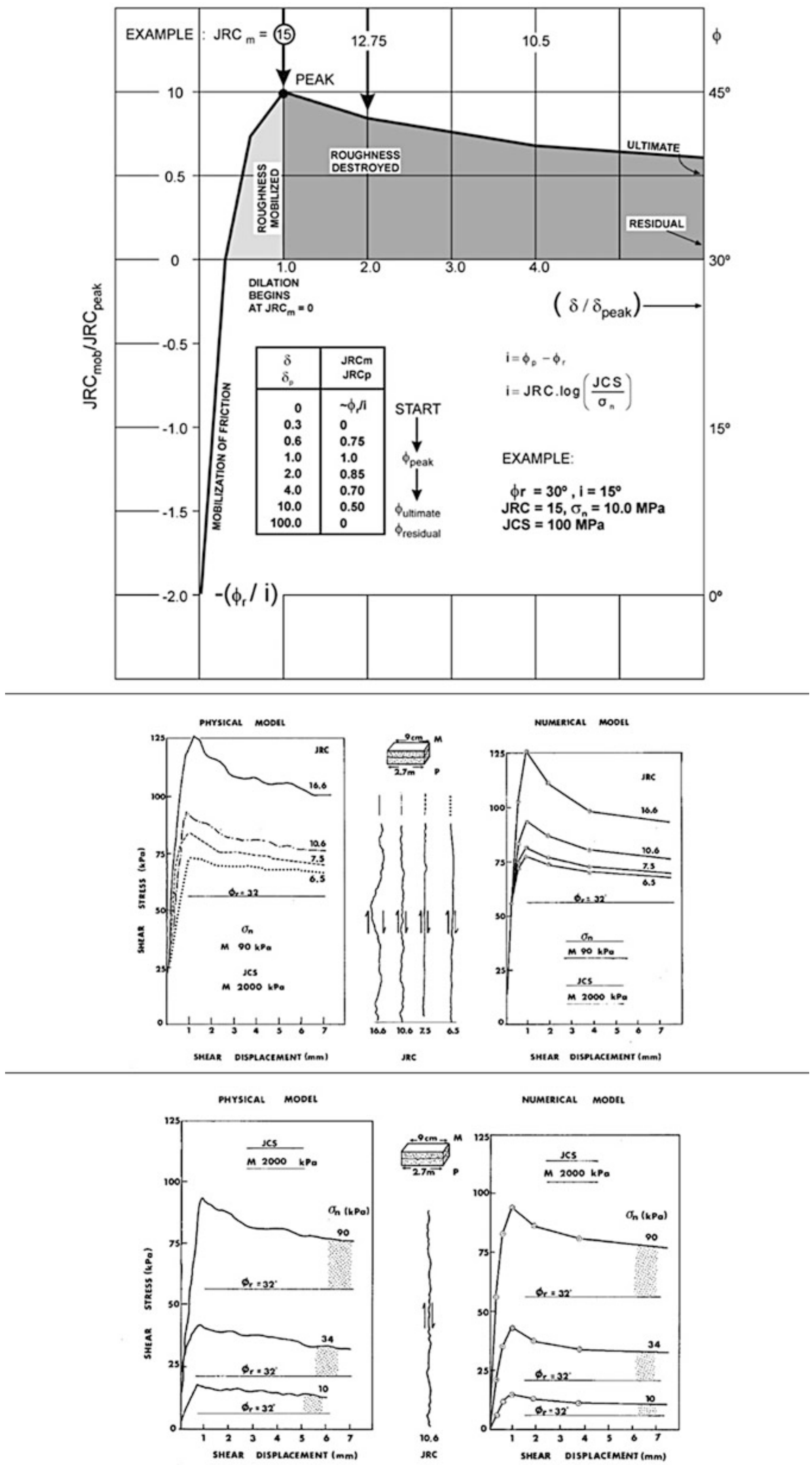


Barton-Bandis Criterion, Fig. 3 One example of the scale-effect studies by Bandis 1980 using replica castings of rock joints, which were direct shear-tested at different scale. (Effect on JRC greatest for the roughest joints) (Bandis et al. 1981)

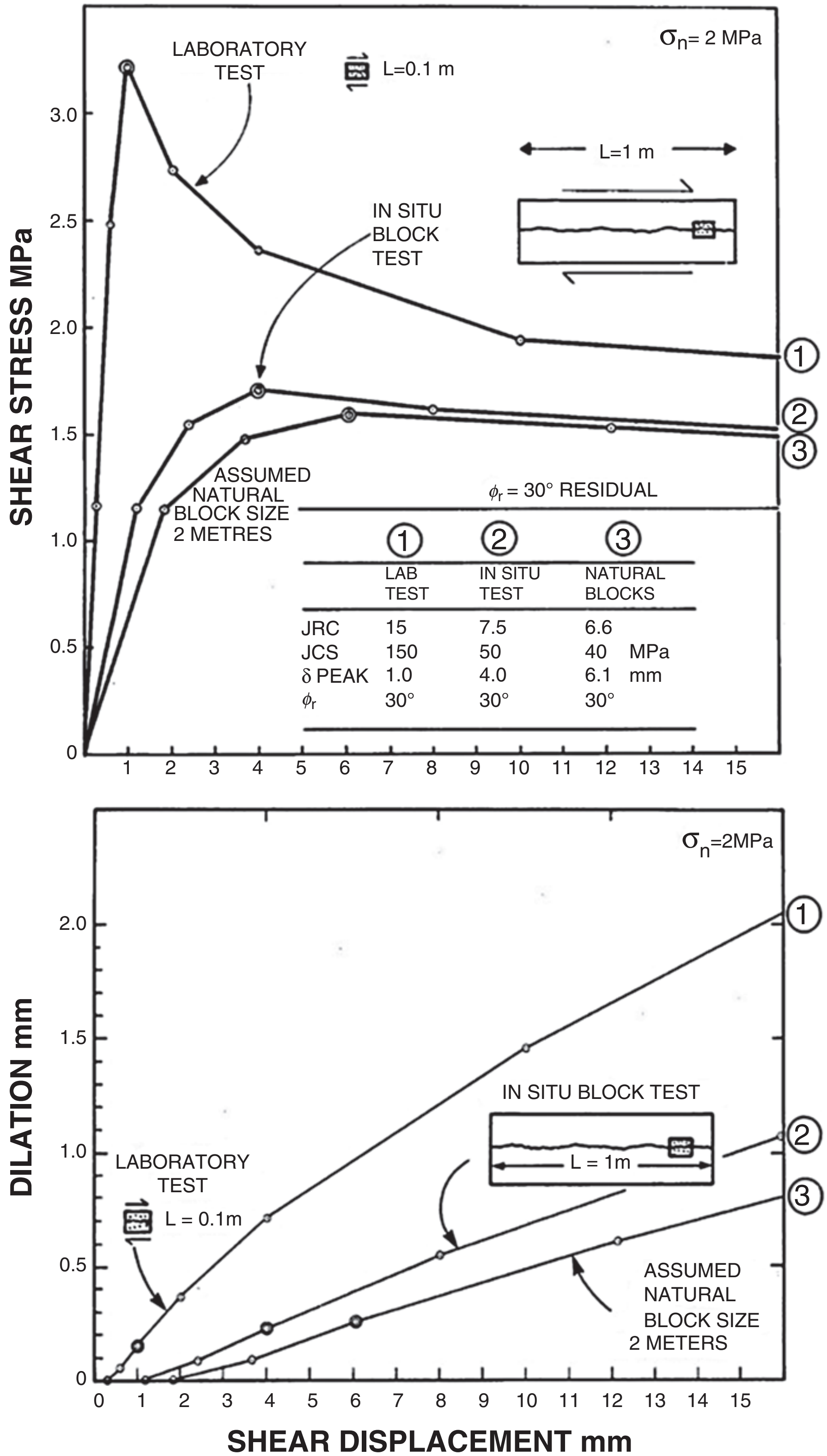


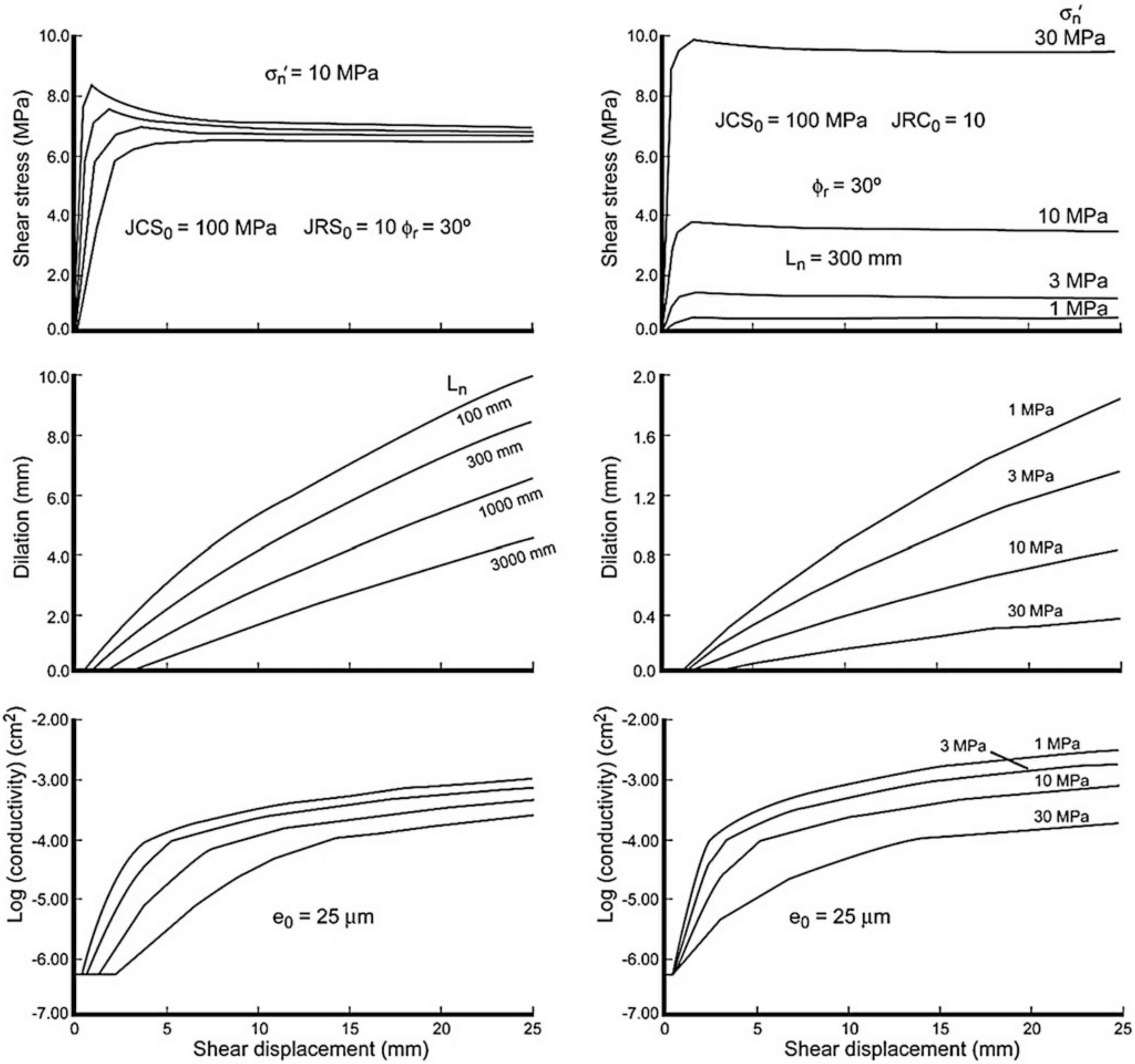
Barton-Bandis Criterion, Fig. 4 Formal allowance for the scale-effect on JRC and JCS which depends on the block length Ln (in practice the mean spacing of a crossing set of rock joints) (Barton and Bandis 1982). Roughness profiles measured on 1,300 mm long diagonally-jointed 1 m^3 blocks, with tilt angles (α) and measured JRC_n values (Bakhtar and Barton 1984)

Barton-Bandis Criterion, Fig. 5 The $JRC_{mobilized}$ concept illustrated in the upper diagram allows shear-strength-displacement (and accompanying dilation and conductivity changes) to be modelled. This coupled behavior is modelled in the distinct element (jointed-media) code UDEC-BB. (Barton 1982; Barton and Bandis 2017)

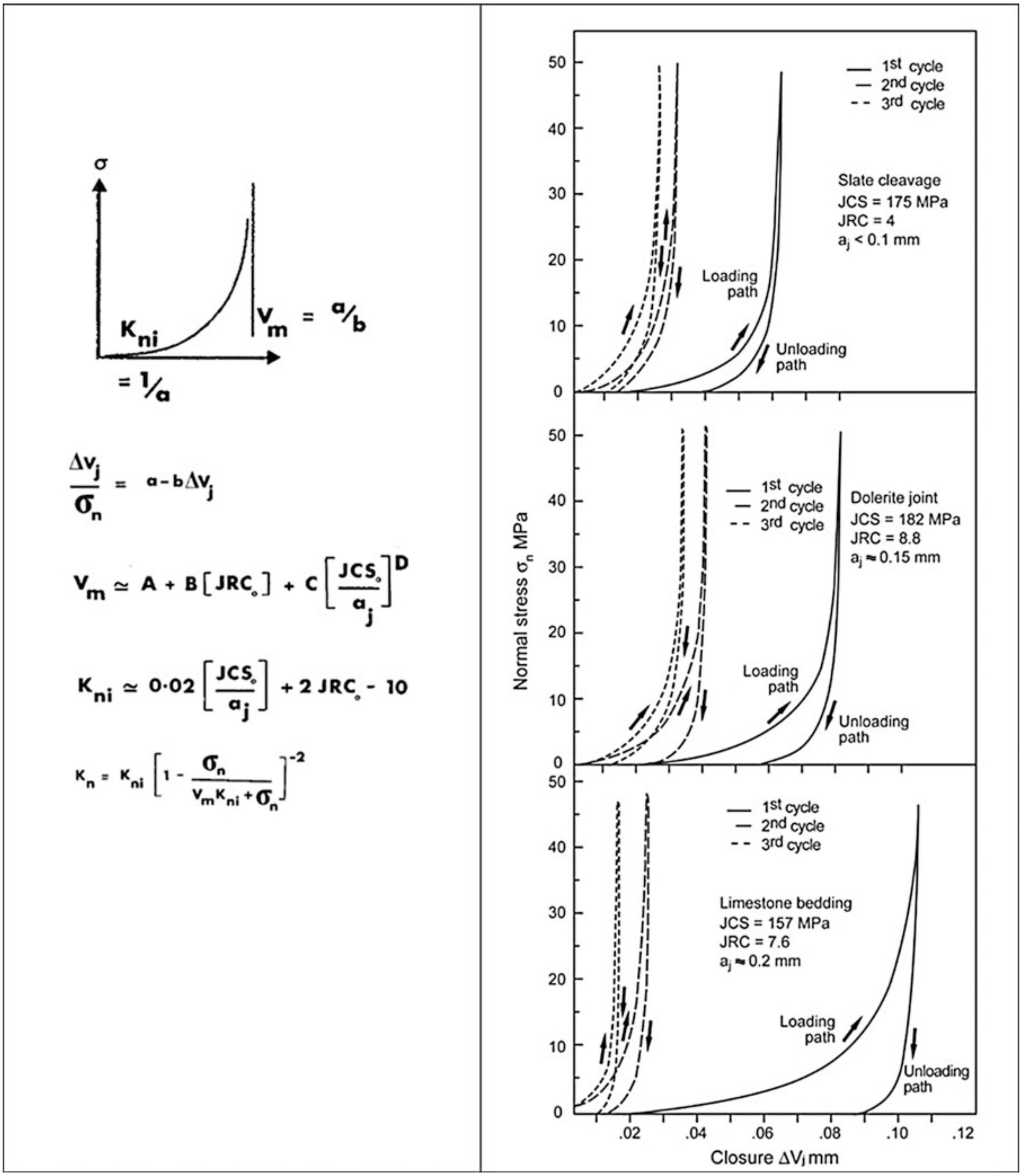


Barton-Bandis Criterion, Fig. 6 Shear-displacement-dilation behavior, for three different block sizes. Barton (1982). Note the inset showing the scaling assumptions from the Bandis et al. (1981) equations given in Fig. 3. Note increase in δ_{peak} as block size increases. Since there is also a reduction in peak shear strength, the peak shear stiffness Ks suffers a double scale-effect as block-size increases





Barton-Bandis Criterion, Fig. 7 Examples of "coupled" shear-dilation-conductivity modelling with the Barton-Bandis modelling assumptions. When block-size variations are involved (left) the delayed dilation and therefore delayed conductivity change can be noted. These curves were produced in 1983 by Bakhtar using a programmable HP calculator (Barton and Bakhtar1983, 1987) the BB equations by now assembled in Barton 1982. ONWI and AECL funded work were responsible for the "finalization" of the BB model prior to its programming (by Mark Christianson of Itasca) into the distinct element code UDEC-BB. (Barton and Bakhtar 1983, 1987), using (see also Barton and Bandis 2017)



Barton-Bandis Criterion, Fig. 8 The nonlinear modelling of joint closure by Bandis (1980) and Bandis et al. (1983). The three load-unload cycles are designed to mirror the experimental evidence of a large hysteresis on the first cycle due to the unavoidable effects of taking a (drill-core) sample which releases the original in situ normal stress. The first load cycle re-closes the joint. The properly consolidated (cycle 3) behavior is incorporated in the distinct-element (jointed) code UDEC-BB

Cross-References

- ► Mohr-Coulomb Failure Envelope
- ► Rock Mass Classification
- ► Shear Strength

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Beach Replenishment

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Synonyms

Beach nourishment

Definition

The process of adding sediments to a beach in order to replace material that has been removed by the continuing physical action of waves and tides over a period of years.

What Is Beach Replenishment?

Beach replenishment is the placement of *beach material*, usually sand, on a beach in order to increase the beach volume (Dean 2002; Kamphuis 2010; Reeve et al. 2011). It is used as a component of *flood defense* to absorb/dissipate incoming wave energy or in *coastal protection* schemes to counter long-term *erosion*. Current practice is to use material of color, size distribution and type as close to the natural local material as possible. In many open coast situations, beach replenishment forms part of a larger flood defense or coastal protection scheme that includes hard structures such as groins (Coastal Engineering Manual 2008).

When Is Beach Replenishment Carried out?

Many of the world's beaches are now actively managed, which will often include monitoring the beach width, levels, and composition. Many responsible authorities will have set criteria, such as a minimum beach level they are willing to tolerate, which act as a "trigger" to commence a replenishment.

Where Is the Material Placed?

There are several options for the placement of replenishment material: at the top of the beach where it acts as a store of material accessed only during storms, across the beach profile to replicate a natural beach, and at the bottom of the active profile where it may be moved onshore during quiescent periods to build up the entire beach profile.

How Is the New Material Deposited on the Beach?

Replenishment at the top of the beach is often the cheapest option as it is a land-based operation. Material will need to be brought to the beach location and shaped to the desired form with bulldozers (Fig. 1). Distribution across the profile is normally achieved with a dredger "rainbowing" a slurry of sand and seawater onto the beach during low tide where it drains and can be shaped using bulldozers. Placement at the base of the beach is usually performed using a bottom-opening barge discharging at high water to avoid the vessel grounding.

Latest Developments

Beach replenishment is a continuing process and might, for example, involve adding material once every 5–10 years over the course of a 50-year project. This can make replenishment a more expensive option due to the mobilization costs involved in repeated placement of material. These costs have prompted the

Beach Replenishment, Fig. 1 Beach replenishment at Herne Bay, UK (With acknowledgments to Claire Milburn, Canterbury City Council)

idea of "mega-nourishments," whereby extremely large volumes of material (in excess of 20 million m^3) are deposited in a single phase, thereby incurring only one set of mobilization costs Sand Engine web. The addition of this amount of sediment to a beach represents a major perturbation and may alter the prevailing morphodynamic conditions, which is the topic of ongoing research.

Cross-References

- ▶ Coastal Environments
- ▶ Erosion
- ▶ Floods
- ▶ Nearshore Structures

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Sand Engine web site.: http://www.dezandmotor.nl/en/the-sand-motor/introduction/

Bearing Capacity

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Definition

Bearing capacity is the maximum stress or pressure that a footing can sustain without failure of the soil or rock that is supporting the footing. Bearing capacity is a function of the shear strength of the soil material or rock mass, but it also depends on the size and shape of the footing and the thickness of soil or rock adjacent to and above the base of the footing.

Context

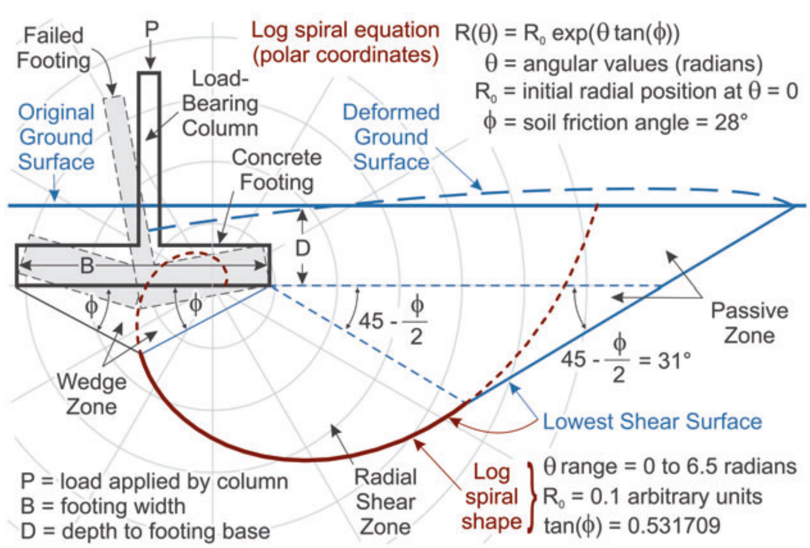
Bearing capacity is a soil–structure interaction phenomenon. Typically, it is associated with *foundations* of buildings, which is the domain of structural and geotechnical engineers. Engineering geologists provide valuable site characterization details pertaining to the nature and uniformity or variability of subsurface earth materials, as well as the geohazards that might affect site suitability or represent constraints that require design provisions or mitigation by ground improvement prior to construction.

For buildings supported on shallow foundations (spread footings), the weight of the building is calculated by the structural engineer and proportioned to the walls and columns that provide the structural support system for the building. Walls are supported by spread footings that extend under the entire length of the wall (continuous spread footings). Columns are supported by isolated spread footings. In some cases in which a shallow foundation are determined to be suitable for structural support, but building performance needs to be enhanced for rare events, such as earthquake shaking, foundation systems may be enhanced by tying isolated spread footings together with grade beams. Grade beams are reinforced concrete elements that are not relied upon to contribute to vertical load-bearing capacity of a building's foundation system, but act as structural elements that add stiffness to transform isolated spread footings into a connected network of spread footings.

In certain geologic settings (for example, Holocene marine clay deposits) or for very heavy foundation structural loads (tall buildings) or for facilities that generate strong ground vibrations (reciprocating compressors), shallow spread footings would have insufficient capacity or would result in intolerable settlement of the building because of consolidation of clayey Earth materials. Deep foundations (shafts or piles) are used to transfer loads deeper into the soil profile to a strong layer or to a depth sufficient for the load to be distributed along the length of a shaft or pile. Deep foundations have bearing capacities which are derived mostly from friction or adhesion of soil along the sides of the foundation elements, with typically small contribution of bearing at the ends of the shafts or tips of the piles.

Geotechnical engineers use shape factors to account for the stress distribution differences associated with footings of different shapes that bear on soil layers that are suitable to support the structural loads. Shallow footings may be isolated or continuous for columns or walls and have widths that are designed for the bearing capacity of the soil. The base of the shallow footing may bear on soil less than 1 m below the ground surface adjacent to the footing, or it may be designed to bear on soil several meters below the ground surface. The ultimate bearing capacity is the maximum load that can be applied on a footing of specified dimensions that approaches, but does not exceed, the calculated soil shear strength. Variabilities in soil properties across the footprint of a building and uncertainties of temporary loads caused by wind and earthquakes are managed with an engineering approach called "factor of safety," which is the ratio of the soil's shear strength to the expected stress transmitted to the soil at the base of the footing. The geotechnical engineer's best estimate of soil shear strength is used with information from the structural engineer and footing shape factors and embedment depths to calculate the ultimate bearing capacity of the foundation soil. The ultimate bearing capacity is divided by the factor of safety, commonly 3 or higher for foundation engineering, to calculate allowable bearing capacity.

Three types of shallow bearing capacity failure can occur: general shear failure, local shear failure, and punching shear failure. Foundation failures typically are rare, but general shear failures (Fig. 1) are relatively more common than the other types. General shear failure results from development of a shear surface below the footing that extends to the ground surface and produces distinctive bulging of the soil. Local shear failure results from consolidation or compaction of soil under a footing in a way that a shear surface is well defined near the footing, but shearing becomes distributed away from the footing; bulging of soil on the ground adjacent to the footing is noticeable. Punching shear failure results from a geotechnical condition of a relatively strong surface soil layer that forms a crust over a weak soil layer; the structural load essentially pushes the footing and strong soil into the underlying weak soil layer, causing consolidation or compaction of the weak soil without noticeable bulging at the ground.



Bearing Capacity, Fig. 1 Cross section of a general shear failure of an isolated shallow spread footing in response to a load P that exceeded the bearing capacity of the foundation soils. Geometry of general failure surface for Terzaghi's bearing capacity formulas is discussed in Coduto et al. (2016)

One type of bearing capacity that involves rock materials is support of pillars in room-and-pillar mines, such as used in some coal mines (Darling 2011). In these cases, engineering geologists or geological or mining engineers measure *in situ* stresses and calculate lithostatic stress that would need to be carried by the pillars. The rock comprising the floor of the mine would be the foundation material for the pillars that act as columns in the structural support system of the mine. Pillars that are too small in cross-section area tend to have stress concentrations that exceed the strength of the rock in the pillar, as well as exceed the bearing capacity of the rock in the mine floor. In coal stratigraphy, a common bottom-to-top sequence might be sandstone-siltstone-claystone-shale-coal-sandstone. The coal formation would comprise the pillars, whereas the claystone-shale would comprise the foundation material. Coal tends to be brittle with a relatively low Poisson's ratio and claystone-shale may be relatively weak, particularly if it becomes saturated.

Cross-References

- ▶ Angle of Internal Friction
- ▶ Consolidation
- ▶ Factor of Safety
- ▶ Foundations
- ▶ Liquefaction
- ▶ Poisson's Ratio
- ▶ Pore Pressure
- ▶ Pressure
- ▶ Shear Strength
- ▶ Site Investigation
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Properties

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Bedrock

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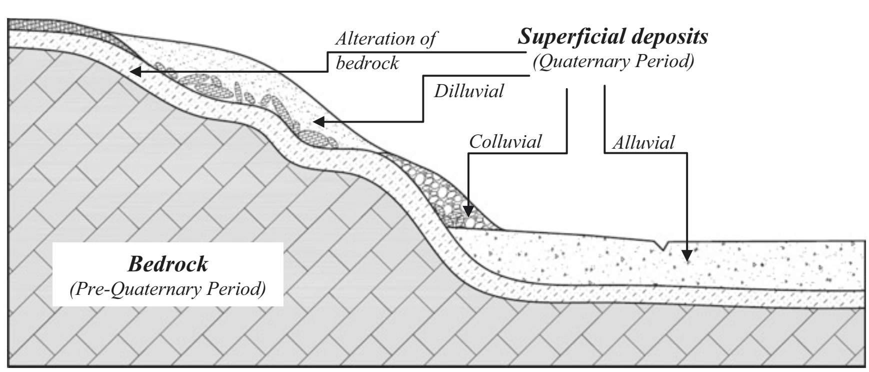
Definition

Bedrock consists of "soils and rocks that were in place before the Quaternary Period" (British Geological Survey 2011). This definition may be classified as "*a stratigraphical criterion*".

The concept of bedrock in geosciences has, despite it's apparent simplicity and worldwide use, different meanings accordingly to the different fields of activity in which it is applied. Thus, all magmatic, metamorphic or sedimentary rocks, beside sedimentary soils older than about two million years, exposed at the Earth's surface (outcrop) or overlain by unconsolidated deposits form the bedrock of a region. But, sometimes, a sedimentary layer from the Quaternary Period, may be classified as bedrock if it was subjected to tectonic stress, reflected in visible folds or faults and lithification. It is appropriate to apply this second "*tectonic criterion*" in seismic regions, where tectonic stress is still active (Florea 1969).

In contrast, unconsolidated Quaternary deposits, as alteration products of bedrock, residual soils, regoliths or saprolites, are distributed over bedrock in different geomorphologic features and formations (alluvial, diluvial, or colluvial) and are defined as "shallow or surficial deposits" (see Fig. 1).



Bedrock, Fig. 1 Superficial deposits resting on bedrock (After Florea 1969)

The basic distinction between bedrock and surficial deposits is applied in engineering geology when acquiring qualitative information regarding soil properties, prior to the execution of geotechnical *in situ* or laboratory tests and is most relevant if we compare mechanical properties of soils of similar kinds. For instance, clay soils in bedrock formations (e.g., terrace formations of the Tertiary Period) tend to be normal to overconsolidated and have low compressibility in contrast to clay soils formed as surficial deposits (e.g., alluvial plains of the Quaternary Period) which are usually underconsolidated with high or very high compressibility.

From the civil engineering point of view, bedrock or engineering rockhead define the rock or soil that has adequate bearing capacity for large structures. Quantitative criteria for differentiation of shallow and bedrock deposits are very variable reflecting the state of professional practice and regulations in the region concerned. The most widely used tests are Standard Penetration Tests for soils and Rock Quality Designations and unconfined compressive strength for rocks.

In earthquake engineering the term "bedrock" is used to define two limits of the geological structure based on the shear wave velocities. The upper limit named "engineering bedrock" is defined by Vs> 700 m/s dividing the shallow deposits from the bedrock. The deeper limit, called "seismic bedrock", is defined by Vs> 3000 m/s and marks the upper interface of the upper Earth crust (Nath 2007).

Bedrock maps reflect the distribution of rock units, their geometric relationships, tectonic setting, as the origin of each unit and may be produced as base research maps for engineering projects, soil chemistry, natural plant ecology, water supply, contaminant transport issues, or other purposes.

Cross-References

▶ Classification of Rocks
▶ Earthquake
▶ Geophysical Methods
▶ Rock Mechanics

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Biological Weathering

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Synonyms

Biodeterioration; Organic weathering; Weathering by organisms

Definition

Mineralogical components of rocks are altered and modified when exposed to Earth surface conditions in response to different atmospheric agents and insolation that may result in the disaggregation (physical weathering) or the decomposition (chemical weathering) of the rock. When these processes are assisted by biologic action they are called biological weathering.

Organisms may alter rock by both mechanical and chemical actions. The penetrating and expanding pressure of plant roots in cracks, fractures, pores and other discontinuities may cause the rupture and disaggregation of the rock, if there are favorable conditions and the strength of the rock is lower than that applied by the roots (Fig. 1). Penetration and expansion of lichen thalli has a similar behavior to that of the roots since some thalli may expand up to 3900 per cent due to their high content of gelatine (Bland and Rolls 1998).

Organic activity, mainly caused by microscopic organisms as bacteria, fungi, lichens, mosses, algae, etc. and also by animals, plays an important role in the decomposition of the rock. Attack is by chemical means, with the segregation of compounds as CO_2, nitrates, and organic acids as metabolic products, resulting eventually in the total alteration of the rock and soil formation.

The presence of water is essential to enable the growth of microorganisms and plants. Production of CO_2 and organic acids and nitrification increase the dissolution capacity of soil water.

Heavy metals (copper and zinc or even metal alloys, such as bronze) may inhibit biological growth.

An overview of biological weathering is presented in Yatsu (1988) where the general aspects and the contribution of microorganisms, plants, and animals are described.

Biological weathering is also observed in natural stone used for buildings and monuments (Caneva et al. 2009) where the damage caused by microorganisms depends on the species, fixation mode, and rock type, as well as the local climate, degree of pollution, maintenance, and other anthropogenic factors. In this case, the term biodeterioration is applied, which is the physical, chemical, and/or biological damage

Biological Weathering, Fig. 1 Example of biological weathering by growth of tree roots in granite

effected by organisms on an object of historic, cultural, artistic, or economic importance (Griffin et al. 1991). Hueck (2001) defines biodeterioration as any undesirable change in the properties of materials caused by the vital activities of organisms.

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Blasting

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Definition

Controlled blasting is the carefully designed and successive placement of explosives with timed sequences of detonations to safely excavate low-tensile strength materials to a defined surface. It may be utilized to conduct rapid removals of materials, while minimizing the risk of varied adverse impacts.

Blasting is an effective procedure, because chemical energy of the explosives is rapidly used to perform work. Two techniques utilize this rapid energy release: controlled blasting of low-tensile strength materials (rock and concrete) to be fractured and displaced to more easily processed sizes; and severance of metal frameworks, which deploys linear shaped charges detonated at discrete locations allowing the framework to be dismantled and displaced.

The performance of blasting and the potential risk of adverse impacts for both of these techniques are determined by several parameters, and confinement of the explosive charges is a major factor. Controlled blasting confines the charge to allow the rapid expansion of the detonation products' gases to perform work. Severance of steel framing is conducted without confinement of the linear shaped charges; this technique will not be further reviewed.

All uses of explosives must be carefully designed, because blasting is inherently destructive and may adversely affect surrounding assets. Controlled blasting is employed, since it is more rapid and cost-effective than mechanical means of excavation or demolition. The primary blasting impacts of air blast (noise), fly-rock (thrown projectiles), underwater overpressure (pressure wave passage through water), and ground vibration are dependent upon the shot pattern parameters, ambient geology and weather conditions, and surrounding built environment.

There are several secondary impacts due to blasting. Blasting projects assess environmental impacts, which in the blasting industry is considered to be only protecting the nearby public or structures. There may be insufficient evaluation of other secondary impacts: sensitive features of certain equipment within buildings or historic and archeological elements; natural resources; and, induced geologic hazards. Natural-resource impacts are those adversely acting upon flora and fauna that have commercial value or are threatened or endangered species. In the USA, natural-resource impacts are often prescriptively assessed under environmental regulations. Several geologic hazards may be induced or triggered by the primary impacts of blasting or may result from the physically excavated or demolished removals. These induced geologic hazards may include: soil displacement toward the removed

Blasting, Table 1 Comparative performance and risks from controlled blasting

			Risk of primary impacts				Risk of secondary impacts				
Blasting location	Confinement	Performance	Air blast	Flyrock	UW OPres	G vibration	Public	Strct'l dm'ge	Sns features	Nat res	Ind geol haz
Land-based blasting possibly near water	*E*	*P*	**L**	**L**	**L**–M	H	H	H	H	**L**	H
	A	**G**	**L**–M	M	**L**	M	**L**–M	**L**–M	M	**L**	M
	P	*P*	H	H	**L**	**L**	H	**L**	**L**	**L**	**L**
Underwater blasting with water depth greater than 0.9 m, possibly near land or structures	*E*	*P*	**L**	**L**	M	H	M–H	H	H	M	H
	A	**G**	**L**	**L**	**L**–M	M	**L**	**L**–M	**L**–M	M	**L**–M
	P	*P*	M	**L**	H	**L**	**L**	M	**L**	H	**L**
Confinement of explosives	*E* excessive; **A** adequate; *P* poor										
Performance of blasting	**G** good; *P* poor										
Impacts' risk level	H high; M moderate; **L** low										
Primary	Air blast [overpressure]	Secondary	Public – nuisance to the public								
	Flyrock [projectiles]		Strct'l Dm'ge – structural damage								
	UW Opres – UW overpressure		Sns features – sensitive features								
	G vibration – ground vibration		Nat res – [biological] natural resources								
			Ind Geol Haz – induced geologic hazards								

material, slope instability, karst collapse, changes of the ground-water flow, and liquefaction/lateral spreading.

Any project, which may involve blasting, should assess whether any adverse impacts may occur before developing contract specifications. Owners need to be informed of the assessed risks from both primary and secondary blasting impacts. Additional insights related to blasting can be found in: Keevin and Hempen 1997, Hempen 2008, California Department of Transportation 2017, Skeggs et al. 2017, and US Army Corps of Engineers (in press).

Table 1 notes the approximate likelihood of meeting a project's objectives and of varied impacts. The location of the blasting on land or beneath water and the confinement of the explosives forecast the blasting's performance and risks of impacts. Contract-specified test blasting programs require the cautious development of explosive weights, which may be increased with successive, acceptable shot patterns to the final blasting design. Adequate confinement and good performance with low risk of adverse impacts may be developed under a sequential test blasting program.

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Borehole Investigations

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Synonyms

CPT sample; Drill hole study

Definition

Method of study to investigate soils and rocks in the Earth's subsurface by means of long and narrow holes drilled using a variety of specialized methods.

Introduction

Successful engineering works often benefit from a clear and better understanding of the nature of soil and rock below ground. In the absence of extensive trenching and excavation and to complement non-invasive geophysical exploration techniques, borehole investigations can be carried out, including the analysis and characterization of the soil and rock recovered. Such investigations allow the identification of the soils or rocks present, as well as an understanding of their physical properties on the basis of field and laboratory tests.

Borehole investigations allow practitioners to determine the nature and location of the different soil/rock layers, collect samples, carry out *in situ* tests and permeability tests, and, if necessary, install piezometers and other subsurface monitoring tools. The location of the boreholes is chosen depending on the objective of the project and characteristics of the tests, with due consideration to the type of works planned.

Borehole Systems

Boreholes may be drilled by two primary systems: percussion or rotary techniques. The former relies on the use of a tool that advances with successive hitting movements, driven by a hammer that is dropped, with its energy transmitted by means of rods to a solid tool or hollow tube (sampler) placed at the bottom of the borehole. This system, which has advantages in unconsolidated soils (silt, sand and gravel), usually takes longer and is more expensive than rotary drilling. The rotary technique (Fig. 1) is the most frequently used method for subsurface exploration. A cutting tool is used to collect samples using a helical auger or drill bit that moves forward by means of a bit crown that is usually WIDEA or diamond tipped.

In the case of auger drilling, alternative methods are needed to obtain samples, which is normally carried out discontinuously. This technique is mainly used in uncemented soft to medium consistency soils or in rocks.

In rotary core drilling, a rock cylinder referred to as "core" may be extracted as the drilling advances and stored in a pipe screwed to the crown, which is called a "core barrel." This may be a simple tube or a rotating double tube in which the inner tube is mounted on bearings.

In the case of loose or very soft soils, a simple tube must be used, whereas a rotating double tube is preferable in all other cases. A casing pipe is introduced into the borehole to prevent cave-ins or stop water leaks; the casing is telescopic and allows the insertion of the core barrel to continue drilling. The exterior diameter of normalized boreholes ranges between 54 and 143 mm.

Borehole investigations for different civil engineering works must be approved by local state agencies, complying with requirements usually determined by ASTM standards or similar specifications.

Sampling

Samples are representative portions of the soil/rock that are collected for visual examination or to conduct laboratory tests. Depending on the means of collection, they may be classified as disturbed or undisturbed samples (USDA 2012).

Disturbed samples only preserve some of the soil/rock properties in their natural state and are usually stored in bags or as core segments. Undisturbed samples preserve, at least in theory, the same properties as the *in situ* soil, reflecting the soil/rock characteristics in their natural state at the moment of collection and, consequently, their physical structure.

In order to undertake laboratory tests, it is necessary to collect undisturbed samples, which are obtained by means of core barrels from the boreholes. Once the core barrel has been extracted, the core within is retrieved and placed in a core box. After the collected core is laid out, it is visually inspected and the recovery obtained is measured.

Core samples must be placed in adequate core boxes made of wood or waxed cardboard, maintaining the original position and

Borehole Investigations, Fig. 1 Rotary system of drilling in unconsolidated sediments orientation, and indicating the depth. For this operation to be properly carried out, the same sequence in which the samples were obtained must be followed, introducing separation blocks between the different core runs and defining sampling depths.

As well as the core recovery percentage, the rock-quality designation (RQD) of all the core samples obtained is determined (Deere and Deere 1988). This index, expressed as a percentage, is defined as a quotient between the sum of the length of the core pieces and the total length of the core run.

There are different tools for the collection of samples, and depending on their characteristics, disturbed or undisturbed samples will be obtained. The use of a Shelby tube sampler is preferred in cohesive silty and clayey soils, whereas a split-spoon sampler is used in sandy soils (Small 2016).

A sample extracted by means of a hand or machine-driven auger consists of a short cylinder that is obtained from the combination of rotation and downward force. Samples collected in this manner are regarded as disturbed.

Undisturbed soil samples may be collected by means of thin-wall coring tubes that are pushed into the ground. Thick-wall coring tubes are driven into the soil with a hammer in order to collect soil samples with some cohesion. The sample within the tube is a representative sample, but it is not considered undisturbed.

In order to avoid dropping the sample from the tube due to the thrust of water when operating below the water table, a valve is located at the top of the sampler and it is seated on the head of the tube to prevent the water from descending and putting pressure on the sample. It is a simple, robust sampler whose greatest disadvantage is that the sample must be pushed to extract it from the tube, which subjects it to a certain degree of deformation.

The Shelby tube sampler is very simple and widely used. It consists of a thin-wall tube, generally made of steel, with a sharp cutting edge. The disadvantage of this type of sampler is that it is necessary to push the sample out of the tube, which causes some disturbance.

Stationary-piston samplers avoid the penetration of mud or prevent the water pressure from affecting the sample as water enters the tube and raises the ball when discharging the water towards the rods. They may be used in soft to moderately stiff clayey soils and in loose sand.

A double-tube soil core barrel has a core lifter that protrudes some 4–9 cm from the crown, which ensures that the drilling fluid will not reach the sample and that the crown will not come into contact with the core. It may be used in clayey soils of hard consistency and the quality of the samples obtained is regular to good depending on the ground conditions.

When the soils are cohesive and their resistance is high, the collection of an undisturbed sample is substituted by dipping in paraffin the longest section of the core obtained.

These sections, once they have been superficially cleaned, must be covered in non-absorbent material and everything must be protected with a paraffin wax seal thick enough to ensure there are no variations in the humidity conditions.

Water samples are collected from boreholes to study the hydrochemical characteristics of the water found in the survey points. It is common to keep a record of the water table level in every borehole (Fig. 2), not only during the drilling but also once it has been completed, at least until the end of the field work.

If bentonite drilling muds or special drilling gels are used during the drilling, the boreholes must be cleaned once they have been completed by means of clean water circulation. Bentonite drilling muds or special gels must be used with caution, especially if the objective is to subsequently carry out permeability tests.

Should it be necessary, or convenient, piezometers may be installed so as to isolate the different aquifers intersected by each borehole.

Every sample and core must be appropriately packed to avoid alterations during transport or storage, and must be shipped as soon as possible to the laboratory. Undisturbed samples must be preserved in a laboratory environment with

Borehole Investigations, Fig. 2 Water samples and water table measurement controlled temperature and humidity. Only the packages with the samples that are going to be tested should be opened, and not until the moment when the corresponding tests are going to be carried out.

Borehole Testing

The main tests undertaken *in situ* in a borehole are as follows:

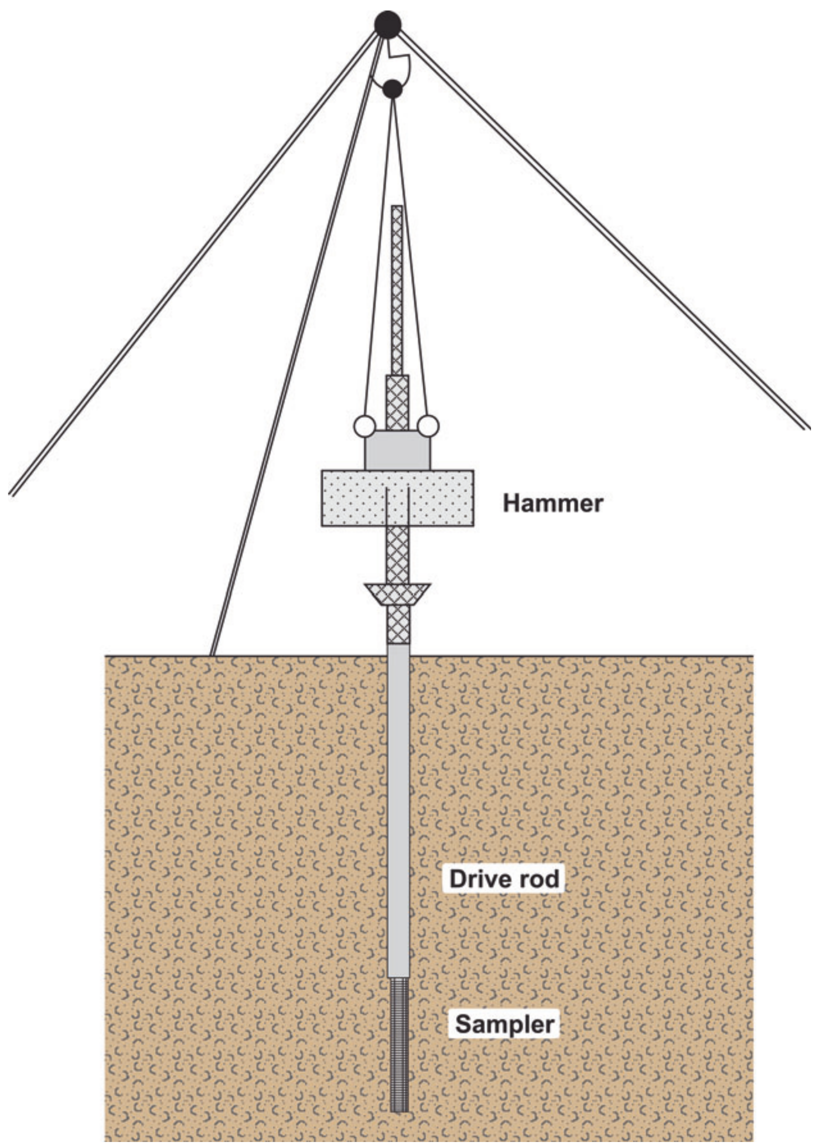
Standard Penetration Test (SPT)

The SPT is the most common test among those conducted within a borehole (Price 2009). It is a simple test and it may be performed while the borehole is being drilled. It may be applied to any type of soil, including soft or weathered rocks. It is possible to correlate the SPT with the mechanical soil parameters; this correlation, together with the data obtained from laboratory tests, helps define the allowable pressure of a soil for a specific type of grouting.

The SPT is an *in situ* dynamic penetration test designed to obtain information on the soil properties, while it also collects a disturbed soil sample to analyze grain size and determine soil classification.

The SPT N-value is defined as the number of blows required to achieve a penetration of 45 cm with a sampler placed in the lower portion of the drive rods. It is driven into the ground by means of a 63.5 kg (140 lb) hammer that is dropped in free fall on the top end of the drive rods from a height of 76 cm (30 inches) (Fig. 3).

Usually, the sampler has an outside diameter of 2 inches and an inside diameter of $1\frac{3}{8}$ inches; in the case of gravel, a conical tip with a diameter of 2 inches and an apex angle of 60° is used.



Borehole Investigations, Fig. 3 Standard penetration test

As a hollow tool is used, the test makes it possible to collect a disturbed sample of the soil in which the penetration test was carried out, so as to analyze in the laboratory.

Pressuremeter and Dilatometer Tests

These are stress-strain tests undertaken directly in the soil in order to identify its geotechnical characteristics, regarding its deformability (pressuremeter modulus) and resistance properties (limit pressure).

They are conducted by the expansion with gas of a cylinder cell against the walls of a borehole, measuring the volumetric deformation of the soil in a horizontal plane corresponding to each pressure until eventually the soil yields.

Regardless of the problem posed by the transformation of the results obtained in the horizontal measurements in the case of the reaction of the foundations, which are usually vertical, and of the fact that soils tend not to be isotropic but heterogeneous, these tests provide isolated and, therefore, discontinuous data as regards the layers encountered. The guidelines to conduct this test are set out in the ASTM D 4719-87 standard.

Permeability Tests

In situ permeability tests are conducted in soils and rocks. The most common ones consist of the addition or extraction of water, under a constant or variable hydraulic head.

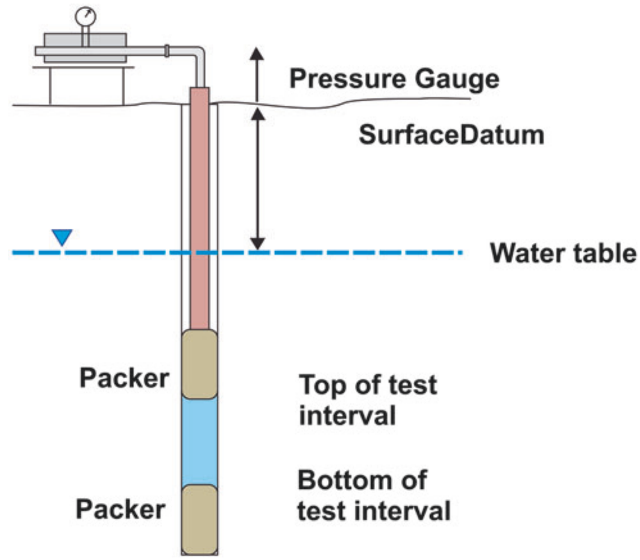
A reliable estimation of the permeability coefficient is possible in surveys that detect the occurrence of the water table and in boreholes in which this coefficient ranges from 10^{-3} to 10^{-5} cm/s. In the case of lower permeabilities, it is necessary to resort to pumping or laboratory tests.

The most frequent ones are referred to as the Lefranc and Lugeon tests (Monnet 2015).

The Lefranc test is carried out within a borehole, during the drilling or once it has been completed. This test estimates the permeability coefficient k in granular soils (gravel, sand, and silt) or in highly fractured rocks occurring below the water table. It is performed by filling the borehole with water and measuring the necessary flow to maintain a constant level (constant-head) or the fall velocity (variable-head). In the constant-head test, as a general rule, the inflow rate is measured at specific time intervals, keeping a constant level at the borehole head. The k coefficient of the section is the average of all the values obtained. The variable-head test is preferably conducted downward, starting from a maximum head of water and recording the decrease in water level within the pipe at different times.

The Lugeon test consists of injecting water under pressure at an isolated section of a borehole bounded by one or two packers and measuring the amount of water that infiltrates into the soil (Fig. 4).

This test can be carried out as the borehole is being drilled or once it is completed. First, the section to be tested is chosen. Once the packers are in place, the injection of water begins, measuring the volume of water injected. The measurement is performed at certain intervals, starting with a minimum pressure and increasing this in stages, all the while measuring the volume of water intake. Starting from the maximum pressure, the same process is repeated but decreasing the pressure at each stage, until the initial pressure is reached. Water is injected by means of a pump, measuring the pressure with a gauge and the volume injected with a flowmeter.



Borehole Investigations, Fig. 4 Lugeon test

This test is applied to medium to low permeability consolidated soils or rocks ($10^{-6} < K < 10^{-9}$ m/s).

Different tests can be performed in the laboratory, which makes it possible to measure a wide variety of soil properties. Some of these properties are intrinsic to the composition of the soil matrix and they are not affected by the disturbance of the sample, whereas other properties depend on the structure and composition of the soil, and these can only be analyzed effectively in relatively undisturbed samples.

Besides geologic logs from drill holes, a suite of geophysical logs can be collected to provide additional information regarding the nature and distribution of materials below the ground surface. Typical data collected in this manner comprise spontaneous potential (SP), resistivity, gamma ray, gamma-gamma, radioactive neutron, and other methods (acoustic, camera televiewer, etc.)

Summary

The direct study methods used in engineering geology are based on geotechnical surveys, which allow for the sampling of subsurface materials and the undertaking of *in situ* tests.

Boreholes are drilled by percussion or rotary techniques, with the latter being the most common. Different types of tools are available to obtain disturbed and undisturbed samples. Undisturbed samples are those that best maintain the physical structure and properties of the soil and lead to more reliable laboratory test results.

Tests undertaken *in situ* in a borehole include penetration tests, pressuremeter and dilatometer tests, and permeability tests. All of these, together with the ones performed in the laboratory on the samples obtained from the boreholes, are essential to understand the characteristics of the soil and to design engineering works to be constructed at the study site.

Cross-References

- ▶ Atterberg Limits
- ▶ Bedrock
- ▶ Boreholes
- ▶ Characterization of Soils
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Boreholes

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Synonyms

Drill hole; Drilled well

Definition

A narrow hole drilled to establish the nature of, sample, test, or monitor soil, bedrock or contained fluids and gases or for abstraction of water or minerals.

Borehole drilling has a long history. By at least the Han Dynasty (202 BC–220 AD), the Chinese used deep borehole drilling for mining and other projects. The British sinologist and historian Michael Loewe (Lowe 1959) states that, at that time, borehole sites could reach as deep as 600 m (2000 ft) but it was after the development of petrol and diesel engines that deep boreholes became generally practicable.

The borehole drilling system consists of a drill head which powers the operation, a drill string which extends down the borehole, and a drill bit which cuts through the substrate. The drill string may be surrounded by a collar separated from the drill string by an annular space. The annular space allows water or mud to be pumped down and for soil and cuttings to be flushed up to the ground surface. The practical limits on the depth and rapidity with which a borehole can be drilled, and the diameter of the bore, is governed solely by the size and power of the rig used. If a borehole is required for use over a period of time, if it is into uncohesive deposits, or if it is deep, a lining or casing of plastic, steel, or iron is sunk to protect the hole from collapse (Arnalich 2011).

The main techniques involved in the construction of boreholes (ICRC 2010) are percussion, rotary, and sonic:

Light cable percussion technique – also known as shell-and-auger, this is a relatively quick and cheap method for drilling to depths up to about 60 m and to recover samples on the auger. It is used extensively in civil engineering and shallow mineral deposit site investigations.

Air percussion technique: This technique of percussion drilling utilizes compressed air to operate a down-hole air hammer on the end of the drill string that helps break the rock formation. This borehole drilling mode always requires the skills of an expert. The compressed air is usually used to operate an air hammer which is situated deep down the hole.

The rotary technique: In rotary drilling, a drill bit is attached to a length of connected drill pipe. The drill bit is made of strong metals such as tungsten, so that the rotating drill bit can easily grind the rock. Drill fluids (sometimes referred to as drilling mud) are circulated through the drill string into the borehole and back to the surface carrying (flush) the broken pieces (cuttings) of bedrock upwards and out of the hole. This fluid also serves as a formation stabilizer preventing possible caving-in of unstable sand or weak rock before the well casing or well screen can be installed. Furthermore, this fluid acts as a drill bit lubricant. As the drill intersects water bearing rock formations, water will eventually flow into the hole. Drillers, or hydrogeologists, on site will carefully monitor the depth of water "strikes" and keep a note of the formations in which they occur.

Sonic drilling technique – an oscillator in the drill head generates high-frequency resonant energy which is directed down the drill string causing soils in a narrow zone between the string and head to lose structure and reduces friction so that soil or cuttings can be readily flushed away.

Boreholes require sophisticated technology with the right appropriate technical design, together with proper knowledge of the underlying target, such as an aquifer or oil reservoir. Unfortunately, the importance of good quality borehole design and construction is often underestimated. The lifetime of a borehole and the efficiency of its functioning depend directly on the materials and the technology used. Borehole "failure" is often linked to incorrect design and construction of the hole. Constructing, or repairing, boreholes requires specialized knowledge and technical expertise, much of which can be gained from the standard literature; but field operations in remote areas or in difficult conditions often require flexibility and imagination in avoiding and solving technical problems.

During borehole drilling, certain problems may be encountered (Azar and Robello Samuel 2007). The drill is expected to act moderately. If pushed down excessively on the rock surface or rotated too quickly, the drill bit may be destroyed. On the other hand, if the drilling does not produce enough force, hard rock layers will not be penetrated. Caution may be needed if the top of bedrock or a competent layer needs to be identified (e.g., prior to piling) because drilling into large boulders might be mistaken for detecting the actual interface. Also, drilling hazards may be encountered (e.g., contaminants, polluted water, unexploded ordnance, or natural gases or oil under high pressure). Problems may also arise if shallow drilling encounters voids such as natural caves or unrecorded mine cavities.

A variety of "downhole tools" are lowered into boreholes for testing and monitoring such as geophysical tools to investigate adjacent strata or to secure data to interpolate between groups of boreholes (Scott Keys, W, Scott Keys 1997) or for monitoring of ground water strata (e.g., piezometers).

Engineering Site Investigations and Works

Site investigations for construction projects or evaluation of potentially polluted or contaminated land mainly consist of relatively shallow drilling and sampling (Clayton et al. 1995; SISG and ICE 2013). Some holes are drilled and then made safe very quickly but others are kept open for monitoring of groundwater or gas emissions. Systematic site investigation requires boreholes to be drilled on a grid pattern with the initial spacing between boreholes being determined by the expected distribution of the ground characteristics that are being examined and new boreholes at different spacings being made if the need arises to investigate unexpected features. Deeper drilling is needed to investigate larger scale problems, for instance to define the slip planes of landslides. Boreholes are also sunk for dewatering of excavations and draining water from landslides.

Water Abstraction

In boreholes for water supply (Figs. 1, 2 and 3), the bottom section of the lining has slots to allow water to enter the borehole. Gravel is placed at the bottom of the lining to improve flow and provide filtration. The design and construction of boreholes and wells for groundwater extraction is very technical and requires expert groundwater hydrologists and engineers. Clark (1988) deals comprehensively with the location and installation of water supply boreholes, whereas the maintenance, monitoring, and rehabilitation of such systems are reviewed by Howsam et al. (1995) and Detay (1997). A less specialized overview is given by Brassington (1995). Unless in an area with artesian conditions, a pump is required to bring water from the borehole to the surface.

Mineral Exploration

When holes or wells are drilled for the purpose of evaluating the content of the hole as retrieved, it is called exploration drilling (Annels 1991). Mining companies utilize this type of

Boreholes, Fig. 1 Drilling a borehole for water

Boreholes, Fig. 2 Drilling a borehole for water

drilling to retrieve mineral samples of a specific location for the purpose of evaluating the samples to determine whether the quality and quantity of a specific mineral are sufficient to make mining at the location viable. Reverse circulation is one method used and it entails retrieval of the drill cuttings, using drill rods that are used for the transport path of the cuttings to the surface. A hammer drives the tungsten steel drill point into the rock or soil. With this method, depths of up to 500 m can be drilled. The retrieved material is dry. Odex and Tricone systems are used in reverse-circulation drilling.

With Odex, the hammer drill bit fits at the end of the steel casing and the hammer is used for crushing the material, which is blown up into the casing where the retrieved cuttings are transported to the surface. The system is used when there is a risk of rock collapsing in the drill area. This is a time-consuming and sometimes very expensive method, but it is

Boreholes, Fig. 3 Drilling a borehole for water

often the only type of method that can be used to prevent rockfalls. The Tricone system is used for water, oil, and petroleum retrieval. It entails roller cone bits grouped into a drill bit that rotates into the rock formation.

It is a suitable method when the drill bits must be protected and when only a small sample of content has to be retrieved. Core drilling is normally used when faster exploration drilling is needed. It can also be used in the construction sector for drilling pipe holes.

Mineral Extraction

A borehole is drilled for extraction of minerals relying on a process that uses high-pressure water. The water jets make it possible to drill into hard rock, whether in an open-pit floor, underground mine space, land surface, or from a vessel in the sea or on a lake. The first step is to drill to the desired depth from the surface. The next step is to lower a casing column into the well with the shoe of the casing situated above the top boundary of the productive mineral. In the case of oil and gas extraction, the fluid may initially rise to the surface through the borehole due to pressure but, as time passes, it becomes necessary to enhance recovery by fracturing the reservoir rocks (Blake 1979).

In ore mining by borehole, a third step involves the lowering of the borehole mining tool into the drill well (Pivnyak et al. 2017). This method requires less capital outlay than many other mining methods and it makes it possible to work in otherwise inaccessible areas that are too dangerous for conventional mining. It has a relatively low environmental footprint and allows for better mobility when it comes to changing mining locations. More selectivity can be applied, meaning less wastage and higher profits. The method is frequently used for mining minerals such as gold, uranium, diamonds, coal, and quartz sand. It is also used for oil and gas extraction. Boreholes can also be used for solution mining, where water is pumped in, dissolves a mineral such as salt to form brine which is then recovered at the surface.

Arrays of boreholes are drilled on a linear or grid pattern for the purpose of inserting charges for blasting to break up the mineral (Gokhale 2010) (Fig. 4).

Directional Drilling

Directional drilling is the drilling of oblique or horizontal boreholes (Azar and Robello Samuel 2007). It is used for a variety of purposes. For instance:

- In deep drilling for oil or natural gas, a borehole is made at a single suitable location and is then used for drilling of a number of oblique borings to further investigate and exploit a reservoir, which minimizes the number of wellheads that affect the surface environment
- To reduce the pressure of a well, in order to minimize the risk of a pressure blow-out

Boreholes, Fig. 4 Grid pattern of boreholes (top left) drilled for blasting at a mineral working (Photograph by Dr. BR Marker)

- For hydraulic fracturing of oil shales to release the hydrocarbons or to exploit coal-bed methane
- Horizontal drilling for installation of utilities

Costs

The costs of boreholes depends on the depth of drilling, the diameter of the hole, and the need for casing, but it is also influenced significantly by the applied design as well as the difficulty to construct a borehole in certain geological formations (Stapenhurst 2009). It is fairly common for developers to try to reduce costs by not allowing for the insertion of casing or by reducing the number of boreholes that is needed to adequately investigate the problem. In the short term, these seem costly but almost always pay off in the long run. Thus casing will allow for the borehole to stay open for years after completion, and if correctly installed, it will also assist in keeping the borehole clean and free of material that could damage a borehole pump. Also, an adequate number of boreholes in a construction site will normally avoid unexpected later costs for remedial works and delays to development. It is important to have professionally competent specialists when designing, undertaking, and using boreholes.

Summary

Boreholes are made for a variety of purposes including the investigation and sampling of the geological succession for construction, water abstraction, or mineral extraction purposes, and monitoring of groundwater behavior and composition.

Cross-References

▶ Aquifer
▶ Artesian
▶ Borehole Investigations
▶ Designing Site Investigations
▶ Dewatering
▶ Drilling
▶ Drilling Hazards
▶ Geophysical Methods
▶ Groundwater
▶ Hydraulic Fracturing
▶ Mining
▶ Piezometer
▶ Water
▶ Water Testing
▶ Wells

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Boulders

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Definition

Rock fragments, particles, or grains larger than 200 mm in size (British soil scale) or greater than 256 mm in size (North American scale). Boulders are the largest grain size within the category of gravel. The term likely derives from the Swedish term “bullersten” or Middle English term “bulderston.”

Context

Boulders range in size from a minimum of 200 or 256 mm to isolated pieces weighing multiple tons (megaboulders or megaclasts). Glacial deposits around the northern hemisphere commonly host boulder sized clasts in the form of isolated erratics or within chaotic multitextural deposits such as ice-contact gravel features. Large glacial erratics are static features generally avoided during construction efforts and more often a tourist attraction (e.g., Okotoks erratic in Alberta, Canada). Typical natural processes capable of transporting boulders include landslides (e.g., debris flows, rock avalanches), lahars, tsunamis, flash floods (including natural and artificial dam bursts), and storms (typhoons, super storms, etc.) (Dewey and Ryan 2017; Hungr et al. 2014). Artificial efforts to move boulder sized rocks require mechanized support (e.g., backhoe).

Resedimentation of boulders is a significant issue and poses risk to humans and infrastructure around the world. In mountainous areas, debris flows are capable of moving large numbers of boulders at fast speeds along natural and engineered channels that are crossed by roads and bridges. The damage to bridge piers and other support structures is substantial and solutions require engineered design needs such as the construction of anti-collision pier structures on the upstream side of bridge piers to reduce potential damage from forces associated with boulder impacts. Comparable considerations are required for dams, weirs and other structures affected by boulder impact and deposition.

In the assessment of boulder movements, the calculated Froude number (a ratio of inertial and gravitational forces) can range appreciably depending on the local conditions: natural versus hard-engineered channels, noncohesive versus bedrock surfaces, bulk density of the fluids, boulder composition and porosity, impulsive forces, etc. Challenges for engineering geologists includes appropriate modelling applications to advise the civil engineers.

Cross-References

▶ Bedrock
▶ Bridges
▶ Drilling Hazards
▶ Fluvial Environments
▶ Glacier Environments
▶ Hazard
▶ Landslide
▶ Mountain Environments
▶ Sediments
▶ Tsunamis

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Bridges

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Synonyms

Trestle

Definition

A raised structure that allows the movement of vehicles or pedestrians over an obstacle.

Introduction

For millennia bridges have been used to cross barriers, typically a river, stream, or valley, by using locally available materials, such as stones, and timber. Originally, cut trees were simply placed across streams to allow crossing. Later, pieces of wood were lashed together to make improvements in functionality of the bridges. Such bridges are known as frame bridges. Since these early times bridge engineering has evolved into a major discipline in itself; one that benefits from the advances made in other engineering disciplines, such as engineering geology, water resources engineering, geotechnical engineering, and structural engineering. Based on these disciplines, modern bridge engineering mainly deals with (a) planning, (b) analysis, (c) design, (d) construction, (e) maintenance, and (f) rehabilitation.

In modern society, bridges facilitate surface transportation for roads and railways and carry facilities such as water/sewer supply pipelines or electric/telephone communication lines across streams or gorges. In congested city centers, flyovers/bridges serve to cross roads without mixing of the traffic moving across in different directions. Therefore, they are an essential part of daily life that aids a prospering trade and commerce in a city. Maintenance and repair of bridges, therefore, has consequences for the economy of the region, which mandates finding technological solutions for increasing their longevity.

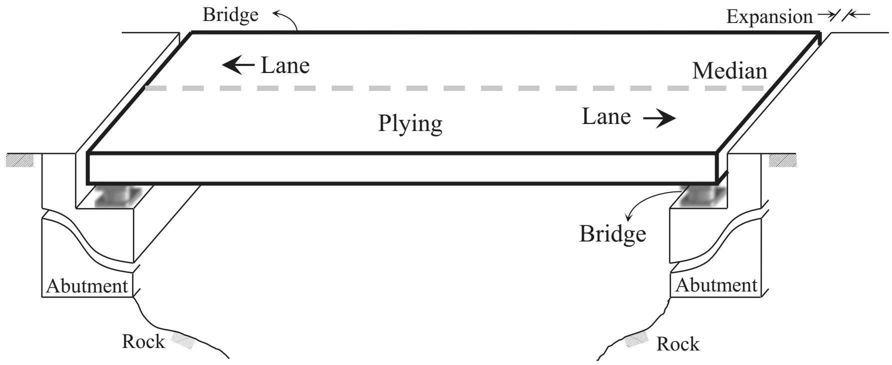
Bridges are called life-line structures because apart from the day-to-day services, during natural calamities such as earthquakes or floods, they facilitate in providing emergency relief by enabling supply of food, medicine, etc., into hazard affected areas. Typically, structural redundancy in bridges is relatively low, which makes them vulnerable to earthquakes and strong winds. The relief and rehabilitation work may therefore be adversely affected if bridges receive severe damage or experience catastrophic failures during natural hazards. Moreover, bridge failure adversely affects commerce and services, incurring hefty repair costs, and of utmost importance, may cause loss of human lives.

Bridge Components, Planning, Analysis, and Design

There are five main components that make up a typical bridge. They include:

- **Pile** – A concrete post that is driven into the ground to act as a leg or support for the bridge. It is driven into the ground using a pile-driver.
- **Cap** – The cap sits on top of a group of piles and disperses pressure to the piles below.
- **Bent** – This is the combination of the cap and the pile. Together, with other bents, act as supports for the entire bridge.
- **Girders** – Girders are like the arms of the bridge. They extend from bent to bent and support the bridge decking. They also help disperse pressure to the bents.
- **Decking** – The decking is the road surface of the bridge. It rests on the girders, supported by the bents that are made of caps and piles.

Bridges are categorized based on their functionality, the type of materials used, type of bridge construction, etc. Flyovers, highway bridges, and railway bridges are some of the major categories of bridges depending upon functionality. Sometimes bridges are built with more than one deck (multiple levels) such that; in a double deck bridge, one deck carries road traffic, whereas the other deck may carry train traffic. The rest of the details provided herein pertain mostly to the highway bridges, but are applicable to other types of bridges with some distinctions. Individual or mixed (combined) materials are used in bridge construction,



Bridges, Fig. 1 A Typical Bridge Structure and Component

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such as steel, concrete, timber. Steel bridges are typically built for faster speed of construction and railway bridges are largely constructed from built-up steel sections. However, steel bridges suffer from corrosion-related degradation, heavier weight, and special detailing requirements to accommodate thermal expansion and contraction. Historically, following the introduction of cement to the construction industry, reinforced concrete (RC) bridges were commonly built. However, today most bridges consist of prestressed concrete (PSC). Furthermore, precast concrete or prefabricated elements are used in bridge construction; however, they are less common. Concrete bridges also suffer from shrinkage and long-term creep-related effects. Hence, newer materials for bridge construction are now being developed.

Typically, a bridge structure consists of a deck supported on bearings, as shown in Fig. 1. The bridge bearings facilitate accommodating temperature-dependent expansion and contraction movements experienced by the deck, and now they are designed to protect the bridge from earthquake-induced lateral forces. The bridge bearings are supported over abutments or pier-caps and effectively transfer the vertical loads from the deck to the abutments or piers. The different types of bearings commonly used in bridges are: laminated/ neoprene bearings, elastomeric (rubber) bearings, sliding bearings, pot-PTFE bearings (Polytetrafluoroethylene), rocker/roller bearings, knuckle bearings, spherical bearings, etc. In addition, when the length of the bridge is long, expansion joints accommodate expansion and contraction in the longitudinal direction (Fig. 1).

The abutments retain Earth on both sides of the stream at the bridge site, whereas they also serve as end supports to the bridge. Sometimes, the abutments are supplemented with wing walls to retain loose soil on either end of the abutments and additionally provide guided protection from erosion resulting from high velocity water flowing in the stream. Behind the abutments, bridge approaches are constructed mostly from earthwork and retaining/RE walls. Hence, the abutments are generally designed as Earth retaining structures of masonry or concrete. Under the action of vertical and lateral loads, such an Earth retaining structure, an abutment is designed as a member that will not develop tensile stress, if steel reinforcement is not to be used. This is ensured by calculating resultant stresses acting within the critical section of the abutment as:

$$\sigma = \frac{P}{A} \pm \frac{M}{I} y \tag{1}$$

where σ is the resultant stress; P is the normal/ direct load on the critical section; A is the area of the critical section in the abutment; M is the bending moment on the section; I is the moment of inertia of the section; and y is the extreme fiber distance.

The piers provide intermediate support to the bridge, which essentially function as columns, that is, predominantly axially loaded members with some bending moments. The bending moments are due to eccentricity of the vertical loads, direct transverse loads applied from earthquake-induced forces or strong winds, lateral earth pressure applied at the abutment may be transmitted through the deck to the piers, thermal movements of the deck induce lateral forces on the piers, and braking force applied by the moving vehicles also induces longitudinal forces on the deck, which may in turn impose bending moments on the piers. For an axially loaded compression member, like a bridge pier, the critical buckling load-carrying capacity (P_{cr}) can be estimated using linear stability theory proposed by Euler:

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{2}$$

where E is the modulus of elasticity; I is the moment of inertia about axis of buckling or second moment of area; and L is the effective length of the compression member, pier. In the presence of the bending moment, in order to calculate the flexural stresses induced in the pier, the classical bending formula can be used:

$$\frac{\sigma}{y} = \frac{M}{I} = \frac{E}{R} \tag{3}$$

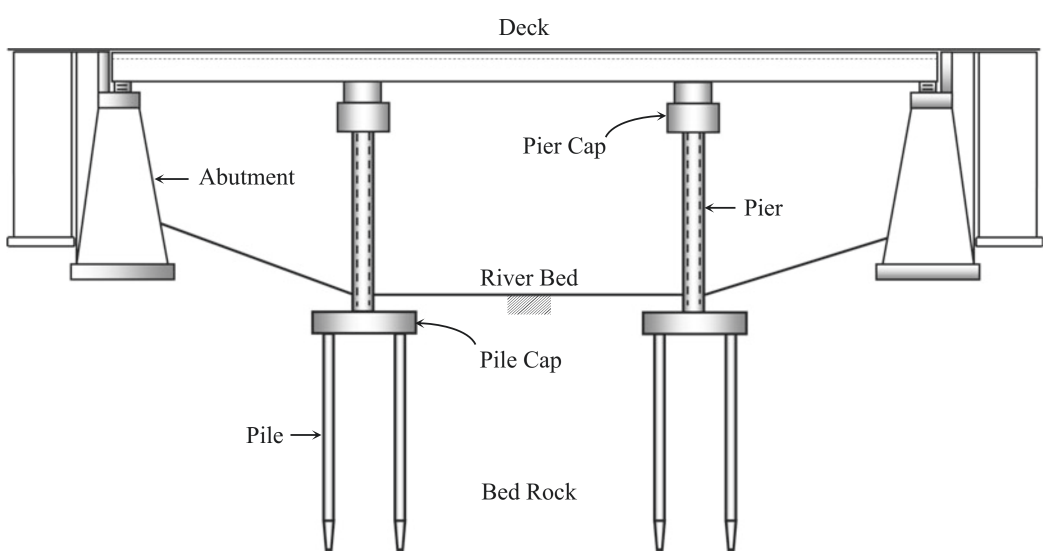
where σ is the bending stress; y is the extreme fiber distance in the cross-section from the neutral axis; M is the bending moment acting on the pier; and R is the radius of curvature of the flexural member, pier. Using Eq. 3, it is possible to calculate maximum compressive or tensile stresses induced in the pier under the action of axial load and applied bending moments. These stresses are subsequently used in the design of the pier.

The bearings divide the bridge vertically in two parts: (a) superstructure above the bearings and (b) substructure below the bearings. Thus, the bridge deck is part of the superstructure, whereas the pier-cap and piers are the parts of the substructure. The bridge deck supports several structural and nonstructural components such as overlays, plying surface, handrails, footpath, curbs, which all form part of the superstructure. Sometimes the substructure also includes returns and wing walls, which are required along with the abutments to support the Earth laterally and prevent erosion of soil on the banks. These two parts of the bridge, superstructure and substructure, are supported over the third part, the foundation.

Depending upon the site conditions, construction feasibility, type of loading applied, and other factors, the type of foundation is determined. Some common types of bridge foundations includes open/isolated footing, spread foundation, well foundation, raft foundation, and pile/pile-group foundation. Figure 2 shows a general arrangement of a typical three-span continuous bridge supported on pile-group foundation. Note that the geology of the bridge site plays an important role in the choice of the foundation. Geological knowledge of the site under consideration for the bridge is useful in the planning, analysis, design, and construction phases. Decisions on the type of bridge to be selected, its foundation type, consideration of the geological features in assessment of the structural behavior of bridges, and construction technology to be adopted are made based on the engineering geology. A reconnaissance survey is conducted during planning stage of the bridge to select such a site along the stream that possibly has well-defined banks, approaches to the bridge on either side are fairly straight, and the foundation strata is firm - preferably hard rock. Boreholes are taken at specific intervals around the bridge site to specific depths to study the bore-logs for assessment of the geological features and thereby deciding the type of foundation to be adopted.

Several types of preliminary surveys are conducted when deciding the site and bridge type, such as traffic survey, topographic survey, geological survey, geotechnical survey, hydrological survey, meteorological survey, which sometimes together is termed as bridge survey. Once the requirements of a bridge on a particular route in terms of volume and characteristics of the traffic are determined from the traffic survey, a topographic survey of the area is conducted to give an idea about the locality or surrounding area about peaks, valleys, obstructions, and elevations. Based on that, a site for bridge construction is decided by subsequently conducting other surveys, including cost-benefit/bridge economics studies and environmental impact assessment, which all form part of the technical feasibility report for the bridge.

For the selected bridge location, a geological survey is subsequently conducted for surface and subsurface investigations to prepare topographic maps, contour maps, and geologic maps, which essentially dictate the foundation design



Bridges, Fig. 2 A Three-span continuous bridge supported on pile-group foundation for the bridge. To avoid any possible support settlement, unyielding founding strata are necessary, which is established based on the boreholes taken in a specific array on the site that provide data about the lithological variation, faults, and rock formations. The borehole locations are selected along the proposed center-line of the bridge with some spacing, which is decided based on variations in the geological features within the stretch. If the geological features are varying substantially, a dense grid of boreholes is warranted. Especially, in case of the long-span bridges where it is anticipated that the geologic environments at the ends and intermediate supports may vary considerably.

The bore-logs extracted from the boreholes are arranged systematically, and borehole data are carefully studied to the desired depths or typically until hard rock is reached. Moreover, a ground water table assessment is made from the geological explorations. At some sites, an aquifer may exist that needs to be accounted for along with the subsurface water, if any, while taking decisions related to the foundation design. Such studies called hydrogeological studies are conducted based on the need at a particular site. Past geological records and reports are also studied to understand variations in the subsurface characteristics through time including reporting of former earthquakes or strong wind-storm events. The seismic data of the site are a key input in the structural design of bridges. For long-span bridges, in particular, the attenuation of the seismic waves along their path of propagation depends upon the geological characteristics, which is also an input for evaluating dynamic response of the long-span bridges under multisupport earthquake excitations. It is essential to reach firm and stable rock in the case of the major bridges; therefore, if necessary geophysical surveys are conducted, such information is used by geotechnical engineers to determine the liquefaction potential at the bridge site. A geological team conducts detailed geological investigation, prepares maps of the local geological features, conducts tests on the sampled geological materials in the field and laboratory, synthesizes and interprets the results, and provides a report to the geotechnical engineer, hydrological engineer, architect, and structural engineer or bridge designer.

For the selected bridge site, based on the local geological investigation, further planning and design of the bridge proceed with hydrological assessment and functional planning. Hydrological data are gathered for the bridge site to determine the water flow characteristics such as discharge, velocity, yearly flood levels, characteristics of rainfall in the catchment area over a typical return period of 120 years. These inputs are used to calculate the design discharge and subsequently to design the free waterway, which is the passage provided underneath the bridges for water to flow from an upstream to downstream direction. The mean velocity (V in m/s unit) of the water current in a stream is calculated using Chezy's formula:

$$V = C\sqrt{R\ s} \tag{4}$$

where R is the hydraulic radius (in m unit); s is sine of the slope (in m/m unit); and C is a constant determined from the Kutter's formula:

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$$C = \frac{23 + \frac{0.00155}{s} + \frac{1}{n}}{1 + \frac{n}{\sqrt{R}}\left(23 + \frac{0.00155}{s}\right)} \tag{5}$$

where n is the surface roughness coefficient.

Due to the construction of the bridge piers, the naturally available linear waterway is reduced to an effective waterway at the bridge site. Thereby, the velocity of water increases in the stream because of the reduced waterway. Such higher velocity of water mandates introducing several design features in the bridge. For example, cutwater and ease-water are provided, respectively, at the upstream and downstream faces of the piers in order to streamline the flow of the water, as much as possible. The projecting shapes of the cutwater and ease-water are designed such that the pier remains unaffected from the striking water or floating bodies. Furthermore, freeboard, afflux, and scour depth are determined based on the hydrological investigation, except in the case of a high bridge where water level is not a concern. The freeboard provided in bridges maintains minimum level difference between the anticipated highest flood level (HFL) in the stream and the formation level of the bridge, i.e., bottom most part of the bridge deck. Note that the HFL is added with afflux, which is the increase in the water level upstream of the bridge site, due to obstruction in the waterway caused on account of the bridge piers. Generally, such vertical clearance above the water level or freeboard is maintained minimally at about 0.5 m. The flow of water contributes to riverbed erosion, called scouring action. While designing the foundation, protection works, and bunds for the bridges, scour depth must be determined. The velocity of the water current varies along the flow, which necessitates estimating conservative scour depth by adopting sounding procedures at different locations in the direct vicinity of the bridge site.

Upon completion of the bridge surveys, and consideration of the inputs received, geometric design of the bridge is undertaken. The width of the highway bridge design is based on the traffic studies conducted on the path and functionalities to be served by the bridge. The width of the bridge accommodates the actual carriage way depending upon the number of traffic lanes, median(s), footpath(s), bicycles path, service lines carried across, etc. In the process of this design, future projections on the traffic requirements are also made. Generally, a service life of 120 years is considered sufficient for a bridge.

The planning stage of the bridge further includes decisions on how many piers must be provided, that is, how many spans the bridge requires. If the number of piers provided is increased, the spans of the bridge are reduced; this increases the cost of constructing piers and foundation, and the cost of constructing the bridge deck reduces. On the other hand, if the number of piers provided is reduced, the required spans for the bridge increase; this increases the cost of constructing the bridge deck, and the cost of constructing the piers and foundation reduces. Thus, because the two requirements contradict each other, in order to achieve economy in bridge construction, an economical bridge span is calculated such that the cost of bridge construction is minimized. In other words, if the costs of construction for the superstructure and substructure are fairly equal, economy in bridge construction is achieved.

The alignments of the bridge in the vertical plane and horizontal planes (footprint) are major planned architectural features. Generally, long-span bridges in deep streams are provided with high vertical curve, such that underneath the intermediate span (s) ship movement can be facilitated. The vertical profile and length of the bridge, including approaches on either side, are determined based on the ruling gradient, which is the maximum allowed slope for the plying-surface. The horizontal alignment of the bridge is governed by the geological features and traffic requirements. Note that depending upon the use of the bridge as highway or railway, the vertical and horizontal alignments and slopes are required to be carefully designed.

The bridges that are curved in plan are provided with super-elevation, that is, banking of the plying-surface. Due to the centrifugal forces experienced by moving vehicles away from the center of the curvature, they have a tendency to be pushed in the outward direction from the bridge. In order to avoid this, the outer edge of the bridge is constructed at a higher elevation relative to the inner edge of the bridge, called super-elevation. The super-elevation provided for a bridge mainly depends upon the design vehicle speed and radius of curvature of the bridge in plan. If e denotes the rate of super-elevation in percent, f is the lateral friction factor (typically assumed as 0.15), V' is the velocity of the design vehicle in m/s unit, R is the radius of circular curve in m unit, and g denotes acceleration due to gravity (9.81 m/s^2) then:

$$e = \frac{V'^2}{gR} - f \tag{5}$$

Based on the locality of the bridge site, availability of the construction material, equipment, labor, and similar factors, choice on the type of bridge is made for the site under consideration. Also, the choice of the bridge material depends upon service life and intended use of the bridge. Today, architects are employed to ensure aesthetics of the bridge is appealing to the landscape and that it offers attractive features of engineering marvel.

Upon completing architectural planning, geotechnical and structural design activities begin. The foundation type of the bridge is governed by the geological features at the site, anticipated loads to be transferred from the bridge, and the economical bridge span. The geological and geotechnical features include type of founding strata, soil condition, water table, and scour depth. Moreover, feasibility of constructing a particular type of foundation, whether shallow or deep, at the site is governed by several factors, though approach to the foundation site remains one of the most crucial factor in the decision-making. If caissons/ cofferdams are required to be constructed for facilitating building the foundation, the cost of foundation construction increases. In shallow depth water, a watertight box or casing, called caisson, is used for facilitating the construction work. A cofferdam is used for deep water depths, to create watertight working space around the foundation by preventing inflow of water at the construction site from the stream. For relatively large bridges, well foundation is a suitable choice to transfer the high vertical and horizontal loads effectively to the hard bedrock. However, it requires a costly sinking operation of the heavy well foundation in the stream. On the other hand, if the bedrock is available at relatively greater depths, then pile or pile-group foundation can be chosen. The piles typically transfer the loads through skin friction and end reaction. The type of piles to used, such as cast-in situ, driven, drilled/bored piles, etc., and the number of piles to be provided underneath the piers depends upon the loads to be transferred as well as the geological and geotechnical features of the bridge site.

The class of design load used for a bridge depends upon the manner in which it is anticipated to experience the loads in its useful service life. Generally, design life of the bridge is considered to be 120 years. Codes of practice in different countries provide guidelines on the load class of the bridge. For highway bridges, impact factor is used as a multiplying coefficient to the moving live loads in order to account for additional forces induced on the bridge. Moreover, due to sudden braking and acceleration of the vehicles plying on the deck, additional forces used in the design are called braking loads. Some of the major loading considerations in the bridge design include vehicle overloads, vehicle collision/impact, earthquakes, strong winds, flood, accidental fire, temperature, blast, fatigue, etc. Some geotechnical engineering concerns in this context pertain to uneven soil settlement, foundation failure, slope instability, excessive scouring, etc.

In the past, graphical approaches and influence line diagrams were favored tools of bridge designers for structural analysis. Later, matrix-based methods became more popular for analysis of bridges to determine design forces under several types of loads and their combinations. One of the well-recognized methods of analyzing bridge deck has been grillage analogy, which is still in use in the bridge design practice. However, today finite element (FE) techniques have replaced most of the classical bridge analysis methods. The state-of-the-art FE packages not only facilitate working conveniently with several load types and their combinations but are also able to model advanced engineered materials introduced in the construction sector of-late. Further, with the development of high-strength materials and modern construction techniques, the components of bridges have become more slender, which therefore typically necessitates conducting nonlinear buckling/stability analysis. Such stability analysis incorporates geometric and material nonlinearities and is facilitated by the advanced FE software.

The design procedures of steel, RC, and PSC bridges were based on working stress method (WSM) formerly, which is also called allowable stress design (ASD). In the WSM or ASD, the loads acting on the bridge structures and strengths of the materials used are deterministic. Hence, a defined factor of safety was in practice then and used to be around 2. The safety factor was defined as the ratio of the strength or capacity of the structural member to the loads or actions imposed. However, quite rationally, the WSM or ASD approach is now replaced by the limit state method (LSM), which is also called load and resistance factor design (LRFD). In the LSM or LRFD, the loads acting on the bridge structures and strengths of the materials used are probabilistic. Hence, actual loads acting on the structural member and resistance offered by the material of the member are taken in to account, which provides a realistic and reliable factor of safety with a defined confidence level. Most of the international standards now follow the LSM or LRFD approach for the design of bridges.

Types of Bridges

Several types of bridges are planned, designed, and constructed in real-life applications depending on needs of the site/location. Apart from the routinely constructed stationary or immovable bridges, under certain circumstances movable bridges are required to serve specific intended purposes. Movable bridge is that category of bridges which can be moved horizontally or vertically from/on their spans or rotated in horizontal or vertical planes, as per the required arrangement. The following are some types of bridges, explained with their functioning.

1. Arch bridge: has a curved geometry and transfers the dead and live loads to the supports obliquely. They may be either be (a) two-hinged arch, with hinges provided only at the piers and abutments, or (b) three-hinged arch, with one hinge provided at the crown of the arch distinct from the hinges at the piers and abutments.
2. Bascule bridge: is used where the bridge span is required to be opened to allow ship traffic underneath, by rotating the bridge span in a vertical plane.
3. Bowstring bridge: consists of a curved bow or rib, which at both ends is connected with a taut horizontal tension chord or rod.
4. Cable-stayed bridge: stay-cables are used to support the deck, whereas the loads are transferred by the stay-cables through a connected pylon. Generally, a vertical column-like member pylon is made an architectural feature.
5. Cantilever bridge or balanced cantilever bridge: the spans cantilever from the pier and often the large cantilever moments are balanced by counteracting moments offered by constructing a cantilevered bridge span in the opposite direction on the same pier.
6. Culvert: are typically short-span box or pipe type concrete bridges used to cross narrow and shallow streams.
7. Draw bridge, revolving draw bridge, rolling draw bridge, or pullback bridge: can be taken from their position, turned aside in a horizontal plane, withdrawn, or retreated longitudinally back to allow ship traffic through a stream.
8. Folding bridge, or jack-knife bridge: can be folded (like collapsing doors) and opened whenever so required.
9. Foot bridge or pedestrian bridge: is exclusively built to serve only pedestrian traffic.
10. Highway bridge or wagon bridge: is built to serve highway traffic.
11. Lattice bridge: such steel bridges have riveted or bolted trusses containing several diagonally placed inclined members.
12. Leaf bridge: leaf or leaves are swayed on hinges to make an opening whenever so required.
13. Lever-draw or motor bridge: are moved by using lever system or motors.
14. Lift/vertical lift bridge, or hoist bridge: can be lifted upwards in a vertical plane on two supporting columns so that movement of ship traffic from underneath is possible.
15. Plate/girder bridge: is made of steel plate girders or lattice.
16. Pontoon bridge, boat bridge, floating bridge, or bateau bridge: float on water by means of pontoons, barges, or boats.
17. Railway bridge: is exclusively built to serve only railway traffic.
18. Skew bridge: the longitudinal axis of the deck meets the abutments obliquely in a horizontal plane.
19. Suspension bridge, stiffened suspension bridge, chain bridge, wire bridge, rope bridge, or hanging bridge: in these types of bridges, the deck is supported by suspenders made of chains, wires, strings, which are hung from cables running over two towers on either side and transfer the loads to the ground at abutment obliquely.

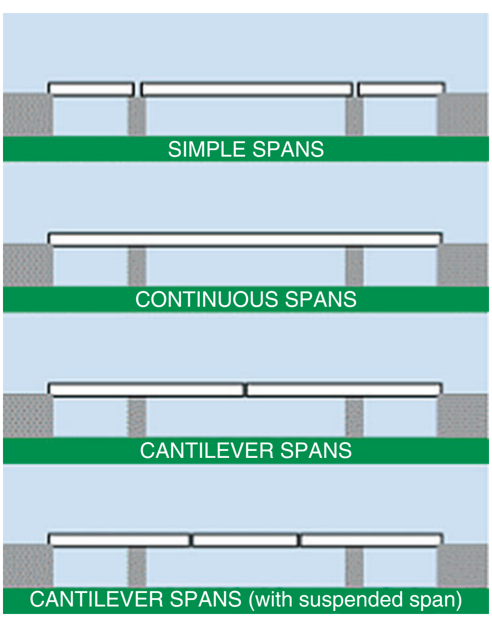
Bridge type definition depends upon the kind of cable employed. Sometimes, stiffening trusses or wires are used in these bridges.

20. Swing bridge, turning bridge, or swivel bridge: the span is able to rotate in a horizontal plane about a vertical axis to allow ship traffic.
21. Trestle bridge: metallic or wooden members form a portal-type structure above which the deck is supported.
22. Truss bridge, through-type, or deck-type: the deck is supported by two parallel trusses placed on the edges in a longitudinal direction. Depending upon location of the supported deck, either at the bottom of the truss or above the truss, they are categorized, respectively, as through-type or deck-type truss bridges.
23. Tubular/arch bridge: enclosed space is created for the users, either using large arcs of tubes or by plate girders forming a box-type of hollow conduit for passage.

The following four main components describe a bridge. By combining these items one may give a general description of most bridge types.

- Span (simple, continuous, cantilever)
- Material (stone, concrete, metal, etc.)
- Placement of the travel surface in relation to the structure (deck, pony, through)
- Form (beam, arch, truss, etc.)

The three basic types of spans are shown here (Fig. 3). Any of these spans may be constructed using beams, girders, or trusses. Arch bridges are either simple or continuous (hinged). A cantilever bridge may also include a suspended span.

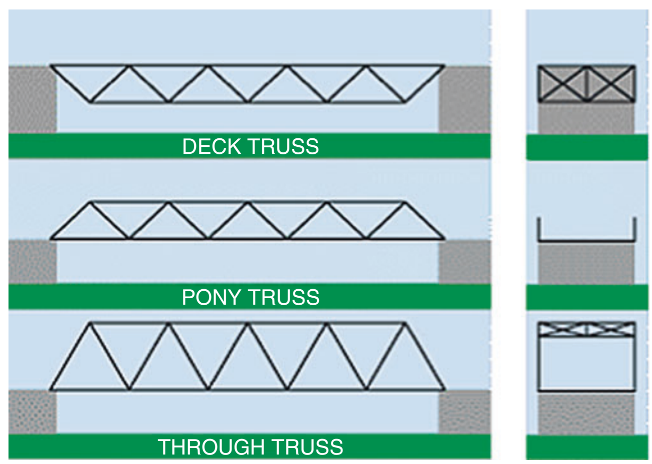


Bridges, Fig. 3 Types of bridge spans

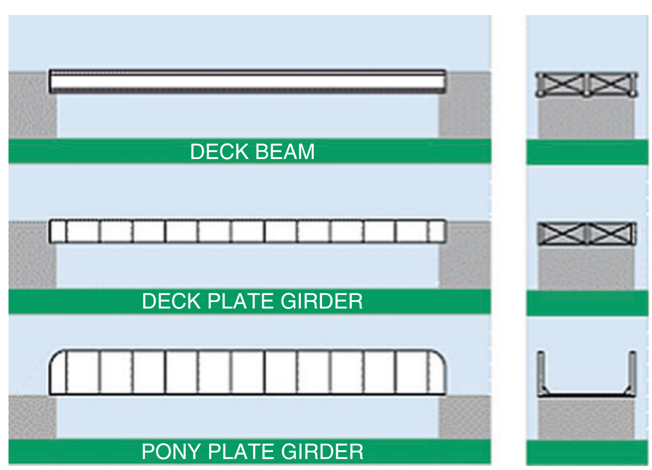
Examples of the three common travel surface configurations are illustrated in the Truss type drawings (Fig. 4). In a Deck configuration, traffic travels on top of the main structure; in a Pony configuration, traffic travels between parallel superstructures that are not cross-braced at the top; in a through configuration, traffic travels through the superstructure (usually a truss) which is cross-braced above and below the traffic.

Beam and Girder Types

Simple deck beam bridges are usually made of metal or reinforced concrete (Fig. 5). Other beam and girder types are constructed of metal. The end section of the two deck configuration shows the cross-bracing commonly used between beams. The pony end section shows knee braces that prevent deflection where the girders and deck meet.



Bridges, Fig. 4 Types of bridge travel Truss surface configurations



Bridges, Fig. 5 Types of bridge beam and girders

One method of increasing a girder's load capacity while minimizing its web depth is to add haunches at the supported ends. Usually the center section is a standard shape with parallel flanges; curved or angled flange ends are riveted or bolted using splice plates. Because of the restrictions incurred in transporting large beams to the construction site, shorter, more manageable lengths are often joined on-site using splice plates (Fig. 6).

Many modern bridges use new designs developed using computer stress analysis. The rigid frame type has superstructure and substructure that are integrated. Commonly, the legs or the intersection of a leg with the deck form a single piece which is riveted to other sections (Fig. 7).

Orthotropic beams are modular shapes that resist stress in multiple directions simultaneously. They vary in cross-section and may be open or closed shapes (Fig. 8).

Arch Types

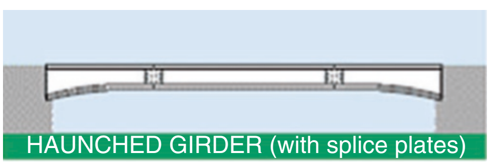
There are several ways to classify arch bridges (Fig. 9). The placement of the deck in relation to the superstructure provides the descriptive terms used in all bridges: deck, pony, and through.

However, the type of connections used at the supports and the midpoint of the arch may be used – counting the number of hinges which allow the structure to respond to varying stresses and loads. A through arch is illustrated, but this applies to all type of arch bridges.

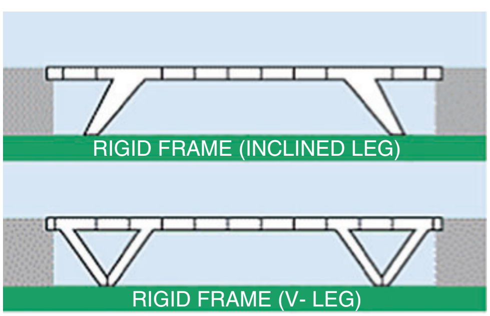
Another method of classification focuses on the configuration of the arch. Examples of solid-ribbed, brace-ribbed (trussed arch), and spandrel-braced arches are shown (Figs. 10 and 11). A solid-ribbed arch is commonly constructed using curved girder sections. A brace-ribbed arch has a curved through truss rising above the deck. A spandrel-braced arch or open spandrel deck arch carries the deck on top of the arch.

Some metal bridges which appear to be open spandrel deck arch are, in fact, cantilever; these rely on diagonal bracing. A true arch bridge relies on vertical members to transmit the load which is carried by the arch.

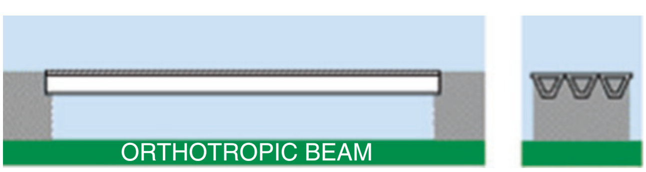
The tied arch (bowstring) type is mainly used for suspension bridges; the arch may be trussed or solid. The trusses that comprise the arch will vary in configuration, but commonly use Pratt or Warren webbing. Although a typical arch bridge passes its load to bearings at its abutment, a tied arch resists spreading (drift) at its bearings by using the deck as a tie piece (Fig. 12).



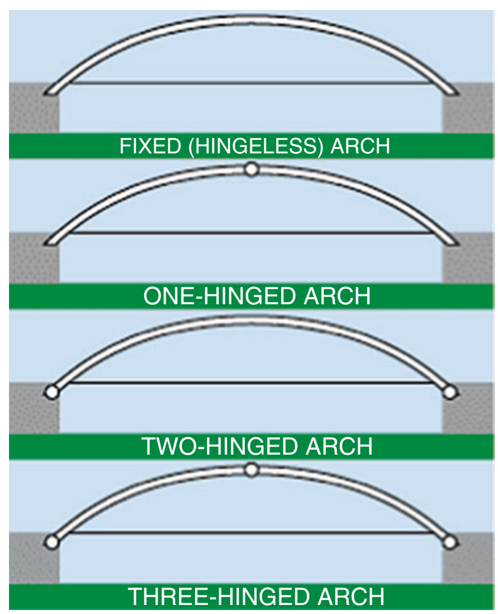
Bridges, Fig. 6 Example of a haunched girder with splice plates



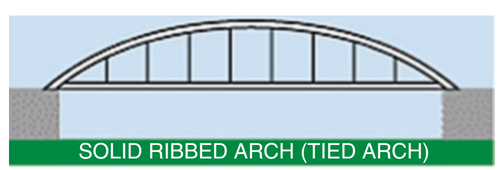
Bridges, Fig. 7 Bridge leg configuration



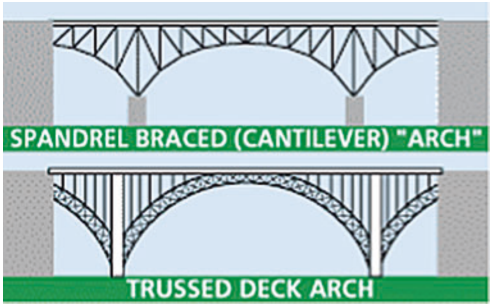
Bridges, Fig. 8 Orthotropic bridge beam



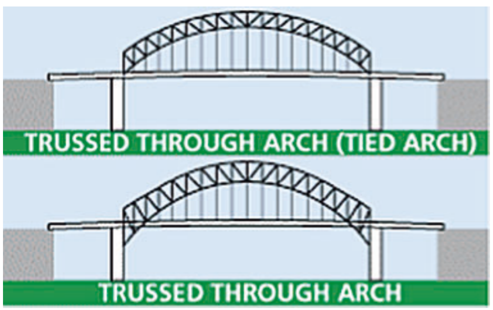
Bridges, Fig. 9 Hinge arch types for bridge classification



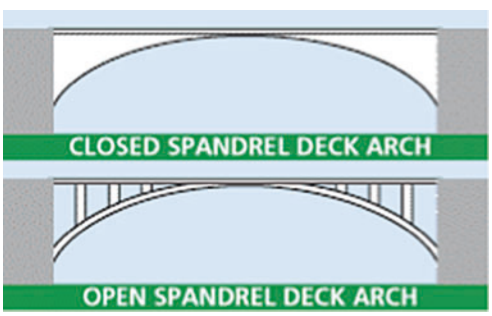
Bridges, Fig. 10 Solid ribbed arch configuration



Bridges, Fig. 11 Spandrel braced and trussed deck arch configuration



Bridges, Fig. 12 Trussed through arch (tied and untied)

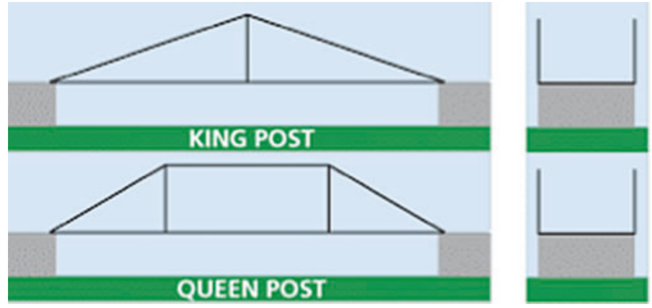


Bridges, Fig. 13 Closed and open spandrel deck arch

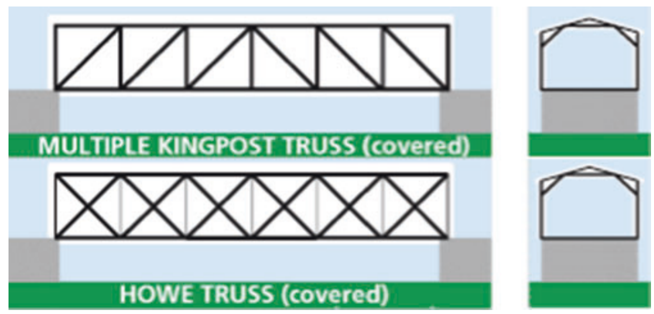
Masonry bridges, constructed in stone and concrete, may have open or closed spandrels (Fig. 13). A closed spandrel is often filled with rubble and faced with dressed stone or concrete. Occasionally, reinforced concrete is used in building pony arch types.

Truss- Simple Types

A truss is a structure constructed of many smaller parts. Once constructed of wooden timbers, and later including iron tension members, most truss bridges are now built of metal. Types of truss bridges are also identified by the terms deck, pony, and through which describe the placement of the travel surface in relation to the superstructure (see drawings above). The king post truss is the simplest type; the queen posttruss adds a horizontal top chord to achieve a longer span, but the center panel tends to be less rigid due to its lack of diagonal bracing (Fig. 14).



Bridges, Fig. 14 King and Queen post trusses



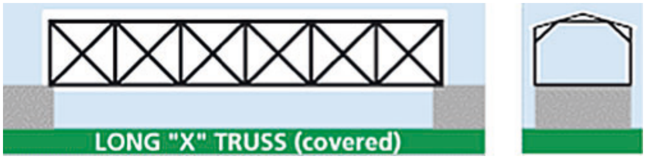
Bridges, Fig. 15 Multiple Kingpost truss and Howe truss

Covered Bridge Types (Truss)

Covered bridges are typically wooden truss structures. The enclosing roof protected the timbers from weathering and extended the life of the bridge. One of the more common methods used for achieving longer spans was the multiple kingpost truss. A simple, wooden, kingpost truss forms the center and panels are added symmetrically. With the use of iron in bridge construction, the Howe truss – in its simplest form – is a type of multiple kingpost truss (Fig. 15).

Stephen H. Long (1784–1864) was one of the US Army Topographical Engineers sent to explore and map the United States as it expanded westward. While working for the Baltimore and Ohio Railroad, he developed the X truss in 1830 with further improvements patented in 1835 and 1837 (Fig. 16). The wooden truss is also known as the Long truss, and he is cited as the first American to use mathematical calculations in truss design.

Theodore Burr built a bridge spanning the Hudson River at Waterford, NY, in 1804. By adding an arch segment to a multiple kingpost truss, the Burr arch truss was able to attain longer spans. His truss design, patented in 1817, is not a true arch as it relies on the interaction of the arch segments with the truss members to carry the load. There are many examples of this type in the Pittsburgh area, and they continue to be one



Bridges, Fig. 16 Long “X” truss

Bridges, Fig. 17 Covered Burr arch truss

Bridges, Fig. 18 Covered Town lattice truss construction

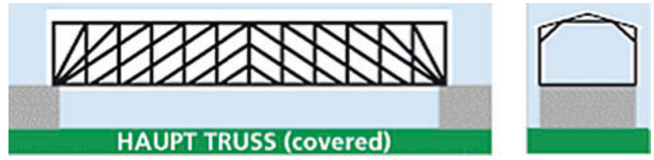
of the most common types of covered bridges. Many later covered bridge truss types used an added arch based on the success of the Burr truss (Fig. 17).

The Town lattice truss patented in 1820 by Ithiel Town is constructed of planks rather than the heavy timbers required in Kingpost and Queenpost designs. It was easy to construct, but tedious. Reportedly, Mr. Town licensed his design at one dollar per foot – or two dollars per foot for those found not under license. The second Ft. Wayne railroad bridge over the Allegheny River was an unusual instance of a Town lattice constructed in iron (Fig. 18).

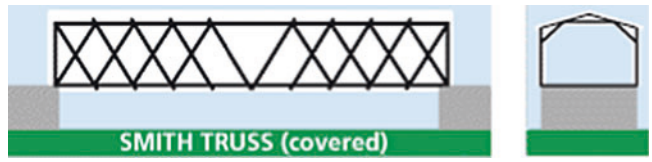
Herman Haupt designed and patented his truss configuration in 1839. He was in engineering management for several railroads including the Pennsylvania Railroad (1848) and drafted as superintendent of military railroads for the Union Army during the Civil War. The Haupt truss concentrates much of its compressive forces through the end panels and onto the abutments (Fig. 19).

Other bridge designers were busy in the Midwest. An Ohio DOT web page cites examples of designs used for some covered bridges in that state. Robert W. Smith of Tipp City, OH, received patents in 1867 and 1869 for his designs. Three variations of the Smith truss are still standing in Ohio covered bridges (Fig. 20).

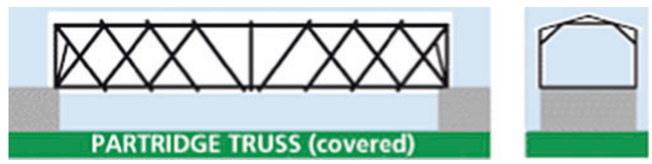
Reuben L. Partridge received a patent for his truss design that appears to be a modification of the Smith truss (Fig. 21). Four of the five Partridge truss bridges near his home in Marysville, Union County, OH, are still in use.



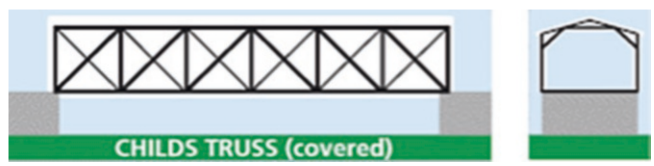
Bridges, Fig. 19 Covered Haupt Truss



Bridges, Fig. 20 Covered Smith truss



Bridges, Fig. 21 Covered Partridge truss



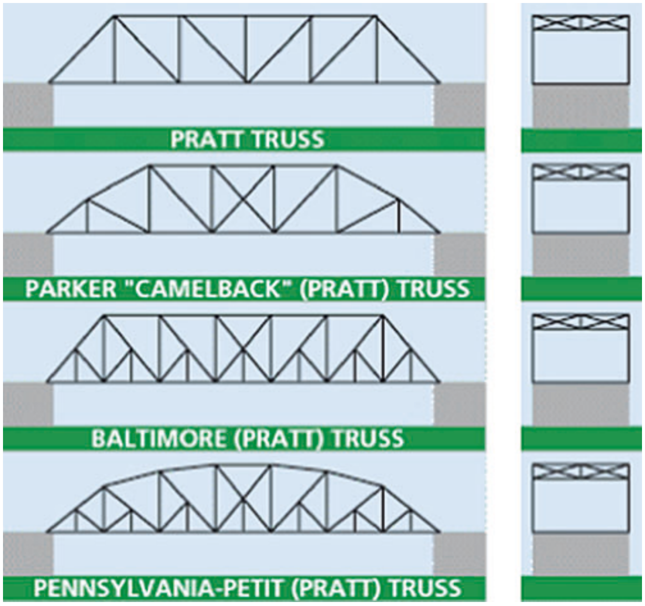
Bridges, Fig. 22 Covered Childs truss

Horace Childs’ design of 1846 was a multiple kingpost with the addition of iron rods (Fig. 22). The Childs’ truss was used exclusively by Ohio bridge builder Everett Sherman after 1883.

Pratt Truss Variations

The Pratt truss is a very common type, but has many variations. Originally designed by Thomas and Caleb Pratt in 1844, the Pratt truss successfully made the transition from wood designs to metal. The basic identifying features are the diagonal web members that form a V-shape. The center section commonly has crossing diagonal members. Additional counter braces may be used and can make identification more difficult, but the Pratt and its variations are the most common type of all trusses.

Charles H. Parker modified the Pratt truss to create a “camelback” truss having a top chord that does not stay parallel with the bottom chord. This creates a lighter structure without losing strength; there is less dead load at the ends and more strength concentrated in the center. It is somewhat more complicated to build since the web members vary in length from one panel to the next.



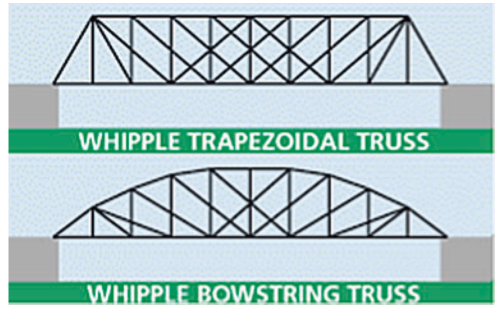
Bridges, Fig. 23 Types of Pratt truss variations

When additional smaller members are added to a Pratt truss, the various subdivided types have been given names from the railroad companies that most commonly used each type, although both were developed by engineers of the Pennsylvania Railroad in the 1870s (Fig. 23).

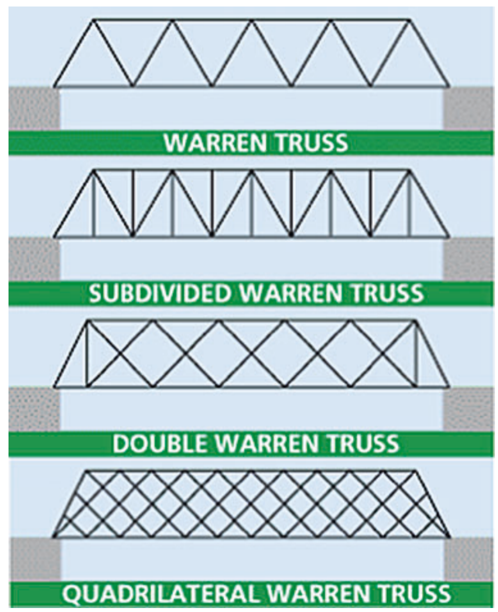
The Whipple truss was developed by Squire Whipple as stronger version of the Pratt truss. Patented in 1847, it is also known as the "Double-intersection Pratt" because the diagonal tension members cross two panels, whereas those on the Pratt cross one. The Indiana Historical Bureau notes one bridge as being a "Triple Whipple" – possibly the only one – built with the thought that if two are better than one, three must be stronger yet. The Whipple truss was most commonly used in the trapezoidal form – straight top and bottom chords – although bowstring Whipple trusses were also built. The Whipple truss gained immediate popularity with the railroads as it was stronger and more rigid than the Pratt. It was less common for highway use, but a few wrought iron examples survive. They were usually built where the span required was longer than was practical with a Pratt truss (Fig. 24). Further developments of the subdivided variations of the Pratt, including the Pennsylvania and Baltimore trusses, led to the decline of the Whipple truss.

Warren Truss Variations

A Warren truss, patented by James Warren and Willoughby Monzoni of Great Britain in 1848, can be identified by the presence of many equilateral or isosceles triangles formed by the web members that connect the top and bottom chords. These triangles may be further subdivided (Fig. 25). Warren truss designs may also be found in covered bridge designs.



Bridges, Fig. 24 Whipple truss variations

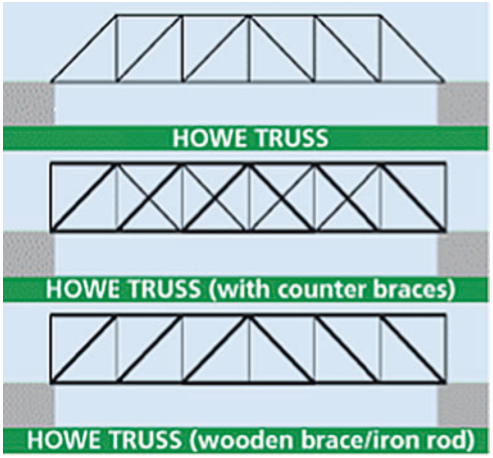


Bridges, Fig. 25 Warren truss variations

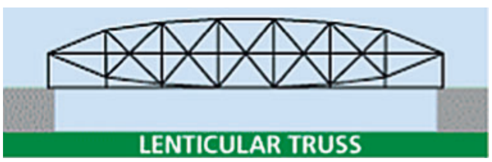
Truss: Other Types

The other truss types discussed are less common on modern bridges. A Howe truss at first appears similar to a Pratt truss, but the Howe diagonal web members are inclined toward the center of the span to form A-shapes. The vertical members are in tension, whereas the diagonal members are in compression, exactly opposite the structure of a Pratt truss. Patented in 1840 by William Howe, this design was common on early railroads. The three drawings show various levels of detail (Fig. 26). The thicker lines represent wood braces; the thinner lines are iron tension rods. The Howe truss was patented as an improvement to the Long truss that is discussed with covered bridge types.

Friedrich August von Pauli (1802–1883) published details of his truss design in 1865. Probably the most famous example of the Pauli truss, better known as the lenticular truss – named because of the lens shape – is Pittsburgh's Smithfield Street Bridge (Fig. 27). Its opposing arches combine the benefits of a suspension bridge with those of an



Bridges, Fig. 26 Howe truss details



Bridges, Fig. 27 Lenticular truss details

arch bridge. But like a willow tree, some of its strength is expressed in its flexibility which is often noticeable to bridge traffic.

Before the use of computers, the interaction of forces on spans that crossed multiple supports was difficult to calculate. One solution to the problem was developed by E. M. Wichert of Pittsburgh, PA, in 1930. By introducing an open, hinged quadrilateral over the intermediate piers, force interaction for each span could be calculated independently. The first Wichert truss was the Homestead High Level Bridge over the Monongahela River in 1937 (Fig. 28).

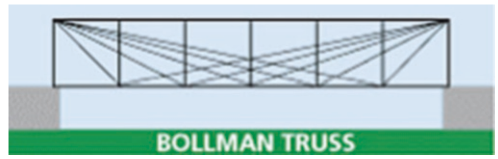
The composite cast and wrought iron Bollman truss was common on the Baltimore and Ohio Railroad. Of the hundred or so following Wendell Bollman's design, the 1869 bridge at Savage, MD, is perhaps the only intact survivor. Some of the counter bracing inside the panels is absent in the drawing for clarity (Fig. 29).

Also somewhat common on early railroads, particularly the B&O, was the Fink truss designed by Albert Fink of Germany in the 1860s (Fig. 30).

Cantilever Truss Types

A cantilever is a structural member that projects beyond its support and is supported at only one end. Cantilever bridges are constructed using trusses, beams, or girders. Employing the cantilever principles allows structures to achieve spans longer than simple spans of the same superstructure type (Fig. 31). They may also include a suspended span that hangs between the ends of opposing cantilever arms. Some bridges that appear to be arch type are, in fact, cantilever truss. These may be identified by the diagonal braces that are used in the open spandrel. A true arch bridge relies on vertical members to transfer the load to the arch. Pratt and Warren bracing are among the most commonly used truss types.

Bridges, Fig. 28 Wichert truss details



Bridges, Fig. 29 Bollman truss details

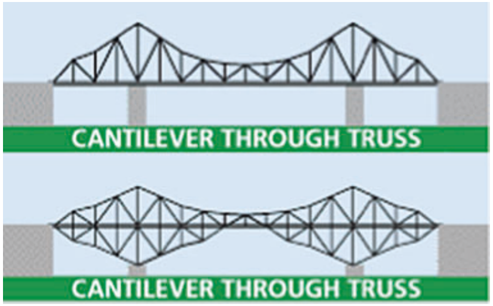
Bridges, Fig. 30 Fink truss details

Bridges, Fig. 31 Spandrel braced cantilever arch

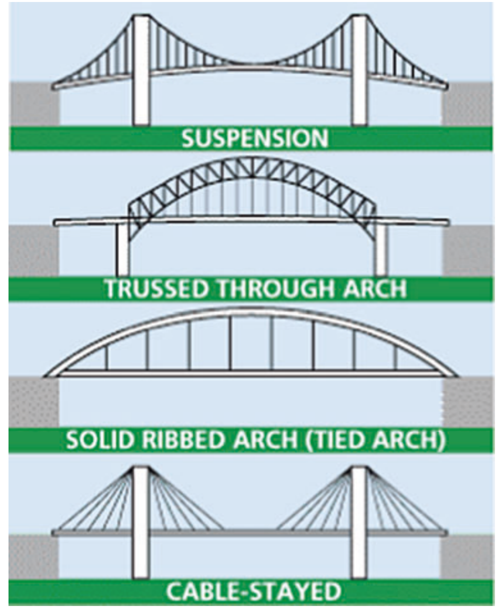
The classic cantilever design is the through truss that extends above the deck. Some have trusses that extend both above and below the deck. The truss configuration will vary (Fig. 32).

Suspension Types

The longest bridges in the world are suspension bridges or their cousins, the cable-stayed bridge (Fig. 33). The deck hangs from suspenders of wire rope, eye-bars, or other materials. Materials for the other parts also vary: piers may be steel



Bridges, Fig. 32 Cantilever through truss variations



Bridges, Fig. 33 Types of suspension bridges

or masonry; the deck may be made of girders or trussed. A tied arch resists spreading (drift) at its bearings by using the deck as a tie piece.

Though the city of Pittsburgh has been a pioneer in bridge design and fabrication, it has had few suspension bridges. The Pennsylvania Mainline Canal entered the city on John Roebling's first wire-rope suspension bridge in 1845 (replacing a failing 1829 wooden structure). A similar structure still stands at Minnisink Ford, NY, crossing the Delaware River. Roebling and his son Washington Roebling, later famous in building the Brooklyn Bridge, began their work in Saxonburg, PA, north of Pittsburgh.

New Developments in Bridge Engineering

The latest advancements in bridge engineering are coming on several fronts, which include rapid and robust construction/deployment techniques, new innovative materials, analysis and design procedures, etc. In the past, bridge bearings used to be the inevitable component of bridges. However, of-late integral bridges have been introduced, in which the conventional superstructure and substructure are integral with each other. Extra-dosed bridges are yet another new approach of bridge design evolving from the cable-stayed bridge and cantilever-girder bridge, wherein the pylon heights are lowered significantly and the stay-cables are provided more parallel to the bridge girder.

Novel materials have been introduced in bridge constructions, which are lighter but relatively stronger. For example, the steel reinforcements are replaced with the fiber-reinforced polymer (FRP) bars. In addition to their high strength to weight ratio, the FRP reinforcements also offer advantages such as noncorrosiveness and durability, low thermal conductivity, nonconduction to electricity, and nonmagnetic. In cold countries, especially, where extensive de-icing salts are used, the steel employed in the bridge construction is highly susceptible to corrosion. In such situations, the noncorrosive FRP reinforcements offer an attractive alternative to the bridge designers. Wide varieties of the FRP reinforcements are now available, made from fibers of glass, carbon, basalt, etc. For instance, carbon fiber reinforced polymer (CFRP) PSC bridges have been successfully constructed in the USA (Grace et al. 2010a, b, 2013a, b). However, it is argued that the CFRP reinforcements are relatively much expensive, which considerably increase the cost of the bridge construction. Nevertheless, it has been plausibly established that the CFRP-PSC bridges are cost-effective over their life-cycle (Grace et al. 2010c). The FRPs are also available in different forms such as rods, cables/strands, fabric, laminates. Some of these FRP materials are used in retrofitting and rehabilitation of the bridges.

Resiliency of bridges has become an important concern, especially in earthquake-prone areas. It defines how quickly a seismically damaged bridge can be restored to its functional use. Some advanced dynamic response control devices, such as base isolation and tuned mass dampers, have been proposed to effectively limit the forces induced in bridges because of earthquakes and strong winds or enhance their seismic performance (Matsagar and Jangid 2006; Matin et al. 2017; Elias and Matsagar 2017). Furthermore, in those areas that experience more than one hazard, the bridges are analyzed for multihazard effects. For example, during flood, the bed erosion may cause increase in the effective length of the bridge piers. Thus, the load-carrying capacity of the bridge reduces significantly, refer to Eq. 2. When such a bridge is subjected to earthquake ground motion, it is seismically more vulnerable. Therefore, multihazard resiliency is routinely investigated for the bridges. Real-time remote/automated health monitoring of bridges; system identification and nondestructive testing; rapid inspection, assessment, and maintenance methods; and improved safety, risk, and resilience quantification of bridges and their networks have been the latest topics of greater research interest and field implementation in bridge engineering.

Summary

Beginning from the inception of bridge engineering, the essential parts of bridges and their components have been identified and discussed. Crucial stages in bridge planning, analysis, and design were explained from the perspective of engineering geology. The usefulness of engineering geology during the planning, analysis, and design stages of a bridge is evident at all stages. Finally, the latest advancements and technological developments in bridge engineering have been summarized.

Cross-References

- ► Casing
- ► Cofferdam

Appendix

Abutment	Part of a structure that supports the end of a span or accepts the thrust of an arch; often supports and retains the approach embankment.
Anchor span	Located at the outermost end, it counterbalances the arm of span extending in the opposite direction from a major point of support. Often attached to an abutment.
Anchorage	Located at the outermost ends, the part of a suspension bridge to which the cables are attached. Similar in location to an abutment of a beam bridge.
Aqueduct	A pipe or channel, open or enclosed, which carries water. May also be part of a canal to carry boats. Sometimes carried by a bridge.
Arch	A curved structure that supports a vertical load mainly by axial compression.
Arch barrel	The inner surface of an arch extending the full width of the structure.
Arch ring	An outer course of stone forming the arch. Made of a series of voussoirs. An archivolt is an arch ring with decorating moldings.
Balustrade	A decorative railing, especially one constructed of concrete or stone, including the top and bottom rail and the vertical supports called balusters. May also include larger vertical supports called stanchions.
Baltimore truss	A subdivided Pratt truss commonly constructed for the Baltimore and Ohio Railroad. It has angled end posts and a top chord that is straight and horizontal. Compare to camelback truss and Pennsylvania truss.
Bascule Bridge	From the French word for "see-saw," a bascule bridge features a movable span (leaf) which rotates on a horizontal hinged axis (trunnion) to raise one end vertically. A large counterweight offsets the weight of the raised leaf. May have a single raising leaf or two that meet in the center when closed. Compare to swing bridge and vertical lift bridge.
Beam	A horizontal structure member supporting vertical loads by resisting bending. A girder is a larger beam, especially when made of multiple plates. Deeper, longer members are created by using trusses.
Bearing	A device at the ends of beams placed on top of a pier or abutment. The ends of the beam rest on the bearing.
Bent	Part of a bridge substructure. A rigid frame commonly made of reinforced concrete or steel that supports a vertical load and is placed transverse to the length of a structure. Bents are commonly used to support beams and girders. An end bent is the supporting frame forming part of an abutment. Each vertical member of a bent may be called a column, pier, or pile. The horizontal member resting on top of the columns is a bent cap. The columns stand on top of some type of foundation or footer that is usually hidden below grade. A bent commonly has at least two or more vertical supports. Another term used to describe a bent is capped pile pier. A support having a single column with bent cap is sometimes called a "hammerhead" pier.
Bowstring truss	A truss having a curved top chord and straight bottom chord meeting at each end.
Box girder	A steel beam built-up from many shapes to form a hollow cross-section.
Brace-ribbed arch (trussed arch)	An arch with parallel chords connected by open webbing.
Buttress	A wall projecting perpendicularly from another wall that prevents its outward

	movement. Usually wider at its base and tapering toward the top.
Cable	Part of a suspension bridge extending from an anchorage over the tops of the towers and down to the opposite anchorage. Suspenders or hangers attach along its length to support the deck.
Cable-stayed bridge	A variation of suspension bridge in which the tension members extend from one or more towers at varying angles to carry the deck. Allowing much more freedom in design form, this type does not use cables draped over towers, nor the anchorages at each end, as in a traditional suspension bridge.
Camber	A positive, upward curve built into a beam that compensates for some of the vertical load and anticipated deflection.
Camelback truss	A truss having a curved top chord and straight bottom chord meeting at each end, especially when there are more than one used end to end. Compare to Baltimore truss and Pennsylvania truss.
Cantilever	A structural member that projects beyond a supporting column or wall and is counterbalanced and/or supported at only one end.
Castellated girder	A steel beam fabricated with a zig-zag cut along its web, and welding the two sides together at their peaks. This creates a beam that has increased depth and therefore greater strength, but not increased weight.
Catenary	Curve formed by a rope or chain hanging freely between two supports. The curved cables or chains used to support suspension bridges are called catenaries.
Centering	Temporary structure or false-work supporting an arch during construction.
Chord	Either of the two principal members of a truss extending from end to end, connected by web members.
Column	A vertical structural member used to support compressive loads. See also pier and pile.
Continuous span	A superstructure that extends as one piece over multiple supports.
Corbelled arch	Masonry built over an opening by progressively overlapping the courses from each side until they meet at the top center. Not a true arch as the structure relies on strictly vertical compression, not axial compression.
Counter	A truss web member that functions only when a structure is partially loaded.
Cradle	Part of a suspension bridge that carries the cable over the top of the tower.
Cripple	A structural member that does not extend the full height of others around it and does not carry as much load.
Crown	On road surfaces, where the center is the highest point and the surface slopes downward in opposite directions, assisting in drainage. Also a point at the top of an arch.
Culvert	A drain, pipe, or channel that allows water to pass under a road, railroad, or embankment.
Deck	The top surface of a bridge that carries the traffic.
Deck truss	A truss that carries its deck on its top chord. Compare to pony truss and through truss.
Elliptical arch	An arch formed by multiple arcs each drawn from its own center. Compare to a roman arch that is a semi-circular arc drawn from a single center-point.
Embankment	Angled grading of the ground.
End post	The outward-most vertical or angled compression member of a truss.
Expansion joint	A meeting point between two parts of a structure which is designed to allow for movement of the parts due to thermal or moisture factors while protecting the parts from damage. Commonly visible on a bridge deck as a hinged or movable connection.
Extrados	The outer exposed curve of an arch; defines the lower arc of a spandrel.
Eye bar	A structural member having a long body and an enlarged head at each end. Each head has a hole through which an inserted pin connects to other members.
False-work	Temporary structure used as support during construction. False-work for arch construction is called "centering."
Fill	Earth, stone, or other material used to raise the ground level, form an embankment, or fill the inside of an abutment, pier, or closed spandrel.
Finial	A sculpted decorative element placed at the top of a spire or highpoint of a structure.
Fixed arch	A structure anchored in its position. Compare to hinged arch.
Floor beam	Horizontal members that are placed transversely to the major beams, girders, or trusses, used to support the deck.

Footing	The enlarged lower portion of the substructure or foundation that rests directly on the soil, bedrock, or piles; usually below grade and not visible.
Gabion	A galvanized wire box filled with stones used to form an abutment or retaining wall.
Girder	A horizontal structure member supporting vertical loads by resisting bending. A girder is a larger beam, especially when made of multiple metal plates. The plates are usually riveted or welded together.
Gusset plate	A metal plate used to unite multiple structural members of a truss.
Haunch	The enlarged part of a beam near its supported ends that results in increased strength; visible as the curved or angled bottom edge of a beam.
Hinged arch	A two-hinged arch is supported by a pinned connection at each end. A three-hinged arch also includes a third pinned connection at the crown of the arch near the middle of a span. Compare to fixed arch.
Howe truss	A type of truss in which vertical web members are in tension and diagonal web members in compression. Maybe recognized by diagonal members that appear to form an "A" shape (without the crossbar) toward the center of the truss when viewed in profile. Compare to Pratt truss and Warren truss.
Humpback	A description of the side-view of a bridge having relatively steep approach embankments leading to the bridge deck.
Impost	The surface that receives the vertical weight at the bottom of an arch.
Intrados	The interior arc of an arch.
Jersey barrier	A low, reinforced concrete wall wider at the base, tapering vertically to near mid-height and then continuing straight up to its top. The shape design directs automotive traffic back toward its own lane of travel and prevents crossing of a median or leaving the roadway. Commonly used on new and reconstructed bridges in place of decorative balustrades, railings, or parapets.
Keystone	The uppermost wedge-shaped voussoir at the crown of an arch that locks the other voussoirs into place.
King Truss	Two triangular shapes sharing a common center vertical member (king post); the simplest triangular truss system. Compare to queen truss.
Knee brace	Additional support connecting the deck with the main beam that keeps the beam from buckling outward. Commonly made from plates and angles.
Lag	Crosspieces used to connect the ribs in centering.
Lateral bracing	Members used to stabilize a structure by introducing diagonal connections.
Lattice	An assembly of smaller pieces arranged in a grid-like pattern; sometimes used a decorative element or to form a truss of primarily diagonal members.
Lenticular truss	A truss that uses curved top and bottom chords placed opposite one another to form a lens shape. The chords are connected by additional truss web members.
Member	One of many parts of a structure, especially one of the parts of a truss.
Parabola	A form of arch defined by a moving point that remains equidistant from a fixed point inside the arch and a moving point along a line. This shape when inverted into an arch structure results in a form that allows equal vertical loading along its length.
Parapet	A low wall along the outside edge of a bridge deck used to protect vehicles and pedestrians.
Pennsylvania truss	A subdivided Pratt truss invented for use by the Pennsylvania Railroad. The Pennsylvania truss is similar in bracing to a Baltimore truss, but the former has a camelback profile, whereas the latter has angled end posts only, leaving the upper chord straight and horizontal. Compare to camelback truss and Baltimore truss.
Pier	A vertical structure that supports the ends of a multispan superstructure at a location between abutments. See also column and pile.
Pile	A long column driven deep into the ground to form part of a foundation or substructure. See also column and pier.
Pin	A cylindrical bar used to connect various members of a truss; such as those inserted through the holes of a meeting pair of eye-bars.
Pony truss	A truss that carries its traffic near its top chord but not low enough to allow cross-bracing between the parallel top chords. Compare to deck truss and through truss.
Portal	The opening at the ends of a through truss that forms the entrance. Also the open entrance of a tunnel.

Post One of the vertical compression members of a truss that is perpendicular to the bottom chord.

Pratt truss A type of truss in which vertical web members are in compression and diagonal web members in tension. Many possible configurations include pitched, flat, or camelback top chords. Maybe be recognized by diagonal members which appear to form a “V” shape toward the center of the truss when viewed in profile. Variations include the Baltimore truss and Pennsylvania truss. Compare to Warren truss and Howe truss.

Pylon A monumental vertical structure marking the entrance to a bridge or forming part of a gateway.

Queen Truss A truss having two triangular shapes spaced on either side of central apex connected by horizontal top and bottom chords. Compare to king truss.

Reinforcement Adding strength or bearing capacity to a structural member. Examples include the placing of metal rebar into forms before pouring concrete, or attaching gusset plates at the intersection of multiple members of a truss.

Revet The process of covering an embankment with stones.

Revetment A facing of masonry or stones to protect an embankment from erosion.

Rib Any one of the arched series of members that is parallel to the length of a bridge, especially those on a metal arch bridge.

Rigid Frame Bridge A type of Girder Bridge in which the piers and deck girder are fastened to form a single unit. Unlike typical girder bridges constructed so that the deck rests on bearings atop the piers, a rigid frame bridge acts as a unit. Pier design may vary.

Rise The measure of an arch from the spring line to the highest part of the intrados, that is from its base support to the crown.

Segmental arch An arch formed along an arc drawn from a point below its spring line, thus forming a less than semi-circular arch. The intrados of a Roman arch follows an arc drawn from a point on its spring line, thus forming a semi-circle.

Simple span A span in which the effective length is the same as the length of the spanning structure. The spanning superstructure extends from one vertical support, abutment, or pier, to another, without crossing over an intermediate support or creating a cantilever.

Skew When the superstructure is not perpendicular to the substructure, a skew angle results. The skew angle is the acute angle between the alignment of the superstructure and the alignment of the substructure.

Span The horizontal space between two supports of a structure. Also refers to the structure itself. The clear span is the space between the inside surfaces of piers or other vertical supports. The effective span is the distance between the centers of two supports.

Spandrel The roughly triangular area above an arch and below a horizontal bridge deck. A closed spandrel encloses fill material. An open spandrel carries its load using interior walls or columns.

Splice plate A plate that joins two girders. Commonly riveted or bolted.

Springer The first voussoir resting on the impost of an arch.

Spring line The place where an arch rises from its support; a line drawn from the impost.

Stanchion One of the larger vertical posts supporting a railing. Smaller, closely spaced vertical supports are balusters. See also balustrade.

Stiffener On plate girders, structural steel shapes, such as an angle, are attached to the web to add intermediate strength.

Stringer A beam aligned with the length of a span that supports the deck.

Strut A compression member.

Substructure The portion of a bridge structure including abutments and piers that supports the superstructure.

Superstructure The portion of a bridge structure which carries the traffic load and passes that load to the substructure.

Suspended span A simple beam supported by cantilevers of adjacent spans, commonly connected by pins.

Suspenders Tension members of a suspension bridge that hang from the main cable to support the deck. Also similar tension members of an arch bridge which features a suspended deck. Also called hangers.

Suspension bridge A bridge that carries its deck with many tension members attached to cables draped over tower piers.

Swing Bridge — A movable deck bridge which opens by rotating horizontally on an axis. Compare to Bascule Bridge and vertical lift bridge.

Through truss — A truss which carries its traffic through the interior of the structure with cross-bracing between the parallel top and bottom chords. Compare to deck truss and pony truss.

Tie — A tension member of a truss.

Tied arch — An arch that has a tension member across its base that connects one end to the other.

Tower — A tall pier or frame supporting the cable of a suspension bridge.

Trestle — While Bridge is the more general term (which may be a single span or multi-span, typically one span is longer than the others). Trestle is a longer, multi-span structure - a series of shorter spans in which most of the spans are of similar length. Trestle is a more common term in relation to railroads, whereas viaduct is a similar long, multi-span structure for streets. Neither term seems to be exclusive. Although described as a single structure, the Ohio Connecting RR Bridge over the Ohio River at Brunot Island can be described as a pair of bridges (one over each river channel) with a trestle at each approach and a trestle connection in the center. But more often, a long structure which does not have a predominantly larger span could be described as a trestle.

Truss — A structural form which is used in the same way as a beam, but because it is made of a web-like assembly of smaller members it can be made longer, deeper, and therefore, stronger than a beam or girder while being lighter than a beam of similar dimensions.

Trussed arch — A metal arch bridge that features a curved truss.

Upper chord — Top chord of a truss.

Vault — An enclosing structure formed by building a series of adjacent arches.

Vertical Lift Bridge — A movable deck bridge in which the deck is raised vertically by synchronized machinery at each end. Compare to swing bridge and vertical lift bridge.

Viaduct — A long, multispan structure, especially one constructed of concrete. More commonly used in relation to structures carrying motor vehicles. Trestle is the term for a similar structure when used in relation to railroads.

Voussoir — Any one of the wedge shaped block used to form an arch.

Warren truss — A type of truss in which vertical web members are inclined to form equilateral triangles. May be recognized by diagonal members that appear to form a series of alternating “V” and “A” shapes (without the crossbar) along the length of the truss when viewed in profile. Often the triangles are bisected by vertical members to reduce the length of the members of the top chord. Compare to Pratt truss and Howe truss.

Web — The system of members connecting the top and bottom chords of a truss. Also the vertical portion of an I-beam or girder.

Wing walls — Extensions of a retaining wall as part of an abutment; used to contain the fill of an approach embankment

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Brownfield Sites

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Synonyms

Contaminated land – sometimes used as a synonym although land that has not been used previously can also become contaminated through water or air pollution; Derelict land; Vacant land

Definition

Previously developed land that has become disused (as opposed to "greenfield sites" – those that have not had previous uses other than agriculture, forestry, or no perceptible human interventions).

Introduction

Brownfield sites are mainly located in urban areas where industrial or commercial buildings and facilities have become disused. But some sites, such as military installations, mines, and waste management sites, may be located in open countryside or wilderness areas. Individual sites may cover many hectares or be small and scattered. Some can be easily redeveloped but many have problems of contamination/pollution, instability, and obstructions from past uses that add to difficulties and costs of either redevelopment or rehabilitation. Surficial deposits on such sites are often of anthropogenic origin.

Sites may be vacant (i.e., essentially intact but not in use) or derelict (abandoned and in poor condition) or may still be partly used (e.g., a scrap yard on a former factory site) (Fig. 1).

Brownfield Sites, Fig. 1 Abandoned industrial site, London, UK (Photograph by the author)

Contamination and Pollution

Contaminants in brownfield sites may include solids, liquids and gases, and volatile organic compounds. The nature and composition of these depends on the various human activities that have taken place. These may include hydrocarbons, solvents, pesticides, potentially harmful elements and inorganic compounds, phenols and related compounds, cyanides and asbestos, and, sometimes, radioactive materials or munitions. Some are toxic, asphyxiant, or carcinogenic. Others can react chemically with construction materials such as concrete and metal form-work.

Treatment of contamination traditionally involved excavation and removal to landfill commonly referred to as "dig and dump." While that is the least expensive option, it simply moves contaminants from one place to another. Alternatives, which are environmentally more sustainable, but are also usually more expensive, include:

- Soil washing – excavation of soils and washing them using internally recycled water to avoid additional pollution, and careful disposal of the resulting water
- *In situ* thermal desorption – to mobilize volatile and semi-volatile organic contaminants
- Bioremediation/phytoremediation – breaking down contaminants using injected or bacteria or fungi, or by using

deep-rooted plants to extract heavy metals and, when fully grown, to cut these and dispose of them to landfill
- *In situ* chemical oxidation – injection of oxidants such as sodium or potassium permanganate, ozone or Fenton's Reagent to treat or reduce the toxicity of certain organic contaminants (e.g., benzene, toluene, and chlorinated solvents)
- *In situ* soil vapor extraction – use of vacuum blowers and extraction wells to induce gas flow through the subsurface so that it can be collected and treated aboveground (Meuser 2012)

Stability and Obstructions

Previously used land sometimes contains:

- Cavities such as shallow mines, cellars, and storage tanks which may need to be excavated and/or filled and shafts and wells that need to be capped or filled to ensure stability
- Tipped materials that may be poorly consolidated and subject to settlement
- Foundations of previous structures and services (e.g., pipes and cables) that may need to be excavated and removed

Demolition

Many sites require demolition of buildings and foundations. It is desirable to, as far as possible, recycle debris (e.g., recovery of metals and soils; crushing of concrete and masonry for use as aggregates). However the components of old buildings may be difficult to separate economically and can be further contaminated by materials such as asbestos in which case they must be landfilled as hazardous waste.

Conservation

Simplistically, it might be thought that brownfield sites are environmentally undesirable and should be fully remediated to benefit the environment, wildlife, and people. That is often true, but there are important exceptions.

Disturbed ground that is left vacant for a protracted period attracts plants that are adapted for invading such settings. These form the basis of new ecosystems and can add to local biodiversity (Angold et al. 2006). Instances are known where deposits of particular chemical compositions have led to unusual plant communities on sites that were of sufficient interest for the sites to be given protected status. Therefore collaboration between engineering geologists and ecologists is important.

Some previously used land may have gone through several rounds of development and abandonment over several hundred years, the early stages of which may now be of archaeological/industrial archaeological importance (Symonds 2010). These require collaboration between engineering geologists and archaeologists.

Investigation

Commonly, the history of a site and consequent problems are not fully known therefore detailed site investigation is normally necessary to identify potential problems so that reliable cost estimates for remedial works can be made. This can be time consuming and expensive. Investigations have five main elements:

- Examination of old maps, documents, publications, and photographs to identify all previous uses of the site to establish which problems may have been inherited from these
- A "walk-over" survey and mapping of the site to observe evidence of problems and potential advantages and to provide the context for ground investigations
- Design of an investigation strategy
- Site investigation (trial pitting, drilling, geophysical survey, sampling, testing and monitoring)
- Preparation of a report setting out the steps and techniques that should be adopted during site remediation and development

Careful investigation is essential to avoid unexpected problems during site works that add to costs and delay development. Brownfield sites may present hazards to:

- Staff undertaking site investigations, remediation or works because of toxins, gases or, sometimes, explosives and even old munitions
- Local communities if works lead to new contamination or pollution of nearby soils, water, or the air

The reliability of geophysical investigations may be affected by metal pipework and cables (some of which may be electrically "live") (Nathanail and Bardos 2004).

Care is needed in site evaluations and operations to confine and minimize risks.

Affordability

Remediation of brownfield sites is usually more, and sometimes prohibitively, expensive than developing greenfield sites making these less attractive to developers. But making use of brownfield sites is often needed to improve derelict land and minimize damage to undeveloped land (de Sousa 2000).

In several countries, the amounts and types of brownfield land are monitored together with progress on redevelopment. In some cases, registers are kept that developers can use to identify sites for possible redevelopment.

Various measures have been developed to incentivize the re-use of brownfield land, for example:

- Collaborations between private companies and insurance companies to underwrite clean-up, guarantee clean-up costs, and limit the exposure of developers to environmental costs and litigation (USA).
- Site assessment using public money to provide certainty to prospective developers (USA).
- Tax incentives.
- Grants paid by Government to offset costs of remediation (UK).

There have been two main philosophies for remediation- to make the site safe for:

- All possible subsequent uses
- Some specific uses following remedial action

The former is usually much more expensive, so the latter is more often adopted.

Summary

Previously developed land often presents problems of contamination, pollution, and instability, which makes it costly to redevelop compared with greenfield sites. Even so, public policies often favor the redevelopment of brownfield land over use of new sites. This usually requires the provision of incentives to developers from public authorities. Redevelopment requires careful site investigation followed by cautious redevelopment to prevent the possibility of contamination/pollution spreading.

Cross-References

- ▶ Artificial Ground
- ▶ Contamination
- ▶ Drilling Hazards
- ▶ Foundations
- ▶ Gases
- ▶ Monitoring
- ▶ Site Investigation
- ▶ Waste Management

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Building Stone

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Synonyms

Dimension stone; Natural stone

Definition

Building stone is a generic term referring to all naturally occurring rock (natural stone as defined by BSI 2002) used in the building construction industry, including a wide variety of igneous, sedimentary, and metamorphic rocks. If after quarrying, the rock has been selected and cut to specific sizes and shapes, it is referred as dimension stone (ASTM 2016).

The availability and durability of stones has made them a major contributor to the legacy of human history. Stones were widely used as structural elements, mostly as irregularly shaped large blocks usually closely fitted (without binders), in the construction of temples, monuments, fortifications, aqueducts, bridges, and housing.

Due to the development and technological improvement of tools and machinery building stones are now quarried in large scale as regularly shaped blocks that can be cut into a wide choice of slab thicknesses and sizes (Fig. 1) and can receive several types of finishing (polished, honed, flamed, bush-hammered, and others). Reinforcement and filling may be used, depending of the rock type and characteristics

Building Stone, Fig. 1 Modern quarrying of building stones (*left*) and an illustration of slabs in different dimensions according to the final use (*right*)

Building Stone, Table 1 Some building stone application and laboratory testing requirements (After ASTM 2012, modified)

	Building stone application					
	Floors		Walls			
Laboratory testing requirements (properties)	Exterior	Interior	Exterior	Interior	Façades	Countertops
Petrography	•	•	•	•	•	•
Bulk density	•	•	•	•	•	•
Water absorption	•	•	•		•	•
Thermal dilatation	•		•		•	
Abrasion resistance	•	•				
Compressive strength			•	•	•	
Modulus of rupture	•					•
Flexural strength					•	•

(presence of pores, cavities, cracks, fissures). Stone processing frequently also includes resination that consists of the cosmetic enhancement of stone slab surface by proper resin application (epoxy, acrylic).

Current uses of natural stone in buildings include load-bearing and self-supporting masonry, masonry façades to framed buildings, cladding and lining, flooring and stone roofing, in which slates have particular importance (Ingham 2011). Another significant application is paving. As ornamental and decorative pieces, they are also extensively used in countertops and counters, sculptures, gravestones, and for landscaping.

Aesthetics, especially color, is the main attribute influencing the architectural choice of building stones. However, it is essential to consider their physical and mechanical properties (also called engineering properties), that are determined by laboratory testing, such as bulk density, water absorption, and mechanical strength, including petrographic analysis (Table 1). These allow to the selection of the rock type that is most suitable to any building design and also indicate stone performance in diverse uses and environments. Test method standardization is secured by statements issued by two important institutions: CEN and ASTM.

Building stones are also used to repair damaged and missing parts of historic buildings that have undergone deterioration by weathering or anthropogenic actions (Winkler 1997). In this case, testing and petrographic examinations are very useful to both diagnose the causes of stone deterioration and to identify the most appropriate matching stones.

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Bulk Modulus

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Definition

Bulk modulus (K) is the ratio of *hydrostatic stress* (p) on an object to the resulting *volumetric strain* (ε_v), which is the ratio of volume change (ΔV) to the initial volume (V_o).

Hydrostatic stress cannot produce *shear stress*; however, *principal stress* acting in one direction produces strain in all three directions, as described by *Hooke's law* and *Poisson's ratio* (v). Therefore,

$$\varepsilon_v = \varepsilon_x + \varepsilon_y + \varepsilon_z \tag{1}$$

$$\varepsilon_x = \frac{\sigma_x}{E} - v\frac{\sigma_y}{E} - v\frac{\sigma_z}{E} \tag{2a}$$

$$\varepsilon_y = -v\frac{\sigma_x}{E} + \frac{\sigma_y}{E} - v\frac{\sigma_z}{E} \tag{2b}$$

$$\varepsilon_z = -v\frac{\sigma_x}{E} - v\frac{\sigma_y}{E} + \frac{\sigma_z}{E} \tag{2c}$$

where E is the *Young's modulus*. Combining Eqs. 1 and 2a, b, c

$$\begin{aligned}\varepsilon_v &= \frac{1}{E}(\sigma_x + \sigma_y + \sigma_z) - \frac{2v}{E}(\sigma_x + \sigma_y + \sigma_z)\\ &= \frac{1-2v}{E}(\sigma_x + \sigma_y + \sigma_z)\end{aligned} \tag{3}$$

Hydrostatic stress is a principal stress acting equally in all directions ($p = \sigma_x = \sigma_y = \sigma_z$); therefore,

$$\varepsilon_v = \frac{1-2v}{E}3p \tag{4}$$

Thus,

$$\frac{\varepsilon_v}{p} = \frac{3(1-2v)}{E} = \frac{1}{K} \tag{5}$$

and

$$K = \frac{E}{3(1-2v)};\quad 0 < v < 0.5 \tag{6}$$

Bulk modulus can be calculated from two basic elastic properties: Young's modulus and Poisson's ratio. A singularity in K occurs at $v = 0.5$, which pertains to "incompressible" materials (Mott et al. 2008) but is not relevant in real materials of interest to engineering geologists.

Cross-References

- ▶ Hooke's Law
- ▶ Poisson's Ratio
- ▶ Strain
- ▶ Stress
- ▶ Young's Modulus

References

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C

California Bearing Ratio

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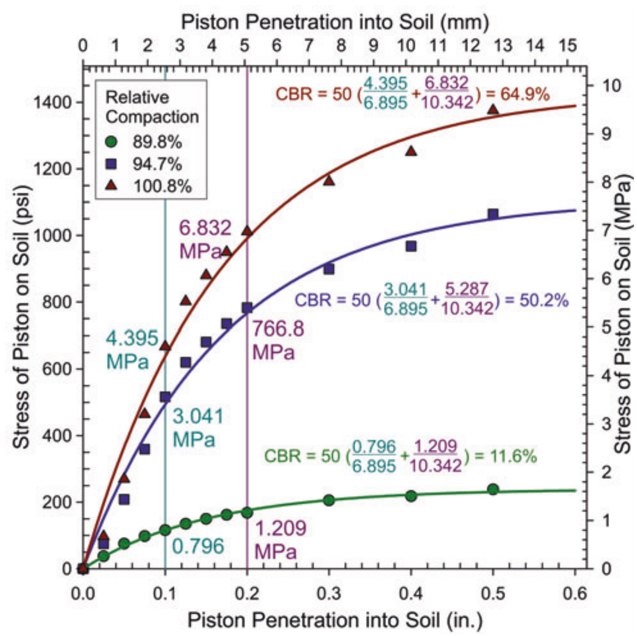
Definition

The California bearing ratio (CBR) is an index that compares penetration resistance of laboratory-compacted soil material to that of a *durable*, *well-graded* (*poorly sorted*), crushed rock material.

Context

The test was developed by the California Department of Highways in the late 1920s with the intention to characterize *cohesive soil* in the subbase and subgrade of pavement sections. It is a standard test with procedures specified by American Association of State Highway and Transportation Officials (AASHTO 2013) and American Society for Testing and Materials (ASTM 2016) in North America. The test uses a standard compaction mold with a diameter of 152.4 mm and a height of 177.8 mm. The degree of compaction and range of moisture content are specified for the test depending on project requirements. In most cases, the sample is compacted into the mold and then submerged in water for 4 days prior to testing. The sample and mold are removed from the water, a ring-shaped surcharge load is applied to the surface of the compacted soil in the mold, and a load is applied to a steel piston that has a diameter of 49.6 mm to attain a penetration rate of 1.3 mm per minute. The load at penetrations of 2.54 mm and 5.08 mm is recorded. The recorded loads are converted to stress values by dividing the load by the area of the end of the steel piston. These stress values are compared to the equivalent crushed-rock-standard stress values of 6.9 MPa for the 2.54-mm penetration and 10.3 MPa for the 5.08-mm penetration. CBR is calculated as the average of the ratio of laboratory stress to standard stress for the two penetration depths expressed as a percentage (Fig. 1) and referenced to an optimum water content and a specified dry unit weight, which usually is given as a percentage of the maximum dry unit determined by a standard compaction test.



California Bearing Ratio, Fig. 1 Plot of California bearing ratio test results for three specimens of the same silty gravel soil compacted to three relative compaction values. Data points and regression curves (two-parameter exponential rise to a maximum value) are plotted; values of stress for the index penetration depths are listed. This test is used widely in pavement design. It has limited value in engineering geology beyond enhancing the geologists' ability to understand the needs of other professionals

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Cross-References

► Compaction
► Crushed Rock
► Density
► Engineering Properties
► Mechanical Properties
► Soil Laboratory Tests
► Soil Properties

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Cambering

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Definition

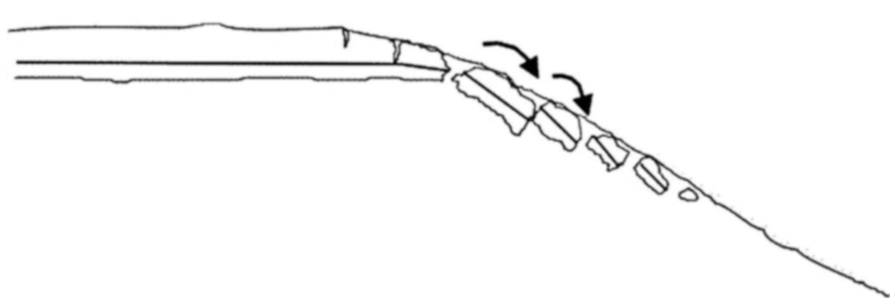
Mass movement caused by gradual lowering and thinning of underlying strata, under gravitational forces, toward an adjacent valley or slope.

Cambering occurs where competent and permeable caprock overlies incompetent beds (e.g., clay, mudstone, siltstone, and sand). Following valley incision, the incompetent material is "extruded" from beneath the caprock initially as a result of stress relief and a reduction in shear strength due to pore pressure increases associated with thawing during periglaciation. The overlying competent beds develop a local dip, or "camber," toward the valleys and, where relatively thin, sets of cross-slope subvertical parallel discontinuities may form, commonly developing into faults separating more steeply dipping blocks, referred to as "dip-and-fault" structure (Fig. 1) (Chandler et al. 1976; Hutchinson 1991). With time, this process breaks the caprock into discrete blocks "floating" in the medium of the underlying, weaker strata. Under lateral extension, the resulting inter-block discontinuities open, and these "gulls" tend to become at least partially filled with disturbed material from adjacent, underlying, and overlying strata. The gulls may or may not be marked at the surface by topographic hollows. Ultimately, the whole mass may be incorporated into landslides on the valley slope (Fig. 2) (Chandler et al. 1976; Forster et al. 1985). The example in Jurassic rocks from Bath, UK, shown in Fig. 2, is in effect a "double"-cambered feature comprising two sets of interbedded weak and strong strata.

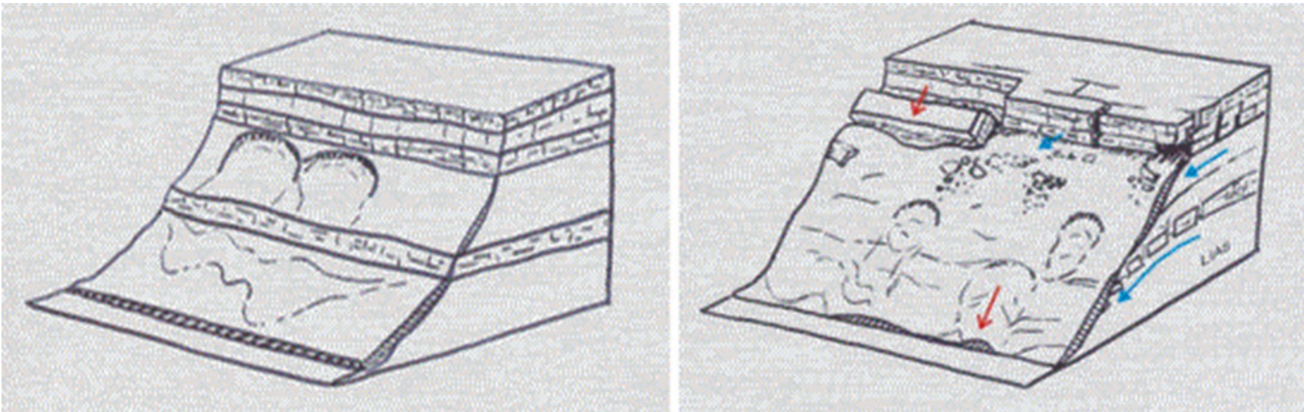
Preconditions for, and mechanisms of, cambering have been discussed (Parks 1991). Many proposed processes have been case specific and may not be universally applicable, for example, the prerequisites of a freeze/thaw component and deep and rapid valley incision. One of the best exposures of cambering was during the construction of a dam at Empingham, UK (Horswill and Horton 1976; Vaughan 1976). While cambering is included in recent classifications of landslides under the category "spreads" (Hungr 2014), such features have not been ascribed to "cambering" per se.

Cambering is often associated with "valley bulges" and "gull caves." The former represents the uplift of the valley floor due to stress relief within incompetent strata (e.g., due to rapid proglacial down-cutting) and the latter the later stages in the development of "gulls" within the caprock resulting in labyrinthine networks penetrating tens or even hundreds of meters from the valley side (Barron et al. 2016; Self and Farrant 2013).

The need for engineering geologists to recognize the presence or likelihood of cambering is paramount so that potential geohazards are not missed. Suitable 3D engineering geological models should be produced (Fookes et al. 2007; Parry et al. 2014); these will tend to be more complex than an uncambered equivalent. Rock mass characteristics of caprock may require reappraisal. Effective investigation methods include geophysical techniques, aerial LiDAR, and traditional geological mapping with augers (Barron et al. 2016).



Cambering, Fig. 1 Schematic example of "dip-and-fault" structure resulting from cambering



Cambering, Fig. 2 Schematic diagrams illustrating the development of cambering in the Jurassic strata of the Bath area; early stage (*left*), late stage (*right*) (Barron et al. 2010)

Cambering is not thought to continue at the present day in temperate regions. This might suggest that periglacial conditions are an essential prerequisite triggering process (Hutchinson 1991). The preponderance of the phenomenon in the UK may be due to the particular circumstances of preservation of periglacial features in the modern landscape of central and southern Britain.

Cross-References

- ▶ Cap Rock
- ▶ Geophysical Methods
- ▶ Geostatic Stress
- ▶ Hazard
- ▶ Landslide
- ▶ LiDAR
- ▶ Mass Movement
- ▶ Rock Mass Classification
- ▶ Shear Strength

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Cap Rock

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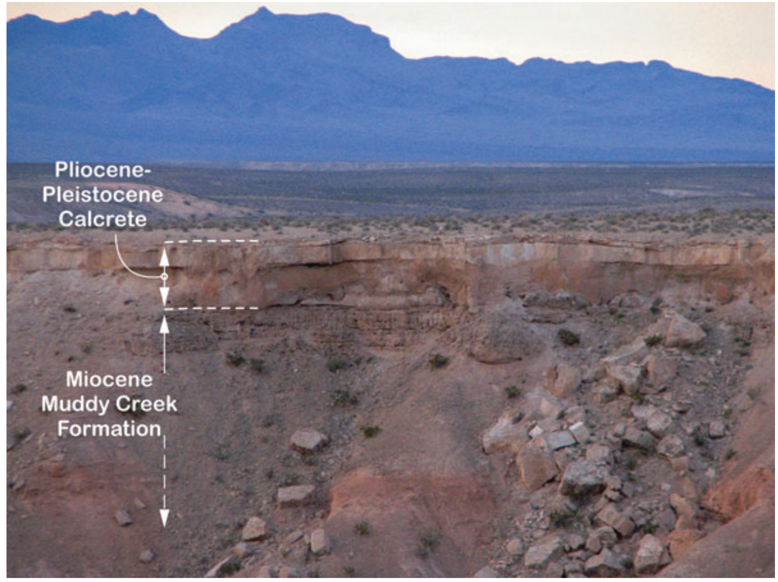
Definition

The upper rock material that is more resistant to *erosion* than the underlying rock material; it also refers to a sedimentary unit of lower *hydraulic conductivity* than that of the underlying oil or gas *reservoir rock* that restricts upward migration of hydrocarbons, thus effectively capping the reservoir.



Cap Rock, Fig. 1 Cap rock comprised of 5- to 8-m-thick indurated calcrete formed in Miocene Muddy Creek Formation approximately 100 km northeast of Las Vegas, Nevada, USA (Photo by Jeffrey R Keaton, 2 January 2007. File name: Cap rock, fig1.png)

In *geomorphology*, the upper rock material that is more resistant to erosion than the underlying rock material is called cap rock. Cap rock typically forms a distinctive ledge at the crest of an *escarpment* (Fig. 1). An irregular escarpment that extends for more than 250 km in the northern part of western Texas in the American southwest marks the boundary between a gently undulating upland surface known as the High Plains of West Texas and New Mexico, with elevations ranging from 1,000 to 1,500 m, and the dissected rolling plains of Central Texas to the east, with elevations typically 300–500 m lower (Collins 1984). Approximately 120 km southeast of Amarillo, Texas, is Caprock Canyons State Park and Trailway, a scenic and recreation area that straddles the cliffs of the escarpment and encompasses numerous canyons eroded into the less durable Permian and Triassic rocks under the cap rock. The cap rock is composed of Neogene Ogallala Formation, a *fluvial* aquifer composed of sand, silt, clay, and gravel; the upper part of the Ogallala Formation is carbonate-cemented silty and clayey sand with gravel known locally as *caliche* and more formally as *calcrete* (Machette 1985). It is the cemented upper part of the Ogallala Formation that comprises the cap rock at Caprock Canyons State Park.

In *petroleum geology*, in addition to a lower-hydraulic conductivity *sedimentary* unit that restricts upward migration of hydrocarbons, cap rock also forms above *salt domes* as a characteristic sequence of calcite, anhydrite, and gypsum that can exceed 300 m in thickness over the halite of the salt dome. The upward movement of the salt dome deforms the overlying rock formation, producing fractures into which the halite penetrates. Groundwater dissolves the upper surface of the intruding salt formation and any impurities in it, producing the anhydrite and gypsum. Interaction of anhydrite and gypsum with bacterial activity can produce sulfur in the cap rock of salt domes, sometimes in deposits of economic value for mining.

Cross-References

- ► Erosion
- ► Reservoirs
- ► Sedimentary Rocks

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Capillarity

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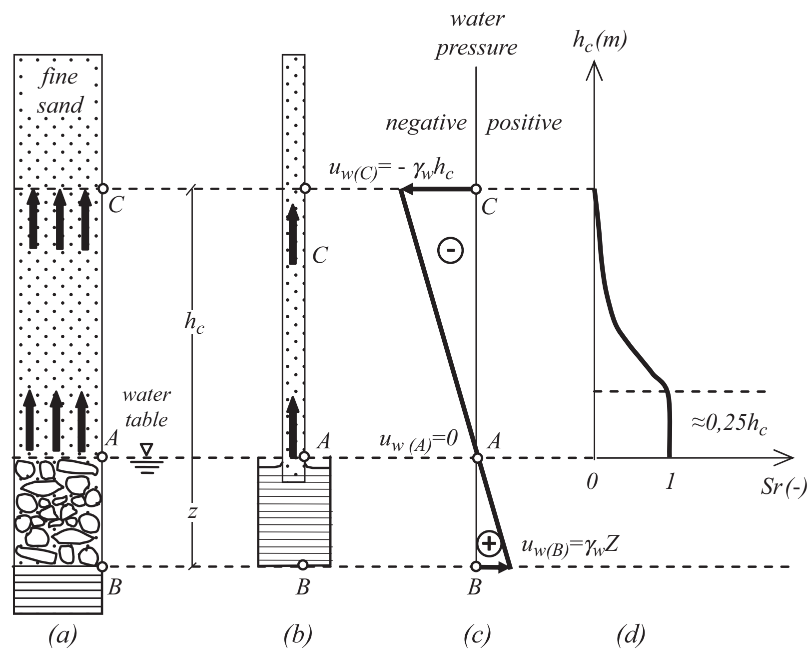
Definition

Capillarity in soils refers to the upward flow of water above the groundwater table.

This natural phenomenon of prevailing ascent of water in soil pores was compared, from the first decades of research, with the capillary rise of water in fine bore tubes (Fredlund and

Capillarity, Fig. 1 Capillary model. (**a**) Natural situation. (**b**) Thin tube filled with fine sand. (**c**) Water pressure distribution. (**d**) Capillary water distribution



Rahardjo 1993). In order to describe this state of water movement in soils, a capillary model must be defined in terms of capillary height and capillary pressure (see Fig. 1).

The length of capillary rise of pure water in thin glass tubes may be expressed in terms of equilibrium between the vertical resultant of the surface tension (T_s) and the weight of the water column and depends mainly on hygroscopic properties of the water and on the radius of the tube (r) (i.e., $h_c=2T_s/(\gamma_w r)$). In the case of soils, the maximum capillarity height is influenced mainly by matric suction (the pressure dry soil exerts on surrounding soils to equalize the moisture content in the overall block of soil), the distribution of effective porosity, which is a function of grain size distribution, and some physical properties of the water (temperature, mineralization). Typical values of h_c vary between 0.10–0.30 m for coarse sand and >2 m for fine soils. The phenomenon develops with a continuing decreasing rate and may last for months if water supply conditions remain unchanged. The capillary moisture decreases from a full degree of saturation near the contact with the water table level to a minimum irreducible degree at h_c level. Early studies (Hogentogler and Barber 1941; Florea 1980) demonstrate that on the first quarter of h_c, the high degree of saturation allows the mass transfer of capillary water and thus an unsaturated flow toward distal parts of the layer. This phenomenon, called "*siphon effect*" or "*capillary flow*," may damage downstream slopes of earth dams or tailings dams despite the apparent stabilizing effect of capillary saturation (i.e., increasing compression of the soil structure and consequently of the shear strength due to matric suction).

Capillary pressures developed inside soil structure during rising of the water are shown in section (c) of the figure. Based on the hydrostatic equilibrium of points A and C the matric suction is defined as the difference between pore-air and pore-water pressures acting on the contractile skin (interface air-water u_a-u_w; u_a = atmospheric air pressure; u_w = water pressure) (Fredlund and Rahardjo 1993). Thus defined, the matric suction is the main factor affecting matric potential gradient (Ψ_m) responsible, beside gravitational potential (Ψ_g), for the unsaturated water flow in both vertical and horizontal directions. This parameter is also involved in evaluation of hydraulic conductivity of unsaturated soils (Brooks and Corey 1966; van Genuchten 1980). The matric suction in soils may attain thousands of KPa for which the main measuring devices are: tensiometers, null-type pressure plates, thermal conductivity sensors, and pore fluid squeezers.

Cross-References

▶ Dams
▶ Tailings

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Casagrande Test

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Synonyms

Liquid limit test

Definition

A standard test to determine the liquid limit of a sample.

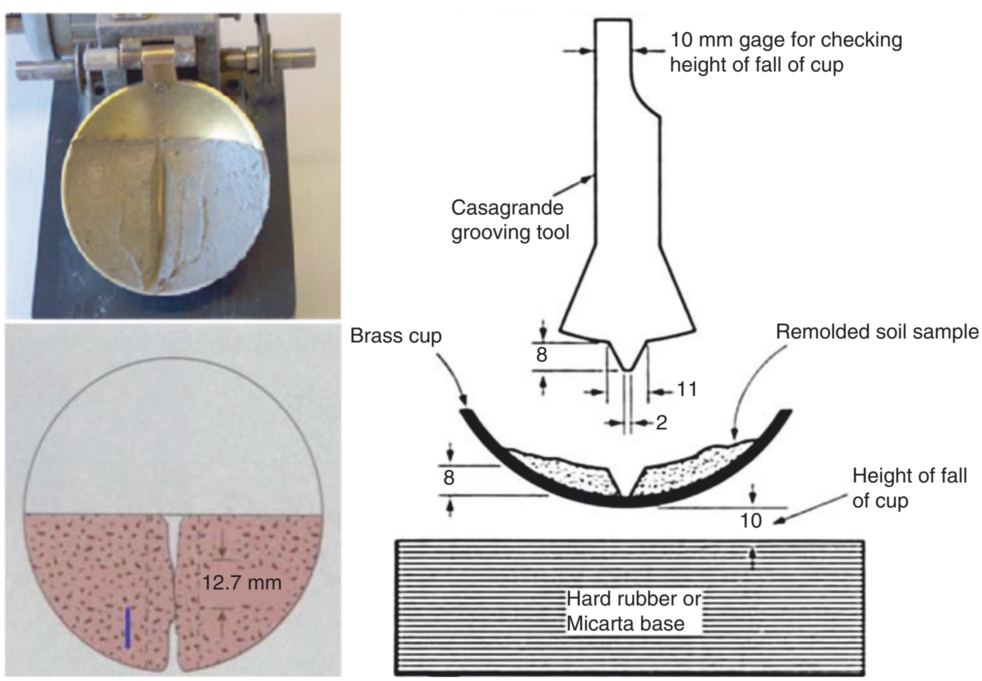
The standard liquid limit test apparatus was designed by Arthur Casagrande in the 1930s based on the procedure developed by Albert Atterberg; therefore, the liquid limit test is sometimes called the Casagrande test. The test apparatus consists of a Casagrande cup, also known as the liquid limit device, and a Casagrande grooving tool.

Soil is patted smooth across the brass cup to a thickness of 10 mm. The grooving tool is used to make a groove completely through the soil pat that is 2 mm wide at the bottom, 11 mm wide at the top, and 8 mm deep (ASTM 2010). The brass cup is hinged on one edge so that a cam shaft with a hand-operated crank can be used to raise the cup and allow it to drop abruptly 10 mm onto a hard rubber base at a rate of 120 drops per minute. The number of drops required to cause the soil to flow from both sides to close the groove over a distance of 12.7 mm is recorded; a sample of soil from the section that closed the groove is collected for determination of the water content. The test is repeated at least five times with the soil pat having different water contents such that the number of drops required to close the groove ranges from about 15 to about 35. The water content and the number of drops are used to calculate the water content that would close the standard groove the standard distance of 12.7 mm in exactly 25 drops.

The original liquid limit test, developed by Atterberg, was similar, except that the bowl was struck many times against the palm of one hand. Casagrande's improvement was to use a mechanical method to standardize the blows, thus making the experiment more repeatable (Fig. 1).

Cross-References

▶ Atterberg Limits
▶ Liquid Limit



Casagrande Test, Fig. 1 Casagrande apparatus, a brass cup hinged on one end and a standardized Casagrande grooving tool. The pat of sediment is divided. The mechanical cup lifts and drops onto a hard base, causing the groove to close (Modified from Holtz and Kovacs 1981)

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Casing

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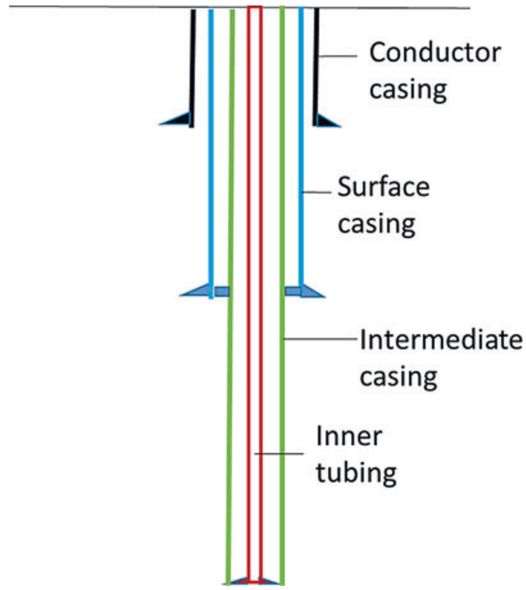
Definition

Tubular structures inserted into a well or borehole or within borings for emplacement of piles.

Casing has many applications in engineering geology. Well and borehole casing is a steel tubular pipe installed underneath the ground surface to access, extract, and transport natural resources from deep formations. Pile casing can serve as a permanent structural member to resist and transfer vertical and horizontal loads from superstructure to founding soil or rock. The term is also used in relation to pipelines but this entry focusses on wells, boreholes, and piles.

Well Casing

Different well and borehole casings are specially designed, installed, and operated for various applications in the energy and environmental engineering sectors. Conventional well casings can be used to access and extract oil, gas, or water from deep reservoirs. In some tight reservoirs where the permeability is in a range of nano Darcies, stimulation techniques such as hydraulic fracturing may be necessary to create new fractures or reactivate *in situ* natural fractures to increase the production rate. In some reservoirs containing heavy oil and bitumen with high *in situ* viscosity (in the range of 100,000–1,000,000 cp), condensed steam is injected into the formation to lower the oil viscosity so that the fluid (oil and water) can be extracted (Butler 1991). Such wells are also called thermal wells, as they are subjected to high pressures (in MPa) and temperatures (up to 300 °C). High temperatures and pressures are also encountered in casing wells that are used to extract geothermal energy from very deep formations by circulating cold water (Martinez-Garzon et al. 2013). Waste water produced from mining or oil extraction can be disposed of in certain contained formations.



Casing, Fig. 1 Diagram showing the configuration of a casing well

Typically, a completed well comprises a series of annular rings of steel casing, well cement, and the surrounding geological formations of varying thickness (Fig. 1). As part of well completion, a borehole is drilled and then advanced with the aid of drilling mud to prevent potential borehole collapse. Then, steel casings are inserted into the drilled hole and filled with mud. The mud is injected into the annular space between the steel casing and the drilled hole and is removed by a clear wash or spacer fluid, followed by injection of a cement slurry. Several casings are installed at multiple intervals to serve different functions. Starting from the well head at ground level, a conductor casing is installed to provide support during drilling operations, to prevent soil collapse near the ground surface, and to allow flowback returns during drilling and cementing of other casings. This casing can normally vary in nominal size from 18 to 30 in. (457 to 762 mm). A casing of smaller diameter ($13\frac{3}{8}$ in. or 339 mm), called a surface casing, is placed inside the conductor casing and cemented in place to isolate freshwater aquifers that may be present near the surface. An intermediate casing may be required for a deep well in which a well blowout induced by formation pore pressure or hydraulic fracturing (caused by the drilling mud weight) may occur. The main functions of casings are for protective or preventive measures. For production and/or injection, inner tubing with a liner (typically 7 in. or 178 mm in nominal size) is used. Tubing is generally easier for replacement and maintenance. Completed wells can vary from vertical to deviated horizontal. Deviated horizontal wells have become more popular because of their greater accessibility and less impact on surface disturbance.

The performance of a completed well is a complex process involving the thermal-hydraulic-mechanical-chemical interaction between the casing, cement annulus, and surrounding formations all subjected to different environmental loading. The structural and hydraulic integrity of a casing well is governed by this coupled interaction. In an ideal situation, all drilling mud should be removed, and the annular space should be filled with a cement slurry. Imperfect oil well cementing can produce weak and inhomogeneous cementitious material in the cement annulus. If the complete displacement of drilling mud by the spacer fluid and cement slurry is not achieved, a residual mud layer adhering to the inside and outside surfaces of the borehole and casing may be left. Localized channel or fingering may occur in the narrow spaces since the fluid displacement takes place in a narrow concentric annular configuration bounded by the circumferential surfaces of the drilled well and the steel casing. Fluid used in hydraulic fracturing or fluid from production can escape through the cement annulus behind the casing if the cement placement is not done properly. In thermal wells under cyclic thermal loading, thermally induced cracking in the cement annulus, formation, and interfaces could potentially jeopardize the structural and hydraulic integrity of the well.

In a production well, withdrawal of water, conventional gas or oil from deep reservoirs can result in reservoir compaction, and consequently subsidence in the formations overlying the reservoir. The subsidence can impair the casing. Reservoirs can experience dilation or expansion when stimulation techniques such as hydraulic fracturing or steam injection are applied during energy recovery. Hydraulic fracturing involves injection of fluids under high pressure into reservoir formations located at depth to fracture the formation. Steam injection produces dilatation of the reservoir due to an increase in pore pressure and thermal expansion of the reservoir, which can exert stress and deformation to the overburden. The resulting deformations induced during the recovery process could be excessive and detrimental to surface and subsurface facilities (Morgenstern et al. 1988). Casing failures have been reported in thermal recovery processes (Talebi et al. 1998). Mechanical energy released from hydraulically induced fractures or casing impairment and failure can generate microseismic events. Passive seismic monitoring using 3-component geophones along with geomechanical modeling have been employed in the field to determine the locations and mechanism of casing and formation failure (Talebi et al. 1998; Wong and Chau 2006).

Piles

Piles are vertical or inclined structural members made of steel, concrete, or timber installed in the ground to transfer vertical and or horizontal load from a superstructure to founding

Casing, Fig. 2 Image showing the installation of cast-in-place pile. A hole is bored by an auger inside the casing ground (Tomlinson and Woodward 2014). Piles can be categorized according to their installation method. Precast concrete, steel section, or timber piles are driven into the ground by displacing *in situ* soil. The driving process causes displacement and disturbance of the soil surrounding the pile. Another type of installation does not involve soil displacement. The soil is removed by drilling or boring to form a shaft with a casing (Fig. 2). In the excavation stage, the casing prevents soil collapse or caving into the bored shaft. Concrete is then cast in the shaft with the casing being left behind or removed to form the pile. Steel casing is required in either type of installation. Steel circular piles made of hollow sections (typically 30–60 cm in diameter) are commonly used as nondisplacement piles. Such piles are easy for driving, handling, and fabrication, and can be installed both onshore and offshore. Each pile must meet two criteria: load capacity and settlement. Both the load capacity and settlement of a pile are determined by the soil-pile interaction along the pile embedment length and at the pile bearing end. The design method depends on the ground in which the pile is installed, whether the soil is cohesive clay, granular sand, or competent bedrock. Piles founded in clay have different behavior in the short and long-term because the pore pressure induced during pile placement and loading requires additional time for dissipation due to the low permeability of clay. Piles embedded in frozen soils in cold regions can creep for years, and thus long-term pile settlement is the critical design criterion. For a complete design of a pile foundation system, one is required to design a pile cap. A pile cap is a structural member transmitting the loads from columns or walls of superstructure to the piles; these are commonly built of reinforced concrete or steel sections. Piles are installed at a certain minimum spacing to maximize their carrying load capacity; otherwise a reduction factor is required for design of a pile group.

In order to exercise quality control on pile performance, pile load tests are carried out in the field. Two standard pile load tests are recommended in practice: static and dynamic load tests. In a static pile load test, axial loads are statically applied to the test pile in increments, and the settlement of the pile is recorded. The axial loads can be applied by stacking concrete blocks on a loading frame attached to the pile, or by exerting reaction forces using hydraulic jacks with a reaction frame. The axial load-settlement record provides the pile load capacity as a function of settlement. The settlement has to be within the allowable range. In a dynamic load test, a dynamic load of known energy (a falling hammer or dynamite charge) is applied to the test pile while recording acceleration and strain on the pile head. The recorded data can be used to analyze the dynamic response of the soil-pile interaction using a wave propagation equation in viscoelastic medium. Test results give information on resistance distribution (shaft resistance and end bearing) and help evaluate the shape and integrity of the pile element. The pile bearing capacity results obtained with dynamic load tests correlate well with those of static load tests performed on the same pile.

Summary

Casing has many practical applications in engineering geology which are critical to economic development in sustainable communities. Installation and operation of casing must be carried out with no or minimal disturbance to the environment. Thus, casing must interact compatibly with the surrounding formations in terms of structural and hydraulic integrity. Casing must be constructed from appropriate materials that prolong longevity with improved protection and self-healing capabilities, allow continual monitoring of performance, and provide detection of early defects.

Cross-References

▶ Boreholes
▶ Compression
▶ Dynamic Compaction/Compression
▶ Ground Preparation
▶ Landslide
▶ Mechanical Properties
▶ Pipes/Pipelines
▶ Soil Field Tests
▶ Subsidence

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Catchment

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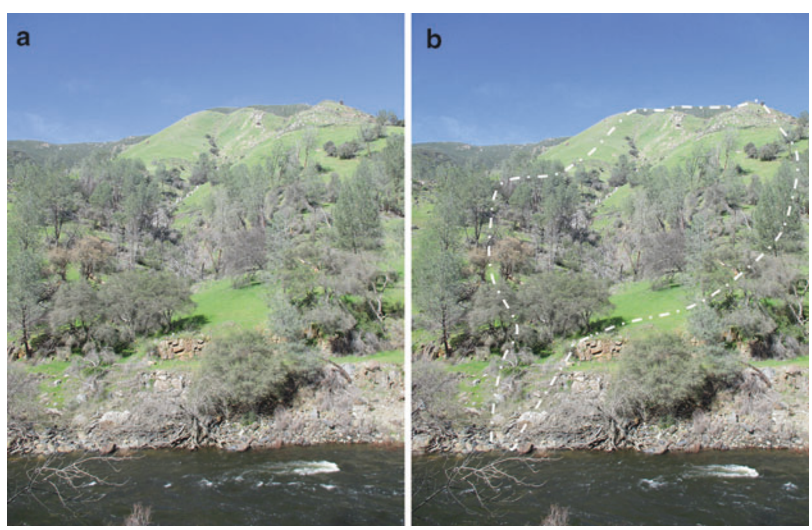
Definition

A catchment is an area on the earth's surface where runoff from rainfall or snowmelt and groundwater discharge from springs and seeps is collected at the same discharge point. In a natural setting, the catchment area is equivalent to a drainage basin (Langbein and Iseri 1960). The water collected within a catchment may be discharged as stream flow into another stream or a body of water. A watershed is one or more catchments discharging to the same downgradient point. Determining the boundaries of a specific catchment uses topographic map or digital terrain model data to find where water would flow inward and downgradient to a particular catchment rather than into an adjacent one. The size of delineated natural catchments is controlled by the physical character of the landscape and the purpose for identifying the component catchments within a watershed. Within the built environment, a catchment would also include runoff from impermeable surface such as roofs and paved areas (New York State 2010). The collected water may be discharged through a constructed drainage system into a settling pond, canal, underground pipe, a body of water, or a natural stream. Regulatory requirements may specifically define the size or limits of a catchment for the purposes of controlling storm water discharge or other offsite discharge (Fig. 1).

Defining catchments is fundamental to addressing many environmental and engineering issues. Assessing runoff contributing various contaminates such as sediment, nitrates, and arsenic being introduced into streamflow is one common environmental issue. Defining the catchments involved is an initial step in studies to better understand this problem. Defining catchments is a necessary design element for determining the size of culverts directing water past roads, railroads, and structures. In constructed drainage systems, knowing the contributing catchment area is basic information for calculating the correct size of elements through which water will be conveyed and those where water will be contained (New York State 2010; San Diego County 2003). The widespread use of catchments for many different environmental and engineering geologic applications has resulted in development of computerized applications and models (see Pullar and Springer (2000) and Schmitt et al. (2004) for examples).

Cross-References

- ▶ Landforms
- ▶ Land Use
- ▶ Run-Off
- ▶ Water



Catchment, Fig. 1 Images A and B show the same catchment along the canyon of the Merced River downstream from El Portal, California. The Merced River is visible in the lower foreground. Image A shows the natural catchment evident to the eye by the shape of the topography and the visible internal channels. Image B adds a general delineation (*dashed white line*) to accentuate the limits of this catchment contributing surface water to the Merced River

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Cement

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Definition

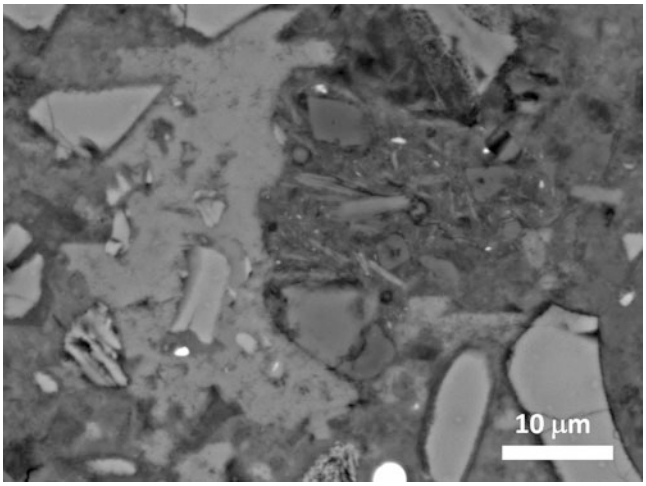
In the broad sense, a material which can bind other materials together into a hardened, cohesive mass. Cements in general may be organic or inorganic, including various plasters and glues, but the most important classes of cements used worldwide are those which are *hydraulic*; that is, harden through addition of water to form a water-insoluble final product. The dominant hydraulic cement used worldwide is Portland cement (Hewlett 1998), which consists primarily of hydraulic calcium silicates in addition to calcium sulfate, aluminate, and aluminoferrite phases (ASTM International 2016). Alternatives to Portland cement in some applications include gypsum or lime (particularly as plasters), geopolymers, calcium aluminate or sulfoaluminate cements, and magnesia-based cements. However, considering the current domination of cement usage by Portland cement, this is the material described in detail here.

Characteristics

Portland cement is produced through thermal treatment (calcination) of limestone ($CaCO_3$, see "▶ Limestone") together with clay or shale, at temperatures around 1400–1450°C. Under these conditions, the limestone is decarbonized, and the resulting lime (CaO) can combine with silica to form tricalcium silicate (Ca_3SiO_5) and dicalcium silicate (Ca_2SiO_4), which are also known in cement chemistry as "alite" and "belite," respectively (Hewlett 1998). These phases are the synthetic analogues of the pure mineral phases hatrurite and larnite and accommodate ionic substitution by many different elements up to levels of ~1%. The alumina and ferric iron supplied along with the silica in the clay or shale also combine with calcium to form tricalcium aluminate ($Ca_3Al_2O_6$) and brownmillerite-type tetracalcium aluminoferrite (Ca_2AlFeO_5). These four calcium-rich hydraulic phases, which constitute the "cement clinker," are retained through relatively rapid cooling to room temperature, and intergrinding of the clinker with approximately 5% calcium sulfate (often gypsum, $CaSO_4{\cdot}2H_2O$, or partially dehydrated forms, e.g., hemihydrate) then yields Portland cement.

The reaction of Portland cement with water initiates a hydration process, which is exothermic. The primary hydration product, and the phase which is responsible for the majority of the strength in a hardened Portland cement, is a disordered calcium silicate hydrate with a layered-chain structure resembling that of tobermorite (Richardson 1999). This phase has a calcium/silicon atomic ratio between 1 and 2, and so the additional calcium provided by the tricalcium silicate and dicalcium silicate precipitates as portlandite, $Ca(OH)_2$. This conditions the pH of the pore fluid within cements to highly alkaline values, often exceeding 12.5. The calcium aluminate and aluminoferrite hydrate together with the calcium sulfate, to form a range of calcium sulfoaluminate hydrates in the ettringite and hydrocalumite families (termed "AFt" and "AFm" respectively by cement practitioners) (Lothenbach and Winnefeld 2006). These phases contribute to the properties of the cement in both the fluid and solid states, particularly in terms of influencing (in either positive or negative senses) the durability of the hardened cement. Surface-active organic admixtures are also often added, at doses of less than 1%, to control the flow characteristics of cements in the fluid state (plasticizers or superplasticizers) and/or to entrain air voids within the material as it hardens (air-entraining agents).

Modern Portland cements are also widely blended or interground with mineral admixtures including coal fly ash, blast furnace slag, natural reactive aluminosilicate minerals (calcined or uncalcined), and also additional limestone (European Committee for Standardization 2011). These admixtures react with the cement constituents during hydration, generally over a more extended timeframe (weeks to months) than the main cement hydration reaction, which is dominant in the first few hours and up to several weeks after mixing. The key reaction of most mineral admixtures involves the portlandite produced in cement hydration, which combines with the silica provided by the mineral



Cement, Fig. 1 Scanning electron micrograph of a polished section of a hydrated Portland-blast furnace slag cement, showing residual cement and slag grains (brighter discrete regions) embedded in a cohesive matrix of calcium silicate hydrate and other hydrate products. Image courtesy S.A. Kearney, University of Sheffield

admixtures to form additional calcium silicate hydrate, thus bringing additional strength and durability to the hardened cement. An example of the complex microstructure formed by hydration of a Portland-blast furnace slag cement is shown in Fig. 1.

The other main reason for addition of mineral admixtures relates to the desire to reduce the environmental emissions footprint of the cement as a whole; because these do not require the same degree of thermal processing as Portland cement, nor the decarbonation of limestone, the overall emissions per tonne of cementitious material can be reduced significantly through the judicious use of mineral admixtures. Given that Portland cement production results in up to 8% of global CO_2 emissions as four billion tonnes of cement are produced annually, this is an important consideration and in many cases is the main reason for the use of blended cements.

Cements are used in combination with aggregates to produce concretes, and concrete is in turn often reinforced with steel to produce reinforced concrete for use in construction and infrastructure. For such applications, the chemistry of the cement must be matched appropriately to the mineralogy of the aggregate to prevent degradation through alkali-silica reactions and must also provide an environment which passivates the steel surface to prevent corrosion, including resistance to environmental attack, e.g., from external chloride. The use of mineral admixtures is important in tailoring the cement chemistry to provide such characteristics. Cements for use in waste management or other specialty applications such as well cementing, often have their chemical and physical properties manipulated to optimize performance in the specified application, including grinding to different particle sizes or blending with additives differing from those which are specified in standards that focus on construction applications.

Cross-References

▶ Aggregate
▶ Alkali-Silica Reaction
▶ Concrete
▶ Geopolymers
▶ Infrastructure
▶ Limestone
▶ Waste Management

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Characterization of Soils

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Synonyms

Engineering behavior of soils; Engineering properties of soils

Definition

A soil is a loose, unconsolidated agglomeration of mineral particles that can be easily separated by hand pressure or by immersion in water (Johnson and DeGraff 1988) and that can be excavated without blasting (West 1995). Geologically, soils are the products of mechanical and/or chemical weathering of rocks (Marshak 2013).

Introduction

Soils are one of the most widely encountered materials in engineering construction. Many engineering structures are either made of soil material (earth dams and levees) or founded on soils (buildings) or located within soils (tunnels and other underground structures). The design and stability of these structures depends on the engineering properties of soils involved.

Based on their origin, soils are categorized as residual or transported (Holtz et al. 2011). Residual soils remain at their place of origin, whereas transported soils are carried away from their place of origin by such agents as gravity (colluvial soils), water (alluvial soils), ice (glacial soils), and wind (aeolian soils). Engineering properties of soils are closely related to their origin. Residual soils are likely to exhibit a well-developed soil profile, colluvial soils are dominated by angular particles resulting in higher friction angle, alluvial soils are generally stratified, glacial soils can be highly heterogeneous with a wide range in particle size, and aeolian soils are characterized by fine, uniform particle size (Holtz et al. 2011; Marshak 2013).

Engineering Characterization of Soils

For characterization purposes, engineering properties of soils are grouped into index properties and design properties.

Index Properties

Soil Texture

Soil texture relates to grain size distribution (gradation) and grain shapes. Soils can be coarse-textured (sand and gravel) or fine-textured (silt and clay), the distinction between the two groups being whether the grains are larger or smaller than 0.074 mm (#200 sieve). Texture controls the behavior of coarse-grained (granular) soils and water controls the behavior of fine-grained (cohesive) soils.

Grain size can vary from boulders (10^3 mm) to colloidal size clay material (10^{-5} mm). Sieve analysis (ASTM D 6913; ASTM 2010) is used to determine grain size distribution of coarse-grained soils, and hydrometer analysis (ASTM D 422; ASTM 2010) is used to determine grain size distribution of fine-grained soils. Figure 1 shows the grain size distribution curves for three different soils. A well-graded soil is one in which all grain sizes are well represented, a gap-graded soil is missing certain sizes, and a uniformly graded or poorly graded soil consists predominantly of one size of grains. A well-graded soil exhibits the best engineering properties, whereas uniformly graded soils can be problematic.

The following quantitative indices are commonly used to describe soil gradation:

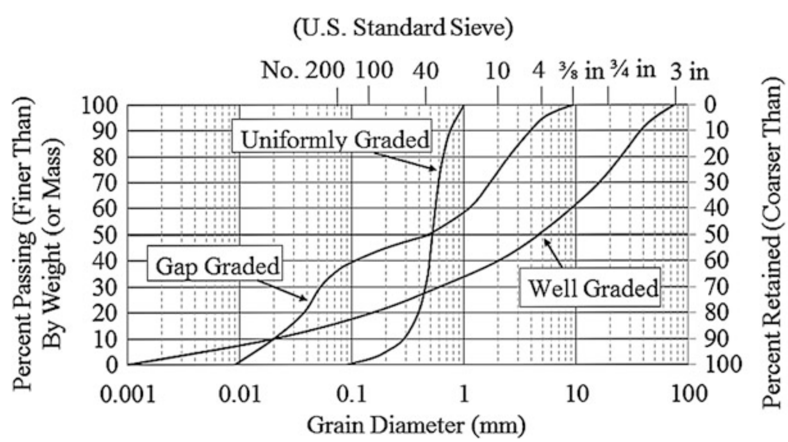
$$\text{Coefficient of uniformity} = C_u = D_{60}/D_{10} \quad (1)$$

$$\text{Coefficient of curvature} = C_c = (D_{30})^2/(D_{10})(D_{30}) \quad (2)$$

where D_{10}, D_{30}, and D_{60} are grain sizes corresponding to 10%, 30%, and 60%, by weight, of the soil finer than the corresponding diameters, respectively. A soil will be well-graded if its C_c is between 1 and 3 and C_u is greater than 4 for gravel and greater than 6 for sand (Holtz et al. 2011).

Phase Relationships

A mass of soil commonly consists of three phases: solid mineral particles, water, and air. For a completely saturated and a completely dry soil, all voids (pores) are filled with water and air, respectively, and the soil mass



Characterization of Soils, Fig. 1 Grain size distribution curves reduces to a two-phase system. Figure 1 shows a schematic representation of the masses and volumes of various phases involved. The interrelationships between these phases define some important index properties used for soil characterization.

Void Ratio (e): Void ratio is the ratio of the volume of voids to the volume of solids ($e = V_v/V_s$). The higher the void ratio, the more compressible is the soil. Typical values of void ratio can range from 0.4 to 1.0 for sands, 0.3 to 1.5 for clay, and much higher for organic soils (Holtz and Kovacs 2011).

Porosity (n): Porosity is the ratio of the volume of voids to the total volume of a soil mass, expressed as a percentage $\{n = \{(V_v/V_t) \times 100\}$. Clayey soils tend to have higher porosity values (30–70%) than sandy soils (20–50%). Void ratio and porosity relate to each other as follows:

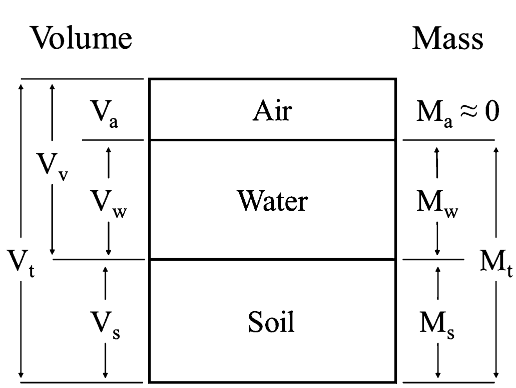
$$e = n/1 - n \tag{3}$$

$$n = e/1 + e \tag{4}$$

Degree of Saturation (S): Degree of saturation is the ratio of the volume of water to the volume of voids in a soil mass, expressed as a percentage $\{S = (V_w/V_v) \times 100\}$. It ranges from 0% for a completely dry soil to 100% for a completely saturated soil. The lower the degree of saturation of an expansive clayey soil, the more will it expand upon the addition of water.

Water Content (w): Water content is the ratio of the mass of water to the mass of solids, expressed as a percentage $\{w = (M_w/M_s) \times 100\}$. The water content for natural soils can range from 0% for a completely dry soil to several hundred percent for some marine organic clay. The higher the natural water content of a soil, the less desirable are its engineering properties.

Density (ρ) : Density connects the two sides of the phase diagram in Fig. 2. Density is the ratio of the mass to the volume. In engineering practice, different types of density are used such as bulk density ($\rho = M_t/V_t$), solid density ($\rho_s = M_s/V_s$), dry density ($\rho_d = M_s/V_t$), saturated density $\{\rho_{sat} = (M_s + M_w)/V_t$, with M_w at S=100%$\}$, and submerged density ($\rho' = \rho_{sat} - \rho_w$).



Characterization of Soils, Fig. 2 Phase diagram showing mass-volume relationships for soils

Atterberg Limits

Atterberg limits are water contents at which marked changes in the engineering behavior of fine-grained soils occur. By comparing the natural water content of a soil with its Atterberg limits, one can predict its engineering behavior. Important Atterberg limits include liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). Liquid limit is the minimum water content at which a soil behaves as a viscous liquid and plastic limit is the minimum water content at which a soil behaves as a plastic material. Liquid and plastic limits for fine-grained soils can be determined by ASTM method D 4318 (ASTM 2010). The numerical difference between LL and PL is referred to as plasticity index (PI). It indicates the range of water content over which a soil behaves as a plastic material. Shrinkage limit is the minimum water content beyond which, upon drying, no further reduction in volume occurs.

Atterberg limits are important for characterizing fine-grained soils as they are used for classifying fine-grained soils and correlate with most other engineering properties. Soils with low SL and high PI values are prone to detrimental volume change with changes in water content.

Liquidity Index

Liquidity index compares the natural water content of a soil with its Atterberg limits as follows:

$$LI = (w_n - PL)/PI \tag{5}$$

where:

w_n = natural water content

A soil will behave as a brittle solid upon shearing if its LI is less than 0, as a plastic material if LI is between 0 and 1, and as a viscous liquid if LI is greater than 1. LI values > 1 characterize ultra-sensitive clay, which lose their strength upon shaking and flow like a liquid.

Activity Index

Activity index (A) indicates the sensitivity of fine-grained soils to changes in water content and is defined as:

$$A = PI/\%2\ \mu m\ (0.002\ \text{mm})\ \text{clay} \tag{6}$$

Clay with A values less than 0.75 are considered inactive whereas those with A values greater than 1.25 are active. Activity is closely related to clay mineralogy, with montmorillonite exhibiting the highest activity. Activity index is useful in predicting the swelling potential of a clay soil (Mitchell 1993).

Soil Classification

The Unified Soil Classification System (USCS), developed by Casagrande (1948), is one of the most commonly used classification systems. According to this system, coarse-grained soils are classified based on grain size distribution and fine-grained soils on the basis of plasticity characteristics as indicated by Atterberg limits. Soils for which more than 50% by weight is retained on sieve No. 200 (0.074 mm) are considered coarse-grained and those with more than 50% passing the No. 200 sieve are classified as fine-grained. Coarse-grained soils are categorized as gravel if more than 50% material is retained on No. 4 sieve (4.75 mm) and sand if more than 50% material passes the No. 4 sieve. Gravel is considered coarse if it is 19–75 mm and fine if it is 4.75–19 mm. Sand is further classified into coarse sand (2.00–4.75 mm), medium sand (0.425–2.00 mm), and fine sand (0.074–0.425 mm).

Silt and clay, according to USCS, are differentiated based on plasticity characteristics, not particle size. This is accomplished by plotting LL and PI values on the Casagrande Plasticity Chart shown in Fig. 3. All points falling above the A-line in Fig. 3 represent clay and those falling below the A-line indicate silt. Further subdivision is based on whether the LL is more or less than 50.

In the USCS, letters G, S, M, C, O, and Pt are used for gravel, sand, silt, clay, organic soil, and peat, respectively. Letters W, P, H, and L designate well-graded, poorly graded, high plasticity, and low plasticity soils, respectively. For example, GW will be used for well-graded gravel, ML for silt of low plasticity (LL < 50), and CH for clay of high plasticity (LL > 50). Dual symbols are used for coarse-grained soils with 5–12% fineness (material finer than 0.074 mm) or for fine-grained soils whose LL and PI combinations fall in the hatched area in Fig. 3.

C

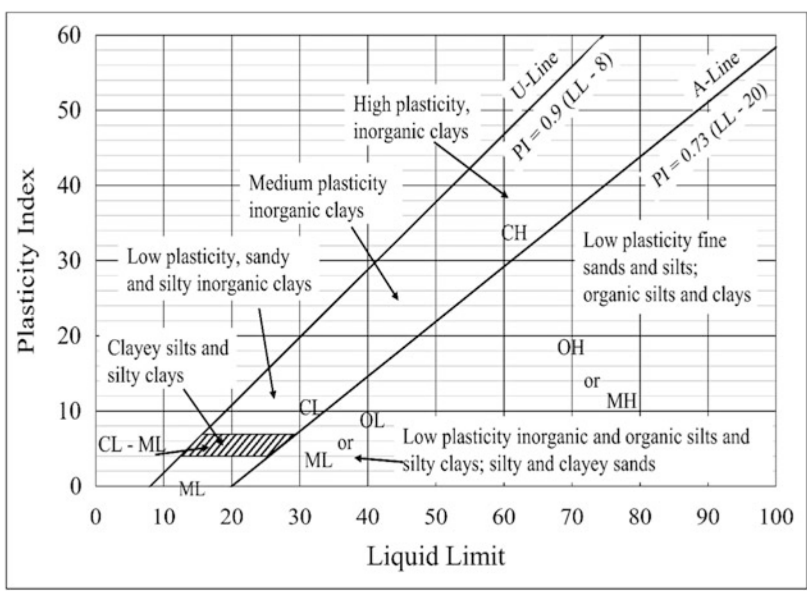
Design Properties

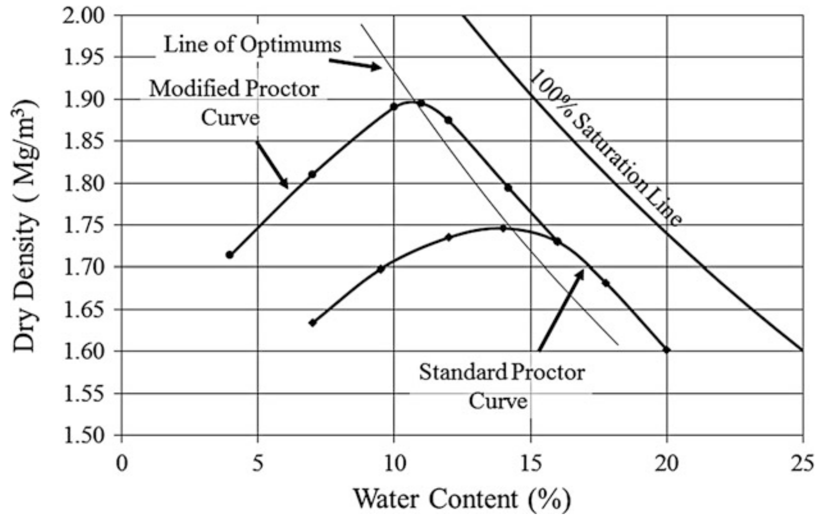
Compaction Characteristics

Compaction is densification of soils through rearrangement of soil particles using mechanical means. Compaction reduces settlement, improves bearing capacity and shear strength properties, and minimizes detrimental volume changes. Compaction is measured in terms of dry density.

The maximum achievable density depends on water content, compactive effort (amount of energy), and soil type (gradation, plasticity characteristics, etc). The compaction curves in Fig. 4 show the relationship between dry density, water content, and increased compactive effort. Tests used to establish the curves in Fig. 4 are the standard Proctor test (ASTM D698; ASTM 2010) and the modified Proctor test (ASTM D1557; ASTM 2010). For a given soil and given compactive effort, maximum dry density (MDD) is achieved at a certain water content referred to as the optimum water content (OWC). An increase in compactive effort increases

Characterization of Soils, Fig. 3 Casagrande's plasticity chart showing classification of fine-grained soils





Characterization of Soils, Fig. 4 Standard and modified Proctor compaction curves

MDD and reduces OWC. Granular soils tend to achieve higher density values at lower values of OWC compared to silty and clayey soils because cohesive forces between clay particles tend to resist rearrangement.

Compaction specifications require that soils be compacted to density values greater than 95% of MDD value and within 2% of OWC value. Smooth wheel and pneumatic rollers can be used for compacting both granular and cohesive soils, sheepsfoot rollers are best for compacting cohesive soils, and vibratory action is most effective in compacting granular soils.

Permeability

Permeability is the ease with which water flows through a mass of soil or rock. Information about permeability is required for problems involving seepage through earth dams, coffer dams, subsurface drains for roadways, water yield of aquifers, and foundation settlement.

Darcy's law expresses flow through a porous medium, as follows:

$$q = kiA \tag{7}$$

where:

q = quantity of flow through a given cross-sectional area
k = permeability
i = hydraulic gradient; a dimensionless number obtained by dividing the loss in head (h) by the distance (L) over which the head loss occurs
A = cross-sectional area through which flow occurs

The quantity of flow per unit area (q/A) defines the velocity of flow (v). Therefore, by substitution:

$$v = ki \tag{8}$$

where:

v and k both have units of cm/s or m/h.

In the laboratory, permeability is tested by using a constant head permeability test (ASTM D2434; ASTM 2010) for coarse-grained soils ($k > 10^{-4}$ cm/s) and a falling head test (ASTM D2435; ASTM 2010) for fine-grained soils. For rough estimates of permeability for clean sand, Hazen's empirical equation (Hazen 1911) is frequently used. According to this equation:

$$k = C(D_{10})^2 \tag{9}$$

where:

k = permeability in cm/s
C = 0.4–1.2, with an average value of 1
D_{10} = effective particle size in mm

For major projects, field-pumping tests (Fetter 1994) are frequently employed to obtain more representative values of permeability.

The three benchmark-values of permeability are: 1 cm/s that marks the boundary between laminar and turbulent flow, 10^{-4} cm/s that separates well-drained and poorly drained soils from each other, and 10^{-9} cm/s that marks the lower limit of permeability values for soil and rock.

Consolidation

Consolidation is the reduction in volume of fine-grained soils due to expulsion of water under the influence of increased stress. As the water drains out, the load previously carried by water is gradually transferred to soil particles. This increases the effective stress and decreases the thickness of a compressible layer that, in turn, results in settlement of the structure.

Since the amount of settlement generally varies over a large site, the differential settlement can result in structural damage. There are two aspects of settlement that are of main concern: (1) total amount of settlement and (2) time rate of settlement. A structure may be able to tolerate a relatively large amount of settlement if it occurs at a slow rate.

In the laboratory, a consolidation test (ASTM D 2435; ASTM 2010) is used to determine the consolidation characteristics of fine-grained soils. In this test, an undisturbed sample of saturated soil is placed in a ring, with porous stones placed on top and bottom to serve as drainage layers, and loaded incrementally. The void ratio is computed at the end of consolidation under each load increment. A void ratio versus load curve, referred to as the compression curve, is plotted on a semi-log paper. The compression index, C_c, which represents the slope of the virgin portion of the curve, is determined to compute settlement using the following equation:

$$\begin{aligned}\text{Settlement} &= \Delta H \\ &= (C_c/1 + e_o) \times H \times \log\left\{(\sigma'_o + \Delta\sigma)/\sigma'_o\right\}\end{aligned} \tag{10}$$

where:

ΔH = settlement
C_c = compression index
H = initial thickness of the clay layer
e_o = initial void ratio
σ'_o = effective stress at the middle of the clay layer
$\Delta\sigma$ = change in effective stress at the middle of the clay layer caused by the structure

The compression index for most soils ranges from 0.1 to 0.4 but can be much higher for organic soils. Methods for determining $\Delta\sigma$ are described in Holtz and Kovacs (2011).

The time for consolidation or settlement to occur depends on the number of drainage boundaries surrounding the clay layer, i.e., whether the clay layer is singly drained or doubly drained as well as the thickness and permeability of the clay layer. The settlement time can be computed from the following equation:

$$t = T_v H^2 / c_v \tag{11}$$

where:

t = time required for consolidation to occur
T_v = time factor
H = maximum length of drainage path
c_v = coefficient of consolidation; determined from the results of consolidation

Procedures for determining T_v and c_v can be found in most soil mechanics books.

Shear Strength

Shear strength is the ability of a soil to resist movement along internal surfaces. It depends on cohesion and angle of internal friction (strength parameters). The shear strength of soils plays an important role in design, construction, and stability of structures built on, in, and of soil materials.

The shear strength of a soil is defined by the following equation:

$$\tau = c + \sigma_n \text{ x } \tan\varphi \tag{12}$$

where τ, c, σ_n, and φ are the shear strength, cohesion, stress normal to the shear surface, and friction angle, respectively. For purely granular soils (clean sand and gravel) under drained conditions, cohesion is zero and $\tau = \sigma_n \times \tan\varphi$. For purely cohesive soils (plastic silt and clay) under undrained conditions, the friction angle is equal to zero and $\tau = c$. However, for most soils, the shear strength is attributable to both cohesion and friction.

The three laboratory tests that are used to determine the shear strength parameters include the direct shear test (ASTM D 3080; ASTM 2010), triaxial test (ASTM D 4767; ASTM 2010), and unconfined compression test (ASTM D 2166; ASTM 2010). Overall, granular soils exhibit better shear strength characteristics than cohesive soils, especially in the presence of water.

Summary

The two classes of properties used to characterize soils are: index properties and design properties. Index properties, used for characterizing soils in general, include grain size distribution, phase relations (void ratio, porosity, water content, degree of saturation, and density), liquid limit, plastic limit, plasticity index, shrinkage limit, liquidity index, and activity index. Design properties influence the design and stability of engineering structures. They include compaction characteristics, consolidation characteristics (amount and rate of settlement), and shear strength parameters (cohesion and friction angle). Both index and design properties can be determined by standardized laboratory tests.

Cross-References

- ▶ Atterberg Limits
- ▶ Casagrande Test
- ▶ Classification of Soils
- ▶ Clay
- ▶ Cohesive Soils
- ▶ Collapsible Soils
- ▶ Compaction
- ▶ Cone Penetrometer
- ▶ Consolidation
- ▶ Density
- ▶ Dewatering
- ▶ Diagenesis
- ▶ Dissolution
- ▶ Dynamic Compaction/Compression
- ▶ Engineering Properties
- ▶ Expansive Soils
- ▶ Gradation/Grading
- ▶ Hydrocompaction
- ▶ Infiltration
- ▶ Liquid Limit
- ▶ Noncohesive Soils
- ▶ Percolation
- ▶ Plasticity Index
- ▶ Sand
- ▶ Shear Strength
- ▶ Silt
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Mechanics
- ▶ Soil Properties

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Chemical Weathering

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Synonyms

Chemical action; Chemical alteration; Chemical decomposition; Chemical process; Chemical reactions

Definition

Weathering of rocks caused by the chemical action of water containing atmospheric oxygen, carbon dioxide, and some organic acids in solution on the rock-forming minerals leading to an adjustment of the mineralogical composition with the formation of new minerals, like hydrous phyllosilicates, iron oxides/hydroxides, soluble salts, and other alteration products, consisting of rock decay by chemical decomposition.

Introduction

Chemical processes need water, occurring more rapidly at higher temperature, so they are more common in warm and wet climates. There are different types of chemical weathering processes, such as solution, hydration, hydrolysis, carbonation, oxidation, reduction, and chelation. Some of these reactions occur more easily when the water is slightly acidic. Weathering of rocks is a fundamental phenomenon for the formation of the soil, and therefore support of Life on Earth.

The term "weathering" can be defined in several ways according to the specific intent of the author, whether an engineering geologist, geomorphologist, pedologist, etc. Thus, Reiche (1950) defined weathering as "the response of materials that were in equilibrium with the lithosphere, in the existing conditions together with its contact with the atmosphere, hydrosphere and biosphere." Ollier (1984) states: "weathering is the decomposition and fragmentation of rocks near the earth's surface, mainly due to the reaction with water and air, resulting in the formation of clay, iron oxides/hydroxides and other alteration products." Selby

(1993) describes weathering as a "process of decomposition and disintegration of the soils and rocks near the Earth's surface by physical, chemical and biological processes." Price (1995) proposes a definition for geotechnical purposes: "weathering is the irreversible response of soils, rocks and their respective massifs to their natural or artificial exposure near the geomorphological surface or in engineering works."

Some authors distinguish the term "weathering" from the term "alteration," stating that the first, deriving from the Anglo-Saxon terminology, constitutes a particular type of alteration, aiming to relate the phenomenon with the external agents, particularly with those of meteoric alteration nature. The term "alteration" has a broader meaning, translating any modification experienced by a rock, whether physical, chemical, or biological, of endogenous or exogenous domain. In fact, the term "alteration" also encompasses endogenous and hydrothermal processes. According to Aires-Barros and Miranda (1989), the effects of the hypogenic, deuteric, or primary alteration phenomena and the supergenic, meteoric, or secondary alteration phenomena are of the same type (kaolinitization, sericitization, chloritization, muscovitization, etc.) and explain how minerals form through these processes. There are minerals, mainly clay minerals, which can be formed by the two processes, or by only one of them and modified by the other.

One aspect gathers consensus among all authors, whatever their basic formation: weathering is one of the most important geological processes because:

(i) It is the geological phenomenon that underlies the formation of agricultural soil, without which Life would not be possible.
(ii) The processes studied in sedimentology and geomorphology are directly related to the weathering of rocks and soils.
(iii) Promotes the formation of residual soils, which sometimes constitute concentrations of minerals with the potential to be economically important, such as kaolin, bentonite, laterite, bauxite, and gravel.
(iv) From the point of view of engineering geology, the interest in the study of the phenomenon of weathering is increasing, due to the expanding use of rocky materials in important engineering works such as dams, roads, railways, and maritime works, which may change over time, and so it is important to predict their medium and long-term behavior as a building material, as a foundation ground or when requested by underground works.
(v) Research into the degradation and conservation of the "natural stone" of historic monuments has also recently been a great incentive for studies about weathering.

In nature, the chemical weathering of rocks and soils rarely works alone; in fact, it is closely associated with physical weathering, often being initiated and potentiated by the latter. For instance, mechanical actions caused by several physical weathering agents, such as temperature, water flow, glaciers, wind, biological activity (roots of trees, cavities made by rodents), provoke fissures, fractures, fragmentation, and expansion of rocks and minerals, providing a favorable environment to chemical weathering by increasing the exposed surface to chemical agents, as well as multiplying the preferred pathways of water. Chemical weathering becomes more effective as the specific surface area of the rock increases. Since the chemical reactions occur largely on the surface of the rocks, therefore, the smaller the fragments, the greater the surface area per unit mass or unit volume available for reaction.

Agents and Processes

Rate of weathering depends on rock type, climate (essentially precipitation and temperature), and geomorphology (Duarte 2002; Duarte et al. 2004). Most authors consider climate to be the most important factor in rock weathering and consequently in soil formation. The prevailing factors that control the way weathering evolves are precipitation, mean air temperature, and the periodicity of the changes involving these two variables (Ollier 1984). Within the climatic factors, rainfall and temperature are undoubtedly the factors most responsible for the physical fissuration and disaggregation of rocks and for the chemical weathering of minerals, contributing much to the quality and quantity of soil output from rock weathering. Even the vegetation cover, which closely depends on climate, could indirectly promote or inhibit the rock weathering processes, either by increasing chemical reactivity or diminishing runoff, and consequently affecting the susceptibility to weathering of rocks and soils.

In the process of weathering, the original rock undergoes a series of physical and chemical modifications due to its interaction with the factors of the environment (exogenous) in which the rock is found, and from certain levels will influence on its intrinsic characteristics (texture, porosity, permeability, color, strength, etc.).

Although the action of the agents of weathering (water, temperature, pH, wind, living organisms, etc.) occurs concomitantly (in time and space) in manifestly diverse degrees, sometimes in disguised ways, for practical reasons, it is common to separate the mechanisms of physical nature and the mechanisms of chemical nature for their better understanding. Thus, the use of expressions such as physical degradation and chemical decomposition have been current (Anon 1995). The first one is responsible for the loss of the structural individuality of the original rock due to the fracture of the rock and mineral particles and to the reduction of their size, without changing the nature of the minerals; the second leads to a modification in the chemical and mineralogical composition of the rock. Physical weathering tends to occur close to the surface, whereas chemical alteration can be extended to depths of the order of tens or hundreds of meters (Chorley 1969). But any process is most active, according to the specific conditions of the site and the prevailing climate. Chemical weathering is predominant in tropical climates, being mainly responsible for the formation of large thicknesses of tropical residual soils (Townsend 1985).

Chemical processes, mainly hydrolysis, cation exchange, and oxidation-reduction, transform the original minerals of the rock and form clay minerals, more stable in the surficial environment. Other mechanisms such as hydration, solution, carbonation, and complexation are more or less important depending on the local climate, drainage conditions, type of rock, vegetation cover, among others. Biological alteration includes chemical action, such as bacteriological oxidation, contribution to chelate formation, and reduction of iron compounds and sulfides (Pings 1968), and dominates in regions where vegetation is luxuriant as in the case of tropical forests.

Most processes of chemical weathering depend on the presence of water and its movement through the massifs. These processes generally include solution, and the intensity with which it acts depends mainly on the amount of water circulating in the massifs, the solubility of minerals, and the pH of the water.

1. **Solution:** The soluble substances present in the rocks are removed by the continuous action of water leading to the formation of holes, and rills or to an increase in the surface roughness of the rock and finally the disintegration and decomposition of rock. The action is considerably increased when the water is acidified by the dissolution of organic and inorganic acids. The relative solubility of the chemical elements is variable, and often being referred to the following order of solubility for the chemical elements involved in the formation of common minerals: $Ca > Na > Mg > K > Si > Al > Fe$.

   (e.g.,) halite (NaCl): $NaCl + H_2O \rightarrow Na^+, Cl^-, H_2O$ (dissolved ions in water)

   The low temperature solutions with low ionic concentration circulate more easily and penetrate through the pores, cleavages, microfractures, and exposed surfaces of the primary minerals, increasing the ionic exchanges that lead to the progressive transformation of the different rock minerals. Subsequent chemical weathering involves processes such as hydration, hydrolysis, oxidation, reduction, and chelation, which produce secondary minerals (mostly clay minerals), which are formed from the primary minerals.
2. **Hydration:** Chemical combination of water molecules with a particular substance or mineral leading to a change in structure. Hydration is the addition of water to a mineral and its adsorption into the crystalline reticulum. Certain minerals are capable of receiving water molecules in their structure, transforming themselves physically and chemically. In hydration, the minerals expand. Dehydration represents the inverse phenomenon, in which the mineral loses water and its volume is reduced. Upon hydration, there is swelling and increase in the volume of minerals. The minerals lose their brightness and become soft. It is one of the most common processes in nature and works with secondary minerals, such as aluminum oxide, iron oxide minerals, and gypsum. For example,

$$\text{(a)}\ \underset{\text{(Hematite) (red)}}{2Fe_2O_3 + 3H_2O} \rightarrow \underset{\text{(Goethite, and/or quasi-amorphous iron hydroxides) (yellow)}}{2Fe_2O_3 . 3H_2O}$$

$$\text{(b)}\ \underset{\text{(Bauxite)}}{Al_2O_3 + 3H_2O} \rightarrow \underset{\text{(Aluminium hydroxide)}}{Al_2O_3 . 3H_2O}$$

$$\text{(c)}\ \underset{\text{(Anhydrite)}}{CaSO_4 + 2H_2O} \rightarrow \underset{\text{(Gypsum)}}{CaSO_4 . 2H_2O}$$

$$\text{(d)}\ \underset{\text{(Olivine)}}{3(MgO . FeO . SiO_2) + 2H_2O} \rightarrow \underset{\text{(Serpentine)}}{3MgO . 2SiO_2 . 2H_2O + SiO_2 + 3H_2O}$$

3. **Hydrolysis:** The most important process in chemical weathering. This is the main process of weathering minerals, aluminum silicates, such as feldspar, majority constituent of granitoids. The hydrolysis corresponds to the reaction between the H^+ and OH^- ions of the water and the elements or ions of the mineral forming the product silicic acid ($H_4\ SiO_4$). If aluminum is available in the decomposed mineral and if favorable environmental conditions exist, a clay mineral is formed. The hydrolysis of the potassium feldspars, namely orthoclase, can be represented by the equation:

$$\underset{\text{(Orthoclase)}}{2K(Al, Si_3)O_8} + 2H^+ + 9H_2O \rightarrow \underset{\text{(Kaolinite)}}{Al_2Si_2O_5(OH)_4} + 4H_4SiO_4 + 2K^+$$

Ions released by hydrolysis during surface weathering of crystalline rocks may follow different pathways (Gomes 1988, 2002): (1) some are removed by the runoff of meteoric waters along the topographic slopes; (2) others participate in the structures of the neoformed supergene minerals (essentially clay minerals), being retained in the residual products of the alteration, forming the soils. Many other equations can be added to the example given, which are

Chemical Weathering, Fig. 1 An example of kaolinitization of feldspars in granite in the central region of Portugal (Photo by I. Duarte)

Chemical Weathering, Fig. 2 The formation of caves in limestone which is rich in calcite, a very soluble mineral, is a common phenomenon in regions where there is plenty of water in circulation. An example of carbonation, located in SW Angola (Photo by I. Duarte)

found in several research papers. In fact, it is by hydrolysis that some minerals are formed such as kaolinite, $Al_2\ Si_2\ O_5\ (OH)_4$; illite, $K\ Al_3\ Si_3\ O_{10}\ (OH)_2$; and montmorillonite, $3Na_{0,66}\ Al_{2,66}\ Si_{3,33}\ (OH)_2$, among others. The hydrolysis of silicates (feldspars of various types, micas, pyroxenes, amphiboles, olivines, etc.) is manifested by phenomena of argillification, kaolinitization (Fig. 1), montmorillonitization, chloritization, saussuritization, and serpentinization (Aires-Barros and Miranda 1989), which are present in the alteration of the rocks

4. **Carbonation:** Carbon dioxide when dissolved in rainwater or in moist air forms carbonic acid, and this acid reacts with rock-forming minerals and brings them into solution. The carbonated water has an etching effect upon some rocks, especially limestone, rich in calcite, mineral particularly vulnerable to carbonation. This type of weathering is important in the formation of caves (Fig. 2). On the other hand, the removal of cement that keeps sand particles together leads to their disintegration. The reactions are as follows:

$$CO_2 + H_2O \rightarrow\ H_2CO_3$$
$$\text{Carbon dioxide} + \text{Water} \rightarrow \text{Carbonic acid}$$

$$H_2CO_3\ +\ CaCO_3 \rightarrow Ca(HCO_3)_2\text{Carbonic acid}$$
$$+\ \text{Calcium carbonate (calcite) slightly soluble}$$
$$\rightarrow \text{Calcium bicarbonate (readily soluble)}$$

5. **Oxidation:** The process of addition and combination of oxygen to minerals. Absorption is usually from O_2 dissolved in soil water and that present in atmosphere. Oxidation is more active in the presence of moisture and

Chemical Weathering, Fig. 3 An example of oxidation of granites located in central region of Angola. The contrast between sound rock (light *gray*) and weathered rock with the formation of iron oxides (*reddish*) is evident (Photo by I. Duarte)

results in hydrated oxides and minerals containing Fe and Mg. When a rock is oxidized, it is weakened and crumbles easily, allowing the rock to break down. Iron oxide is brownish-red in color, and this explains why some rocks look rusty (Fig. 3). Examples of reactions:

$$4FeO\ (\text{Ferrous oxide}) + O_2 \rightarrow 2Fe_2O_3(\text{Ferric oxide})$$

$$4Fe_3O_4(\text{Magnetite}) + O_2 \rightarrow 6Fe_2O_3(\text{Hematite})$$

$$2Fe_2O_3(\text{Hematite}) + 3H_2O \rightarrow 2Fe_2O_3 . 3H_2O\ (\text{Goethite, and/or quasi} - \text{amorphous iron hydroxides})$$

6. **Reduction:** The process of oxygen removal is the reverse of oxidation, being equally important in changing the soil color to gray, blue, or green as ferric iron is converted to ferrous iron compounds. Under the conditions of excess water or waterlogged conditions (less or no oxygen), reduction takes place.

$$2Fe_2O_3(\text{Hematite}) - O_2 \rightarrow 4FeO\ (\text{Ferrous oxide} - \text{reduced form})$$

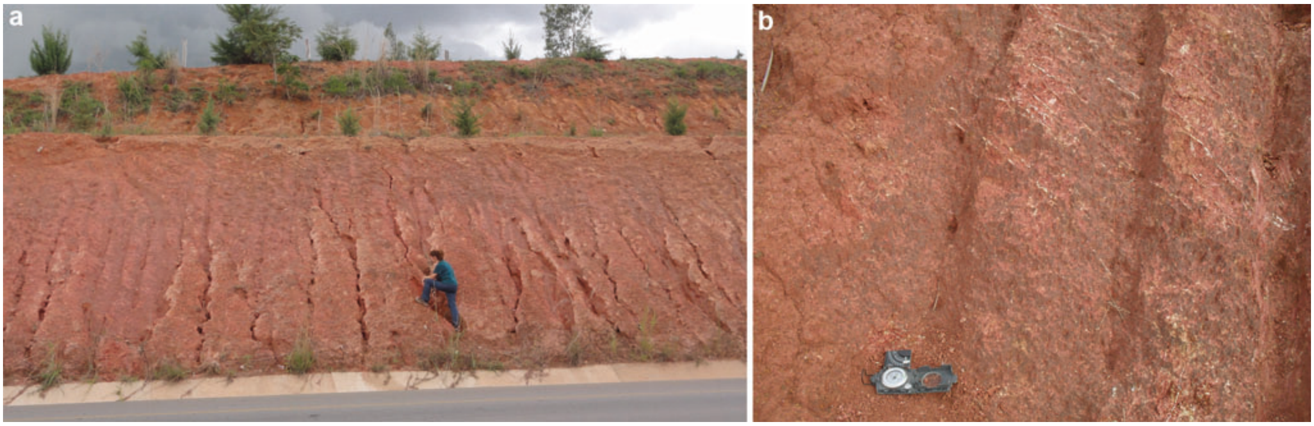
7. **Chelation:** The chemical removal of metallic ions from a mineral or rock by weathering can provide their combination with organic compounds. The decomposition of dead plants in soil may form organic acids which, when dissolved in water, cause chemical weathering. Extreme release of chelating compounds can easily affect surrounding rocks and soils, and may lead to podsolization of soils. It is considered also the most common form of biological weathering, the release of chelating compounds and of acidifying molecules (organic acids) by plants so as to break down aluminum and iron containing compounds in the soils beneath them. This hot chemical weathering is attributed to organic chelating agents (e.g., peptides and sugars) that extract metal ions from minerals and rocks.

Products and Applications

Hydrolysis of silicates is perhaps the most important process of chemical weathering but associated certainly with biological weathering (Formoso 2006). The silicate bearing rocks are leached by rain, and surface waters dissolve and remove the more mobilized chemical species forming the silicates, particularly alkali and earth alkali elements. Some species being less mobilized than others could become concentrated at the top horizons of the weathering profile.

Chemical weathering is not only a destructive process of minerals and rocks. Indeed, it is a creative process too, producing the soil, source, and support of life. When chemical weathering is intensive and lasting, the residual concentration of less mobile chemical species (e.g., Al, Fe, Ti, P) could provide the ultimate formation of mineral deposits of paramount industrial and economic importance, such as kaolin, laterite, and bauxite, all resulting from the concentration of neoformed supergene minerals of fine grain size: hydrous phyllosilicates (kaolinite and halloysite) in kaolin, iron oxides-hydroxides (hematite and goethite) in laterite, and aluminum oxides-hydroxides (gibbsite, boehmite) in bauxite.

The supergene alteration is dependent upon climate (rainfall and temperature), topography, rock nature and characteristics (fissural systems, texture, and composition), soil microorganisms, and drainage conditions.



Chemical Weathering, Fig. 4 (**a**) Laterite in central region of Angola, where granitoid rocks are intensely weathered due to the wet subtropical climate, resulting in the formation of a *reddish* or *brick* colored and hard residual soil; (**b**) a detail of the laterite structure (Photos by I. Duarte)

Kaolin is a white or whitish colored unconsolidated rock in which clay minerals classified into the kaolinite group are significantly represented, and their concentration could be increased through adequate technological processes (refining and beneficiation) providing distinct commercial kaolin grades, addressed to ceramics, paper, cement, rubber, paint, refractories, pharmaceuticals, and several other applications (Murray 1993). According to Ross and Kerr (1930), the name kaolin is a corruption of the Chinese term *kauling* meaning *high ridge*, the name of a hill near Jauchou Fu, in China, where the material was obtained for the manufacture of whiteware centuries ago. Also, according to the same authors, Johnson and Blake (1867) appear to have first clearly intended the name kaolinite for the "*mineral of kaolin.*" Kaolin is derived from the alteration of feldspar-rich rocks, for instance, granite, granodiorite, gneiss, anorthosite, and arkose. Genetically, there are two types of kaolin deposits: residual and sedimentary, the second one mainly resulting from the dismantling, remobilization, and redeposition of the first one. Frequently, the formation of residual kaolin deposits could be the result of weathering/hydrothermal processes.

Laterite is a reddish or brick colored and hard residual soil (Fig. 4), and is a product of intense weathering of iron-containing parent rocks, commonly considered to have formed in hot and wet tropical and subtropical climate conditions that could show scoriaceous and pisolitic structures. It is rich in secondary iron oxides/hydroxides (hematite and goethite) associated to kaolinite minerals and quartz, and in which the bases (alkali and earth-alkali elements) are almost absent. The term laterite was coined by the British East India Company's surgeon Francis Buchanan in 1807, during a reconnaissance trip through places in Angadipuram, Kerala, and Karnataka of Western India, to a soft reddish rock that could be cut to produce bricks which after drying in the sun would harden irreversibly. In general, laterite deposits were formed in Lower Cretaceous and Lower Tertiary, and huge occurrences exist in southern Asia and central Africa. Some laterite deposits developed on granite, rhyolite, carbonatite, and on some metamorphic rocks can be an important source of (rare earth elements) REE which are enriched in supergene zones near the top of laterites (Cocker 2014). The same happens in some bauxite deposits. Laterite is commonly used for house and monument building and for road building too. Also, it can be used as iron, nickel, and aluminum ores. Aleva (1994) produced an excellent compilation of concepts, geology, morphology, and chemistry of laterites.

Bauxite is a white, gray, or reddish residual lateritic soil enriched in free aluminum hydroxides, such as gibbsite, boehmite, and diaspore, under very intense chemical weathering that could be an aluminum ore when Al content of the raw material is over 15–20%, content that could be much increased if the raw material is purified using the Bayer process. First discovered in Les Beaux, in southern France, by Pierre Berthier in 1847, it is mostly found and extracted in a belt around the tropics, with major occurrences in Jamaica, Surinam, Northeast Brazil, and countries of Southern Asia, Russia, and Australia (Bardossy and Aleva 1990; Meyer 2004).

Summary

Chemical weathering of rocks is the most important geological process for soil formation and consists of an adjustment of rock-forming minerals to the prevailing conditions near the Earth's surface by several chemical processes (solution, hydration, hydrolysis, carbonation, oxidation, reduction, and chelation) that are ruled by the chemical action of water with dissolved substances. Chemical reactions can become more effective when the water is slightly acidic or/and when the surface area of the rock increases, leading to mineral transformation and chemical decomposition of rock, thereby

contributing to the decay of rock. The products resulting from the various phases of weathering are essentially composed of secondary minerals and more resistant primary minerals, the residual soils. Chemical weathering and physical weathering work together; however, the predominance of each one depends on the climatic conditions, geomorphology, lithology, and the presence of vegetation.

Cross-References

- ▶ Acid Mine Drainage
- ▶ Acidity (pH)
- ▶ Alkali-Silica Reaction
- ▶ Alteration
- ▶ Biological Weathering
- ▶ Classification of Rocks
- ▶ Classification of Soils
- ▶ Clay
- ▶ Climate Change
- ▶ Desert Environments
- ▶ Diagenesis
- ▶ Dissolution
- ▶ Durability
- ▶ Environments
- ▶ Erosion
- ▶ Evaporites
- ▶ Geochemistry
- ▶ Groundwater
- ▶ Hydrothermal Alteration
- ▶ Limestone
- ▶ Mineralization
- ▶ Physical Weathering
- ▶ Residual Soils
- ▶ Rock Properties
- ▶ Saline Soils
- ▶ Sediments
- ▶ Soil Properties
- ▶ Tropical Environments
- ▶ Water

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Chézy Formula

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Definition

A theoretical relationship that relates mean velocity of flowing water to key channel parameters.

The Chézy formula was developed in about 1775 by French engineer Antoine de Chézy. The mean velocity of water ($\bar{v}$) is related to *hydraulic radius* (R), slope of the channel taken to be the slope of the *energy grade line* (S_e), and a parameter called the Chézy coefficient (C):

$$\bar{v} = C\sqrt{RS_e} \tag{1}$$

where $\bar{v}$ is in m/s, R is in m (*cross-section area*, A, divided by *wetted perimeter*, P), and S_e is in m/m, resulting in C, a term related to *friction losses*, having the units of $m^{1/2}$/s. Expressions describing friction losses are applicable to *uniform flow* conditions but have been used for estimating energy losses in gradually varying nonuniform flow. Friction losses are governed by the dimensionless *Reynolds number* (Re), which is the mean velocity times the hydraulic radius divided by the *kinematic viscosity* of water (ν).

$$\mathrm{Re} = \frac{\bar{v}R}{\nu} \tag{2}$$

where ν is taken to be 1×10^{-6} m^2/s. For flow conditions with Re > 5000, the flow is completely turbulent, which would be of general interest in engineering geology applications involving natural *river and stream channels*. The Darcy-Weisbach energy loss (h_f) formula is applicable for *turbulent flow* and relates a roughness coefficient (f), a length (L), mean flow velocity, the acceleration of gravity (g), and hydraulic radius.

$$h_f = \frac{f L \bar{v}^2}{8 g R} \tag{3}$$

where h_f is dimensionless (dimensionally equivalent to m with length L in m), f is dimensionless, g is 9.807 m/s^2, and other parameters are as defined for Eq. 1. In *laminar flow* conditions, f is K_0/Re, with K_0 equal to 24 for smooth surfaces, 400 for *gravel* surfaces, and as much as 40,000 for surfaces covered with tall, tough grass common to savanna ecosystems (Kruger 2006).

For completely turbulent flow (Re > 5000), the complete theoretical expression for mean velocity is

$$\bar{v} = 5.75\sqrt{gRS_e}\log\left(\frac{12R}{k_s + \frac{3.3\nu}{\sqrt{gRS_e}}}\right) \tag{4}$$

where k_s is an absolute roughness coefficient representing the size of irregularities on the *channel bed and banks* in m. Since ν is very small compared to k_s, Eq. 4 can be simplified to the Chézy formula (Eq. 1)

$$\bar{v} = 5.75\sqrt{gRS_e}\log\left(\frac{12R}{k_s}\right) = C\sqrt{RS_e} \tag{5}$$

where

$$C = 5.75\sqrt{g}\log\left(\frac{12R}{k_s}\right) = 18\log\left(\frac{12R}{k_s}\right) \tag{6}$$

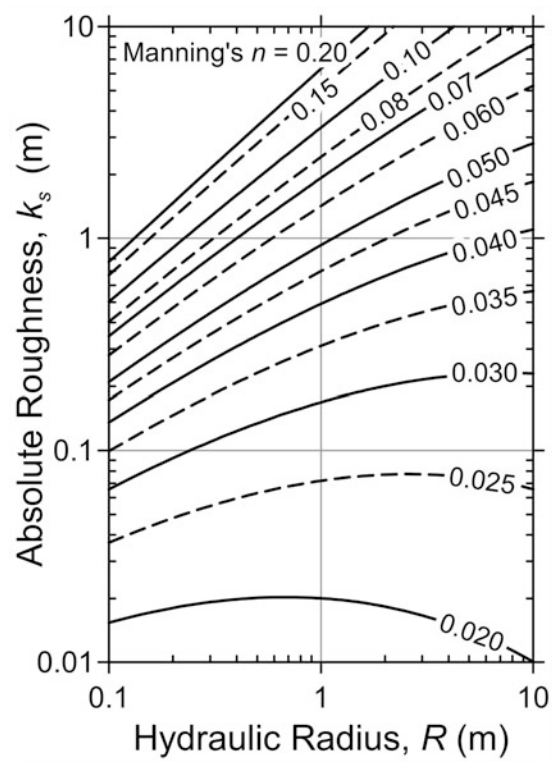
It should be noted that the empirical *Manning formula* also applies to channels:

$$\bar{v} = \frac{1}{n} R^{2/3} S_e^{1/2} = \frac{R^{1/6}}{n}\sqrt{RS_e} \tag{7}$$

where Manning's n has the units of s/m$^{1/3}$. Thus, based on the Manning equation, the Chézy coefficient from Eq. 5 is

$$C = \frac{R^{1/6}}{n};\; n = \frac{R^{1/6}}{18\log\left(\frac{12R}{k_s}\right)} \tag{8}$$

Comparing these formulas reveals that Chézy's C coefficient and Manning's n value are not constants for a particular stream channel, but vary with hydraulic radius (R) and channel irregularity or roughness (Fig. 1).



Chézy Formula, Fig. 1 Manning's n value plotted against absolute roughness and hydraulic radius

Cross-References

- ▶ Current Action
- ▶ Erosion
- ▶ Floods
- ▶ Fluvial Environments

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Classification of Rocks

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Definition

Rocks are naturally formed aggregations of mineral matter. A mineral is a solid, inorganic, crystalline substance with a definite chemical composition and atomic structure (Klein and Hurlbut 1998). Some rocks may also contain non-mineral materials, such as fossils and glass. Rocks are an essential part of the Earth's crust. They remain intact in water and cannot be excavated without blasting (West 2010). Rocks are important for design and stability of engineering structures, and classification of rocks provides an adequate means for predicting and communicating their properties. Several classifications of rocks are available, some based on texture and mineral composition and others on origin.

Introduction

Rocks are the natural building blocks of the Earth. Rocks form by crystallization of magma and lava deposition of sediment carried by rivers into a body of water, precipitation of dissolved minerals (calcite, dolomite, salt), and alteration of existing rocks under the action of high temperature and pressure. All rocks formed below the surface become exposed at the surface by tectonic uplift followed by removal of overburden materials by weathering and erosion. Rocks are continuous, polycrystalline solids, consisting of mineral grains within the framework of discontinuities. Rock properties are evaluated and described using hand specimens or tested in the laboratory. A hand lens or microscope can be used to examine the crystalline grains and microstructure of the rocks. Geological classification of rocks, based on their mineral content, texture, and origin, is essential for all engineering geology-related studies.

Geological Classification

Mineral composition and texture are the primary bases for geologically classifying rocks. A geological classification of rocks may provide information regarding the physical and chemical interactions between the mineral grains and their weathering pattern and weathering product. Detailed geological classifications are widely available in any textbook on petrology (Raymond 2002). Rocks are divided into three primary groups according to their origin: igneous, sedimentary, and metamorphic.

Igneous Rocks

Igneous rocks form by solidification of magma (molten rock material below the Earth's surface) or lava (molten rock material, ejected from volcanoes onto the Earth's surface). Magma originates in the asthenosphere (at a depth range from about 100 to 250 km) or above subducting lithosphere (crust and mantle to a depth of about 100 km). The term "igneous" comes from a Latin word "ignis" meaning fire, as igneous rocks are associated with volcanic and magmatic activities.

Classification of Igneous Rocks

Igneous rocks are classified on the basis of three parameters: color, mineral composition, and texture (size, shape, and arrangement of grains) (Winter 2010). The variation in color, mineral composition, and texture depends on the origin and chemical character of the magmas. Based on the color difference, igneous rocks can be either *mafic* or *felsic* (Table 1). *Mafic* rocks, such as gabbro and basalt, are composed primarily of dark-colored minerals, whereas felsic rocks, such as granite and rhyolite, contain light-colored minerals. With fractional crystallization of magma during the cooling process, *felsic* rocks like granite and rhyolite form first. *Intermediate* rocks, representing a transition from mafic to felsic rocks, form next and include diorite and andesite. Rocks with very dark-colored minerals are called *ultramafic* rocks, for example, peridotite and pyroxenite. Igneous rocks are also classified based on their mineral chemistry. Magmas with silica (SiO_4^{-4}) content above 75% produce minerals like potassium feldspars and quartz (light colored) and the resulting rocks are felsic, whereas magmas with less than 50% silica content produce minerals like amphibole, pyroxene, and olivine (dark colored) and the resulting rocks are mafic.

Classification of Rocks, Table 1 Classification of igneous rocks

Chemical composition		Felsic	Intermediate	Mafic	Ultramafic
Texture	Phaneritic (coarse-grained)	Granite	Diorite	Gabbro	Peridotite
	Aphanitic (fine-grained)	Rhyolite	Andesite	Basalt	Komatiite (rare)
	Porphyritic	"Porphyritic" precedes any of the above names whenever there are appreciable phenocrysts			Uncommon
	Glassy	Obsidian			
	Vesicular	Pumice and scoria			
	Pyroclastic	Tuff (fragments less than 2 mm) volcanic breccia (fragments greater than 2 mm)			

Classification of Rocks, Table 2 Classification of sedimentary rocks

Texture	Grain size	Composition	Comments	Rock name
Clastic	Pebbles, cobbles, and/or boulders embedded in sand, silt, and/or clay	Mostly quartz, feldspar, and clay minerals; may contain fragments of other rocks and minerals	Rounded fragments	Conglomerate
	Sand		Angular fragments	Breccia
			Fine to coarse	Sandstone
	Silt		Very fine grain	Siltstone
	Clay		Compact; may split easily	Shale
Evaporites	Fine to coarse crystals	Halite	Crystals from chemical precipitates and evaporites	Rock salt
		Gypsum		Rock gypsum
		Dolomite		Dolostone
Chemical/ biochemical	Microscopic to very coarse	Calcite	Precipitates of biologic origin or cemented shell fragments	Limestone

When magma cools slowly inside the Earth, the rocks formed are called *intrusive or plutonic rocks. Extrusive or volcanic* rocks form when lava from volcanic eruptions cools rapidly on the Earth's surface. Very rapid cooling can result in glassy texture where no minerals can be identified. *Intrusive* rocks exhibit *phaneritic* texture, consisting of coarse crystals (1/2 mm to a few cm), visible without the aid of a hand lens. *Extrusive* rocks exhibit *aphanitic* texture where only small crystals (about 1/2 mm) can be identified using a hand lens. *Porphyritic*-textured rocks are made up of two crystal sizes with the larger size referred to as phenocryst and the finer size referred to as the groundmass. These rocks form in two stages of magmatic cooling: one at depth where the larger phenocrysts form and the other near the Earth surface where the groundmass crystallizes. Another common igneous texture includes *vesicular* texture, where cavities (vesicles) result from removal of trapped gas bubbles after volcanic eruptions. Common examples include *pumice* and s*coria*. Additionally volcano-generated *pyroclastic* materials like pyroclastic breccia, lapilli, tuff, and ash are also common during violent volcanic eruptions. Table 1 shows the classification of igneous rocks.

Sedimentary Rocks

Sedimentary rocks, comprising about 75% of the rocks exposed on the Earth's surface, form by deposition of Earth materials in a body of water. The deposited material may be organic (plant and animal remains), inorganic (formed by chemical decomposition), or weathered and eroded fragments (also known as *clastic fragments*) of any preexisting rocks. Over time, the deposited sediment changes to sedimentary rock through the process of lithification (compaction, cementation, and crystallization).

Classification of Sedimentary Rocks

Sedimentary rocks are classified as *clastic* (lithification of broken rock fragments of varying sizes) and *chemical/ biochemical* (precipitation and crystallization of dissolved material (Tucker 2001) (Table 2). Clastic rocks are subdivided on the basis of clast size and shape, which are indicators of source, mode of transportation, and depositional environments. A rock dominated by clasts greater than 2 mm in size and angular in shape is called *breccia* – a product of mass wasting, indicative of source not far from environment of deposition. If the clasts are subrounded or rounded, the rock is called *conglomerate*, deposited in marine (sea), glacial, or fluvial (stream) environments. A rock composed of sand-sized grains, less than 2 mm but greater than 1/16 mm, is *sandstone*, deposited in fluvial, lacustrine (lake), marine, or desert environment. Rocks with very fine grains, less than 1/16 mm, are collectively known as *mudrocks* or *argillaceous rocks*. Most mudrocks form in marine or lacustrine areas, because these depositional environments provide non-turbulent waters necessary for deposition. In this category, the clay percentage determines the rock. *Siltstone* is a fine-grained rock with <33% clay and has a gritty texture; *mudstone* contains 33–66% clay, and clay stone contains >66% clay and has a smooth texture. Table 2 summarizes the classification of sedimentary rocks.

Rocks can disintegrate into their chemical components and then can get precipitated by physical or biological process leading to chemical or biochemical sedimentary rocks, respectively. *Limestones* are common chemical sedimentary rocks formed in shallow to deep marine environments by carbonate (calcium-rich carbonate is called calcite)-secreting organisms. *Dolostones* are a variation of *limestones*, where calcite changes to magnesium-rich dolomite by diagenetic conversion. *Evaporites* are rocks formed from minerals like gypsum and halite, precipitated from solution during evaporation. *Cherts* are microcrystalline silica that can form chemically by movement of silica-rich groundwater, or biochemically from shells of silica-rich organisms which can dissolve and recrystallize, forming chert nodules or layers.

Metamorphic Rocks

The term "metamorphic" arises from the word "metamorphism" or "change in form" of an existing rock to a new and changed rock. Metamorphism of existing rocks occurs due to the action of high pressure, temperature, and chemically active fluids, referred to as the agents of metamorphism. There are two types of metamorphism: contact metamorphism and regional metamorphism. Contact metamorphism occurs in the vicinity of igneous intrusions, whereas regional metamorphism occurs over large areas where a subducting plate is subjected to increasing temperature and pressure as it plunges deeper into the earth. The original rock which undergoes metamorphism is called *protolith*. The increased temperature changes the rock's chemical composition through formation of new minerals and assists in crystal growth. The pressure from the overlying rocks, referred to as the *lithostatic* pressure, and the *directed* pressure from plate motion cause changes in the rock's texture (Winter 2010).

Classification of Metamorphic rocks

Like igneous and sedimentary rocks, classification of metamorphic rocks depends on texture and mineral assemblage. On the basis of texture, metamorphic rocks are classified as *foliated* and *non-foliated* (Table 3). *Foliation* is caused primarily by a parallel orientation of platy minerals like micas, needle-shaped minerals (hornblende), and tabular minerals (feldspar). Foliated metamorphic rocks include *slate*, *phyllite*, *schist, and gneiss*. With an increasing degree of metamorphism, the sizes of mineral grains gradually increase from very fine-grained *slate*, to fine-grained *phyllite*, coarse-grained *schist*, and very coarse-grained gneiss. On the basis of presence of abundant minerals, prefixes are used to name metamorphic rocks. For example, *schist* containing muscovite and garnet is called *muscovite–garnet schist*, or gneiss containing hornblende and biotite is called *hornblende–biotite gneiss*. The *non-foliated* metamorphic rocks are composed of minerals that are not elongated but are mostly equidimensional in shape, like quartz and calcite. Common *non-foliated* metamorphic rocks include *quartzite* and *marble*. If the *non-foliated* rock is very fine grained, where individual minerals are not recognized, the rock is called *hornfels*.

Classification of Rocks, Table 3 Classification of metamorphic rocks

Rock name		Texture	Grain Size	Protolith
Slate	Metamorphism increasing ↓	Foliated	Very fine	Shale, mudstone, or siltstone
Phyllite			Fine	Slate
Schist			Medium to coarse	Phyllite
Gneiss			Medium to coarse	Schist, granite, or volcanic rocks
Marble		Non-foliated	Medium to coarse	Limestone, dolostone
Quartzite			Medium to coarse	Quartz sandstone

Engineering Significance of Rock Classification

A classification of rocks based on mineral composition and texture provides important information on the rock's physical properties and engineering behavior (Tugrul and Zarif 1999). Coarse-grained igneous rocks are generally lower in strength and hardness than fine-grained igneous rocks, thus less preferred in engineering practice. On the other hand, volcanic rocks and *pyroclastic* materials can exhibit varying degrees of anisotropy and fracturing (West 2010). Additionally, silica-rich igneous rocks like *volcanic glass*, *pyroclastic* material, *rhyolite*, and *andesite* can result in alkali–silica reaction when used in portland cement concrete. The alkali–silica reaction is a chemical reaction that occurs where high-alkali cement reacts with the noncrystalline or fine-grained silica present in igneous rocks. The reaction product, alkali–silica gel, expands on water absorption, causing concrete to crack (West 2010).

Sedimentary rocks are very diverse in nature and, consequently, their engineering behavior is extremely variable. *Cherts* can be problematic when used as concrete aggregates. Due to the high porosity of weathered *chert*, they come out of concrete that undergoes freezing. Moreover some *cherts* respond to an alkali–silica reaction. For the same reason, *siltstone*, *shale*, *quartz sandstone*, and *conglomerate* are generally not acceptable aggregate materials for construction, whereas *limestone* and *dolostone* often make very good aggregates. *Shales* and *siltstones* provide good foundations for buildings, dams, and bridges. Sinkholes, solution channels, and underground tunnels in *limestone* and *dolostone* can pose great challenges in foundations of civil structures and must be properly handled. Slaking (disintegration from weathering) can result in slope instability and subsidence in *shales* when used as rock fills in highway embankments.

Non-foliated metamorphic rocks produce more predictable behavior, whereas *foliated* metamorphic rocks exhibit directional anisotropy, causing strength, hardness, and permeability to vary with respect to rock foliation. Caution should be taken to avoid load transfer from bridges, dams, and building foundations in a direction parallel to the foliation. *Non-foliated* rocks like *marble*, when fractured, are subject to cavities and channels like *limestone* and show similar problems. *Quartzites* are massive and very resistant hard rock and can damage crushing and sizing equipment. *Foliated* metamorphic rocks commonly produce rock pieces that are elongated in shape when crushed, causing mixing problems in fresh concrete. *Schist* and *gneiss* can flake from freeze–thaw and wetting–drying effects and are not recommended as aggregates because of the presence of abundant mica. Rock slides commonly occur in *foliated* rocks when foliation planes dip steeply into the slopes.

Conclusion

Based on origin, there are three groups of rocks: igneous, sedimentary, and metamorphic. The classifications of all three groups of rock are based on texture and mineral composition. For design and construction of engineering structures, properties of both intact rock and rock mass are evaluated. Voluminous research has been conducted on relating petrographic characteristics (texture and mineral composition) of rocks to their engineering properties. Thus, a classification of rocks, based on texture and mineral composition, can be particularly useful in predicting the engineering behavior of intact rock. In addition to the rock classification, site-specific understanding of the regional history, structure, and stratigraphy allows for optimal engineering investigation.

Cross-References

▶ Abrasion
▶ Aggregate
▶ Aggregate Tests
▶ Alkali-Silica Reaction
▶ Alteration
▶ Armor Stone
▶ Bedrock
▶ Blasting
▶ Building Stone
▶ Cap Rock
▶ Concrete
▶ Crushed Rock
▶ Dissolution
▶ Durability
▶ Erosion
▶ Evaporites
▶ Gabions
▶ Geochemistry
▶ Igneous Rocks
▶ International Society for Rock Mechanics (ISRM)
▶ Limestone
▶ Mechanical Properties
▶ Metamorphic Rocks
▶ Mineralization
▶ Petrographic Analysis
▶ Rock Bolts
▶ Rock Coasts
▶ Rock Field Tests
▶ Rock Laboratory Tests
▶ Rock Mass Classification
▶ Rock Mechanics
▶ Rock Properties
▶ Sedimentary Rocks
▶ Sequence Stratigraphy

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Classification of Soils

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Synonyms

Characterization of soils; Description of soils; Properties of soils; Systems for soil description

Definition

An unconsolidated natural set of solid mineral particles that result from physical disintegration and chemical decomposition of rocks, which may contain organic matter and voids between the particles, isolated or linked, which may contain water and/or air.

Introduction

Classification of soils consists of the division of soils into classes based on their genetic, textural, chemical, mineralogical, physical, or geotechnical characteristics. The nature of the parent rock influences the composition of the resulting soil. The weathering processes and type and amount of transport before deposition, as in the case of sedimentary soils, affect the structure of the soils and their engineering properties.

Soil has several meanings according to the professional perspective of the person who defines it. The main purpose of the systems for soil classification is to group different types of soil into classes having similar characteristics to thereby provide a systematic method to describe the soil. Many geologists consider that a soil classification based only on particle size distribution is sufficient but the engineering geologist requires a classification relevant to engineering applications. Soil classification for engineering purposes should involve simple index properties of soils, which can be easily accessed, such as particle size distribution and plasticity.

Soils and soil masses occupy a large part of the Earth's surface, such as submerged regions, coastal regions, or in the valleys of the great rivers, where they can reach significant thicknesses. Engineering soils are important because they constitute one of the main types of building materials and because, mainly in coastal regions where there is a tendency for a greater concentration of population and large urban areas, the majority of the civil engineering structures are founded on soil masses which influence foundation design.

The type of soil and its evolution depends on the rate of weathering and the nature of the parent rock. It is influenced by several factors, such as the grain size and mineral composition of the parent rock, the temperature during the weathering processes, and the presence of water.

Soil can contain the three phases of matter: solid, liquid, and gas. The solid phase is usually the mixture, in varying proportions, of mineral particles resulting from the weathering of rocks and, when present, by solid particles of organic material, of very variable dimensions, commonly vegetable material (humus). The voids between the solid particles can be occupied either by water, the liquid phase, and/or by air, the gas phase. When the voids are completely filled by water the soil is said to be saturated. When the voids are only filled by air, the soil is said to be dry.

The inter-relationships between the volumes and weights of the three phases of a soil define the fundamental physical properties, such as the void ratio, the porosity, the bulk density, the dry density, the specific gravity, the water content, and the degree of saturation, thus contributing to the definition of the engineering properties of a soil (Bell 2000).

The most common classification systems of soils group them in an orderly and systematic way, into classes, with similar physical properties that can be easily identified. The criteria generally used in soil classifications are of three main types: (a) the type and dimensions of soil particles; (b) the origin of the soil; (c) applications of the soil for engineering purposes.

The first criterion divides soils according to the dimensions of particles (clay, silt, sand and gravel, cobbles, and boulders). In the case of granular soils, the classification is according to compactness, whereas in the case of fine soils, classification is according to consistency. In the second criterion, the soils can be classified as sedimentary or transported soils, when the soils result from the action of the weathering processes on the parent rock, are then transported and deposited at a certain distance away from its origin, or as residual soils, when the soils result from the physical disintegration and chemical decomposition of the parent rock, forming and remaining at the location of the parent rock, and not subjected to any transport and deposition. The third criterion describes the soil in terms of its suitability as building or foundation material to predict its geotechnical behavior in an engineering work.

Soil Classification Systems

Different soils with similar properties may be classified into groups and subgroups according to their engineering behavior. Classification systems provide a common language to concisely express the general characteristics of soils, which are infinitely varied, without detailed descriptions. Currently, two elaborate classifications systems are commonly used by soils engineers. Both systems take into consideration the particle size distribution and Atterberg limits. They are the American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System (USCS).

The behavior of soil during and after construction primarily depends on the properties of the undisturbed soil. Valuable information concerning the general characteristics of a soil can be inferred from its proper classification according to one of the standard systems available to the practitioners. The practitioners use both AASHTO and Unified Soil Classification System (USCS) depending on the specific use in its design and construction operations. AASHTO classification is mostly used for the highway and pavement whereas Unified Soil Classification System is widely used for geotechnical purposes.

Unified Soil Classification System (USCS)

The original form of the Unified Soil Classification System was proposed by Casagrande in 1942 during World War II for use in airfield construction undertaken by the US Army Corps of Engineers. This proposal gave rise to a subsequent publication (Casagrande 1948). At present, it is widely used by engineers (ASTM D-2487, 2011).

According to the USCS classification, soil is divided into: coarse grained soil, fine-grained soil, and highly organic soil. The particle size distribution of soil and consistency limits are used in classification of soils.

The basic idea of this classification relies on marking the soil with symbols that consist of two letters. The exceptions are cases when the soil is marked with double symbols consisting of four letters.

The first letter for the symbol for coarse-grained soil denotes the main type of soil:

G – gravel
S – sand

The second letter in the coarse-grained soil symbol describes characteristics of the main group:

W – well graded sand or gravel
P – poorly graded sand or gravel
M – silty sand or gravel
C – clayey sand or gravel

The first letter in the symbol for fine-grained soil denotes the main type of soil:

M – silt
C – clay
O – organic soil

The second letter in the fine-grained soil symbol describes the characteristics of the main group:

L – low plasticity, lean for clay
H – high plasticity, fat for clay, elastic for silt

Highly organic soil has a two-letter symbol for the main group of soil:

PT – peat

The USCS classification of soil is presented in Table 1. In addition to Table 1, the plasticity diagram presented in Fig. 1 is also used for soil classification.

AASHTO Soil Classification System

Created by Hogentogler and Terzaghi (1929), this was one of the first engineering classification systems. Intended specifically for use in highway construction, it still survives as the American Association of State Highway and Transportation Officials (AASHTO 2012) system. It rates soils for their suitability for support of roadway pavements and is still widely used in such projects.

The AASHTO system uses both grain size distribution and Atterberg limits data to assign a group classification and a group index to the soil. The group classification ranges from A-1 (best soils) to A-8 (worst soils). Group index values near 0 indicate good soils, while values of 20 or more indicate very poor soils. However, a soil that may be "good" for use as a highway subgrade might be "very poor" for other purposes, and vice versa.

The system itself requires only that a portion of soil to pass through a 3-inch sieve. If any material does not pass the 3-inch sieve, its percentage by weight should be recorded and noted with the classification.

Table 2 can be used to determine the group classification. Begin on the left side with A-1-a soils and check each of the criteria. If all have been met, then this is the group classification. If any criterion is not met, move to the right and repeat the process, continuing until all the criteria have been satisfied. Do not begin at the middle of the chart.

The group index can be found by using the following equation:

$$\text{Group Index} = (F - 35)\,[0.2 + 0.005(w_L - 40)] + 0.01(F - 15)(I_P - 10).$$

Where:

F = fines content (expressed as a percentage).
w_L = liquid limit.
I_P = plasticity index.

When evaluating the group index for A-2-6 or A-2-7 soils, use only the second term in the equation. For all soils, express the group index as a whole number. Computed group index values of less than zero should be reported as zero.

Finally, express the AASHTO soil classification as the group classification (A-1 through A-8), followed by the group index in parentheses. For example, a soil with a group classification of A-4 and a group index of 20 will be reported as A-4(20).

Classification of Soils, Table 1 USCS classification of soil according to ASTM D 2487 (2011)

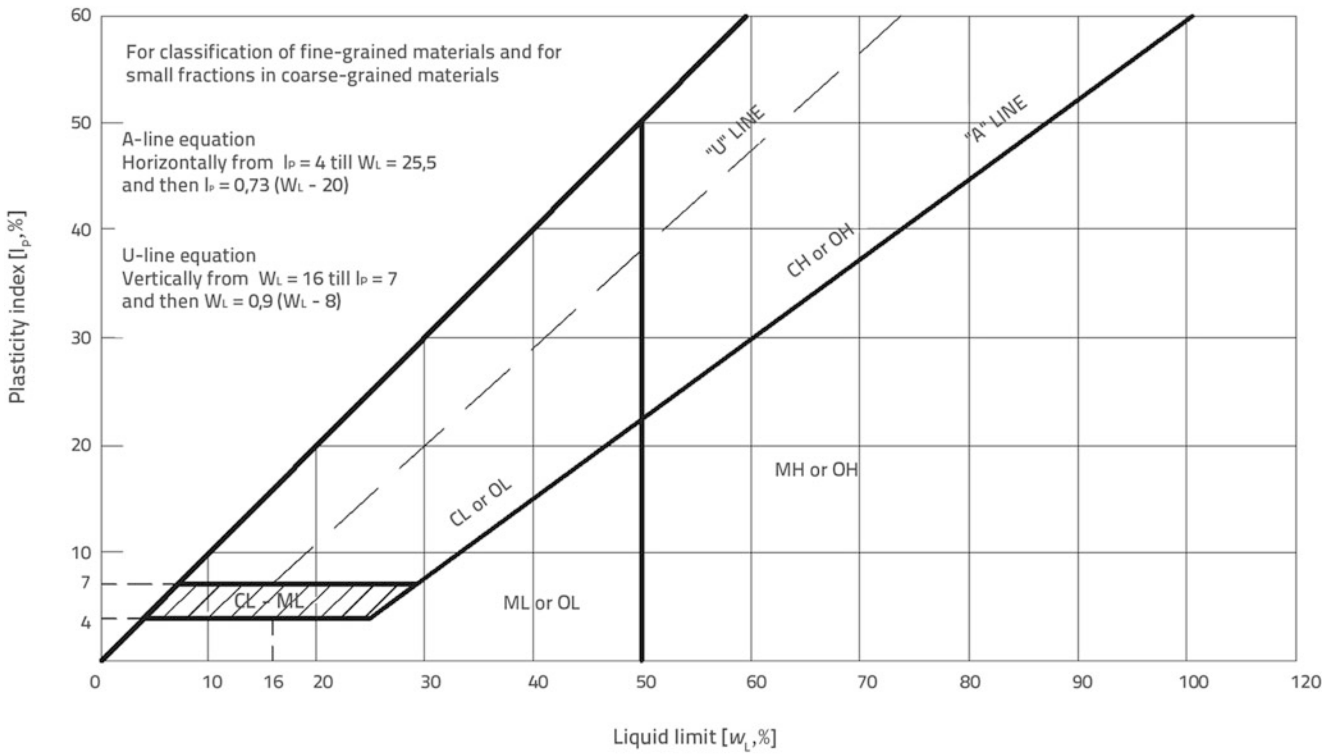
Criteria for allocation of symbols and names to individual soil groups based on laboratory testing [a]				Soil classification	
				Symbol	Group name [b]
Grained Soils (more than 50% remains on sieve No. 200 – 0.075 mm)	Gravel More than 50% retained on the sieve (N° 4–4.75 mm)	Pure gravel (less than 5% of fine grains [e])	$c_u \geq 4$ and $1 \leq c_c \leq 3$ [c]	GW	Well-graded gravel [d]
			$c_u < 4$ and/or $1 > c_c > 3$ [c]	GP	Poorly graded gravel [d]
		Gravel with fine grains (more than 12% of fine grains [e])	Fine grains are classified as ML or MH	GM	Silty gravel [d, f, g]
			Fine grains are classified as CL or CH	GC	Clayey gravel [d, f, g]
	Sand 50% or more grains passing (N° 4–4.75 mm)	Pure sand (less than 5% of fine particles [i])	$c_u \geq 6$ and $1 \leq c_c \leq 3$ [c]	SW	Well-graded sand [h]
			$c_u < 6$ and 7 or $1 > c_c > 3$ [c]	SP	Poorly graded sand [h]
		Sand with fine grains (more than 12% of fine grains [i])	Fine grains are classified as ML or MH	SM	Silty sand [f, g, h]
			Fine grains are classified as CL or CH	SC	Clayey sand [f, g, h]
Fine-Grained Soils (50% or more passing through sieve No. 200 – 0.075 mm)	Silt and clay (liquid limit less than 50%)	Inorganic	PI >7 and at or above A-line [j]	CL	Lean clay [k, l, m]
			PI <4 or below A-line [j]	ML	Silt [k, l, m]
		Organic	(LL – drying in oven) / (LL – without drying in oven) < 0.75	OL	Organic clay [k, l, m, n]
					Organic silt [k, l, m, o]
	Silt and clay (liquid limit in excess of 50%)	Inorganic	PI and at or above A-line [j]	CH	Fat clay [k, l, m]
			PI below A-line [j]	MH	Elastic silt [k, l, m]
		Organic	(LL – drying in oven) / (LL – without drying in oven) < 0.75	OH	Organic clay [k, l, m, p]
					Organic silt [k, l, m, q]
Highly organic soil		Primary organic matter, dark in color, with organic odor		PT	Peat

[a]Based on materials passing through the sieve of 3-in, 75 mm
[b]If soil samples in situ contain pieces or blocks or both, the name of the soil group must be extended with "with pieces" or "with blocks" or "with pieces and blocks"
[c]$c_u = D_{60}/D_{10}$; $c_c = (D_{30})^2/(D_{10} x D_{60})$
[d]If soil contains ≥15% of sand, the name of the soil group must be extended with "with sand."
[e]Gravels with 5 to 12% fine grains get double symbols: *GW-GM* well-graded gravel with silt, *GW-GC* well-graded gravel with clay, *GP-GM* poorly graded gravel with silt, *GP-GC* poorly graded gravel with clay
[f]If fine grains are classified as CL-ML, then double symbols GC-GM or SC-SM should be used
[g]If fine grains are organic, the name of the soil group should be extended by adding "with organic fine grains."
[h]If soil contains ≥15% of gravel, the name of the soil group should be extended by adding "with gravel."
[i]Sand with 5 to 12% of fine grains get double symbols: *SW-SM* well-graded sand with silt, *SW-SC* well-graded sand with clay, *SP-SM* poorly graded sand with silt, *SP-SC* poorly graded sand with clay
[j]If a pair of values (w_L, IP) in the plasticity diagram is situated within the hatched area ($4 < IP < 7$), the soil is designated as CL-ML, as silty clay
[k]If the soil contains 15 to 30% of material above the sieve N° 200–0.075 mm, the name of the soil group should be extended by adding "with sand" or "with gravel," depending on which of these two materials is dominant
[l]If the soil contains ≥30% of material above the sieve N° 200–0.075 mm, and if the sand is dominant, the name of the soil group should be extended by adding "sandy"
[m]If the soil contains ≥30% of material above the sieve N° 200–0.075 mm, and if the gravel is dominant, the name of the soil group should be extended by adding "gravelly"
[n]$I_P \geq 4$ and at or above the A-line
[o]$I_P < 4$ or below the A-line
[p]I_P at or above the A-line
[q]I_P below the A-line

Grouping and Classification of Residual Soils

A number of geological and engineering geological schemes have been used to describe and classify weathered rocks and residual soils for various engineering purposes. However, no system was fully accepted for the description or classification of residual soils. This is due to the diverse nature of residual soils, being unlikely that a universal scheme is either desirable or a practical possibility.

Various attempts have been made to group or classify residual soils, but none are particularly useful. Some, such as that of the Geological Society of London (1990), make use



Classification of Soils, Fig. 1 Plasticity diagram for the USCS classification according to ASTM D 2487 (2011)

Classification of Soils, Table 2 AASHTO classification of highway subgrade of soil

General classification	Granular materials (35% or less passing 0.075 sieve)							Silty-clay materials (more than 35% passing 0.075 sieve)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7-5 A-7-6
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
Sieve analysis, % passing											
2.00 mm (N° 10)	≤50	–	–	–	–	–	–	–	–	–	–
0.425 mm (N°40)	≤30	≤50	≥51	–	–	–	–	–	–	–	–
0.075 mm (N°200)	≤15	≤25	≤10	≤35	≤35	≤35	≤35	≥36	≥36	≥36	≥36
Characteristics of fraction passing 0.425 mm sieve (n° 40): Liquid limit Plasticity index	– 6 max		-NP	≤40 ≤ 10	≥41 ≤ 10	≤40 ≥ 11	≥41 ≥ 11	≤40 ≤ 10	≥41 ≤ 10	≤40 ≥ 11	≥41 ≥ 11
Usual types of constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General subgrade rating	Excellent to good							Fair to poor			

Note (1): Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30 of soil science classifications and are not very suitable for engineering purposes. Terms such as vertisols and andosols are not normally meaningful to engineers, and the variation in properties within these groups is likely to be so large as to make these groups of little relevance.

The amount, wide range, and global quality of data representative of all weathering levels give the possibility of checking available classifications and help to improve them to be useful in design practices.

In spite of the existence of various approaches, based both in petrographic (Lumb 1962, 1965; Country Roads Board 1982) and chemical (Rocha Filho et al. 1985; Irfan 1996) indexes, the fact is that classifications based in mechanical responses are better suited for engineering design approaches. "Geotechnical Engineering – Identification and Description of Rock" (ISO/CEN 14689–2, 2001), approved by International Organization for Standardization (2003), represents rock massifs well, but is not useful in soil masses and quite inefficient in transition massifs where the rock matrix is quite disintegrated. In these situations, it is rather common to apply classifications defined for sedimentary soils (Unified and AASHTO classifications) based on grain size distribution and Atterberg limits. However, these classifications for residual soils frequently lead to erroneous interpretations (Wesley 1988; Vaughan et al. 1988), since these focus primarily on the properties of the soil in its remolded state, thus not considering *in situ* structures inherited from the original rock or developed as consequence of weathering. Considering these problems, Wesley (1988) proposed a classification based on mineralogy, micro and macro-fabric features.

Focusing on two main factors, namely mineralogical composition and structure, provides a basis for dividing residual soils into groups that can be expected to have fairly similar engineering properties. Starting with mineralogy, the following groups can be established:

(a) Soils without a strong mineralogical influence (e.g., those containing low activity clays).

Many residual soils fall into this category, especially those derived from the weathering of sandstones, or igneous rocks such as granite. These soils are likely to be fairly coarse grained with a small clay fraction. Structure is likely to be an important concept in understanding the behavior of these soils. The weathered granite soils of Hong Kong and Malaysia fall into this group.

(b) Soils with a strong mineralogical influence, from "conventional" clay minerals (e.g., those containing high activity clays).

One very important worldwide group comes into this category – the "black cotton" soils or "vertisols," also called Houston Black Clay in Texas, Tropical Black Earths of Australia, "Tirs" of Morocco, etc. The predominant clay mineral is smectite, a group of which montmorillonite is a member. These black cotton soils are highly plastic and highly compressible and have high shrink/swell potential. Structural effects are almost zero with these soils. They normally form in poorly drained areas and have poor engineering properties.

(c) Soils with a strong mineralogical influence, coming from special clay minerals not founded in sedimentary clays.

The two most important clay minerals found only in certain residual soils (especially tropical residual soils of volcanic origin) are halloysite and allophane. These are both silicate clay minerals. Apart from the silicate minerals, tropical soils may contain non-silicate minerals, in particular the hydrated forms of aluminum and iron oxide, gibbsite, and goethite. The most unusual of these minerals, in terms of understanding soil behavior, is allophane.

Soils of Group C which contain these unusual minerals include:

(i) Tropical red clays: the predominant mineral is halloysite but may also contain kaolinite, with gibbsite and goethite. Halloysite particles are generally very small in size but are of low activity. Soils containing halloysite as the predominant mineral generally have good engineering properties. Red clays generally form in well-drained areas in a tropical climate having a wet and dry season. Red clays may be referred to as lateritic soils or as latosols. There is a wide range of engineering properties found in red clays, but they should not be confused with laterite itself.

(ii) Volcanic ash soils (or andosols or andisols): these are found in many tropical and subtropical countries (including New Zealand) and are formed by the weathering of volcanic "glass." The predominant clay mineral is allophane (frequently associated with another mineral called imogolite).

(iii) Laterites: the term laterite is used very loosely, but should refer to deposits in which weathering has reached an advanced stage and has resulted in a concentration of iron and aluminum oxides (the sesquioxides gibbsite and goethite), which act as cementing agents. Laterites therefore tend to consist of hard granules formed by this cementing action; they may range from sandy clay to gravel and are used for road subbases or bases.

Table 3 shows this grouping system for residuals soils, and Table 4 attempts to list some of the more distinctive characteristics of these soil groups and indicates the means by which they may possibly be identified.

Following on from mineralogy, the next characteristic which should be considered is structure, which refers to specific characteristics of the soil in its undisturbed (*in situ*) state. Structure can be divided into two categories:

(a) Macrostructure, or discernible structure: this includes all features discernible to the naked eye, such as layering,

Classification of Soils, Table 3 A classification or "grouping" system for residual soils (Wesley 2009, 2010)

Grouping system		Common pedological names used for groups	Descriptive information on in situ state	
Major division	Subgroup		Parent rock	Information on structure
Group A Soils without a strong mineralogical influence	(a) Strong macrostructure influence	Miscellaneous	Give details of type of rock from which the soil has been derived	Describe nature of structure: – stratification – fractures, fissures, faults, etc. – presence of partially weathered rock (%?)
	(b) Strong microstructure influence	Miscellaneous		Describe nature of microstructure and/or evidence of it: – influence of remolding – sensitivity – liquidity index
	(c) Little or no structural influence	Miscellaneous		Indicate evidence for little or no structural effect
Group B Soils strongly influenced by normal clay minerals	(a) Smectite (montmorillonite) group	Black cotton soils Black soils Tropical black earths Grumusols Vertisols		
	(b) Other clay minerals?	?		?
Group C Soils strongly influenced by clay minerals essentially found only in residual soils	(a) Allophane subgroup	Volcanic ash soils Andosols or andisols Andepts		Give basis for inclusion in this group. Describe any structural influences, either macrostructure or microstructure
	(b) Halloysite subgroup	Tropical red clays Latosols Oxisols Ferralsols		As above
	(c) Sesquioxide subgroup -gibbsite, goethite, haematite	Lateritic soils Laterites Ferralitic soils Duricrusts		Give basis for inclusion in this group. Describe any structural effects - Especially cementation effects or the sesquioxides

discontinuities, fissures, pores, presence of unweathered or partially weathered rock, and other relict structures inherited from the parent rock mass.

(b) Microstructure, or non-discernible structure: this includes fabric, interparticle bonding or cementation, aggregations of particles, micropores, etc. Microstructure is more difficult to identify than macrostructure, although it can be inferred indirectly from other behavioral characteristics such as sensitivity. High sensitivity indicates the presence of some form of bonds between particles which are destroyed by remolding.

This grouping system (Table 4) is intended to help geotechnical engineers find their way around residual soils, and to draw attention to the properties likely to be of most significance for geotechnical engineering. It is not intended to perform a function as a rigorous classification system. Several authors have proposed changes in Wesley's classification in order to make it more applicable to geotechnical purposes. Cruz et al. (2015) proposed the use of the laboratory uniaxial compressive strength (UCS) and *in situ* SPT tests as index parameters for classification purposes.

Summary

Different soils with similar properties may be classified into groups and subgroups according to their engineering behavior. Classification systems are a common language to concisely express the general characteristics of soils, which are infinitely varied. Most of the soil classification systems of transported soils that have been developed for engineering purposes are based on simple index properties including particle-size distribution and plasticity, such as the USCS and AASHTO systems. Although several classification systems are now in use, none is totally definitive of any soil for all possible applications because of the wide diversity of soil properties. In addition, in the case of residual soils, their specific features are not adequately covered by conventional methods of soil classification. In this case, classification systems such as that proposed by Wesley (2009) are based on their mineralogical composition and soil micro and macrostructure. These different classification systems are intended to provide an orderly division of residual soils into groups which belong together because of common factors in their formation and/or compositions, which can be expected to give them similar engineering properties.

Cross-References

► Alteration
► Biological Weathering
► Characterization of Soils
► Chemical Weathering
► Classification of Rocks
► Clay
► Cohesive Soils

Classification of Soils, Table 4 Characteristics of residual soils group (Wesley 2009, 2010)

Group				
Major group	Subgroup	Examples	Means of identification	Comment on likely engineering properties and behavior
Group A Soils without a strong mineralogical influence	(a) Strong macrostructure influence	Highly weathered rocks from acidic or intermediate igneous or sedimentary rocks	Visual inspection	This is a very large group of soils (including the "saprolites") where behavior (especially in slopes) is dominated by the influence of discontinuities, fissures, etc.
	(b) Strong microstructure influence	Completely weathered rock, formed from igneous or sedimentary rocks	Visual inspection, and evaluation of sensitivity, liquidity index, etc.	These soils are essentially homogeneous and form a tidy group much more amenable to rigorous analysis than Group (a) above. Identification of nature and role of bonding (from relic primary bonds to weak secondary bonds) important to understanding behavior.
	(c) Little or no structural influence	Soils formed from very homogeneous rocks	Little or no sensitivity, uniform appearance	This is a relatively minor subgroup. Likely to behave similarly to moderately over-consolidated soils.
Group B Soils strongly influenced by normal clay materials	(a) Smectite (montmorillonite) group	Black cotton soils, and many similar dark colored soils formed in poorly drained conditions	Dark color (gray to black) and high plasticity suggest soils of this group	These are normally problem soils, found in flat and low-lying areas, having low strength, high compressibility, and high swelling and shrinkage characteristics.
	(b) Other clay minerals?			Likely to be a very minor subgroup.
Group C Soils strongly influenced by clay minerals essentially found only in residual soils	(a) Allophane subgroup	Soils weathered from volcanic ash in the wet tropics and temperate climates	Position on plasticity chart, and irreversible changes on dying	Characterized by very high natural water contents and Atterberg limits. Engineering properties generally good, though in some cases high sensitivity may make earthworks difficult.
	(b) Halloysite subgroup	Soils often derived from volcanic material, especially tropical red clays	Reddish color, well drained topography, and volcanic origin are useful indictors	These are generally very fine grained soils of low to medium plasticity, and low activity. Engineering properties generally good. (Note that there is often some overlap between halloysite and allophane clays.)
	(c) Sesquioxide subgroup -gibbsite, goethite, haematite	Laterites, or possibly some red clays referred to as "lateritic" clays	Nonplastic or low plasticity materials, generally of granular, or nodular appearance	This is a very wide poorly defined group, ranging from silty clay to coarse sand and gravel. Behavior ranges from low plasticity silty clay to gravel. These materials are the end products of a very long weathering process.

▶ Collapsible Soils
▶ Consolidation
▶ Desiccation
▶ Erosion
▶ Expansive Soils
▶ Geochemistry
▶ Liquid Limit
▶ Noncohesive Soils
▶ Organic Soils and Peats
▶ Physical Weathering
▶ Plastic Limit
▶ Residual Soils
▶ Saline Soils
▶ Sand
▶ Silt
▶ Soil Field Tests
▶ Soil Laboratory Tests
▶ Soil Mechanics
▶ Soil Nails
▶ Soil Properties

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Clay

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Definition

The term “clay” is applied both to Earth materials with a particle size equal to or less than 0.005 mm and to those minerals that are microcrystalline, layered, hydrous aluminum phyllosilicates, occasionally with variable amounts of iron, magnesium, and alkali metals (Gillott 1968; West 2010). Therefore, cohesive soils may be composed of mixtures of clay minerals and clay-sized materials like quartz, feldspar, and carbonate. Both clay minerals and clay-sized particles are the product of weathering from pre-existing rocks and found on or near the earth surface.

Characteristics

Globally, clay-bearing sediments, also referred to as argillaceous sediments, make up about 60% of the Earth’s surface, with clay minerals comprising up to two-thirds of the components. The atomic structure of clay minerals involves two basic units, tetrahedral silicate sheets (Si^{+4} cation occurs in fourfold and tetrahedral coordination with oxygen) and octahedral hydroxide sheets (Al^{+3} occurs in sixfold or octahedral coordination). A 1:1 clay would consist of one tetrahedral sheet and one octahedral sheet, and examples would be kaolinite and serpentine. A 2:1 clay consists of an octahedral sheet sandwiched between two tetrahedral sheets, and examples are talc, chlorite, vermiculite, and montmorillonite. The consecutive lattices of some clay minerals are joined by hydrous ions and have a great affinity for water. Atterberg limits (liquid limit, plastic limit) are commonly used as a means of estimating the plasticity of fine-grained materials like clay and silt (Casagrande 1940). Plasticity is the critical water content at which material can stay deformed after applied external stress is removed.

Clays are important to engineering geologists because civil structures frequently rest upon clay-rich formations, Earth materials containing clay are used in embankments and landfill linings, and clays are used in pozzolan, brick, and grout. However some clays are considered to be a major hazard in engineering works because they are susceptible to change in volume in response to applied stress, vibration, and changing moisture content (Holtz et al. 2011). For example, expanding clay may consolidate with additional load or expand with the addition of water. As a result, the engineering foundations can settle or heave. Moreover drying of originally moist clay can lead to reduction in volume. The shrinkage can lead to cracking and disruption of structural integrity of the Earth material and can accelerate slope creep of superficial deposits. This can result in loss of cohesion within the Earth material and can increase the landslide hazard in soil slopes, particularly after a long drought. Other problems like liquefaction, where the strength of a soil is reduced by earthquake shaking, can take place in low plasticity clay (Perlea et al. 1999).

Cross-References

▶ Atterberg Limits
▶ Characterization of Soils
▶ Chemical Weathering
▶ Classification of Soils
▶ Cohesive Soils
▶ Collapsible Soils
▶ Cone Penetrometer
▶ Dewatering
▶ Diagenesis
▶ Expansive Soils
▶ Geochemistry

- ▶ Ground Motion Amplification
- ▶ Hydrothermal Alteration
- ▶ Liquefaction
- ▶ Liquid Limit
- ▶ Marine Environments
- ▶ Noncohesive Soils
- ▶ Plastic Limit
- ▶ Quick Clay
- ▶ Saline Soils
- ▶ Sinkholes
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Mechanics
- ▶ Soil Properties
- ▶ Voids

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Climate Change

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Synonyms

Global change

Definition

"Climate change refers to a change in the state of the climate that can be identified (e.g., by using statistical tests) by changes in the mean and/or the variability of its properties and that persists for an extended period, typically decades or longer. Climate change may be due to natural internal processes or external forcings such as modulations of the solar cycles, volcanic eruptions, and persistent anthropogenic changes in the composition of the atmosphere or in land use. Note that the Framework Convention on Climate Change (UNFCCC), in its Article 1, defines climate change as: 'a change of climate which is attributed directly or indirectly to human activity that alters the composition of the global atmosphere and which is in addition to natural climate variability observed over comparable time periods'. The UNFCCC thus makes a distinction between climate change attributable to human activities altering the atmospheric composition and climate variability attributable to natural causes." (IPCC 2014a).

Evidence and Causes of Recent Global Warming

The increasing availability of observations provided by weather instruments (e.g., thermometers, pluviometers, barometers) and remote sensing techniques (e.g., satellites) allows one to investigate changes that occurred in climate system from the mid-nineteenth century up to the present. In this regard, the data record the on-going variations in atmosphere, cryosphere, oceans, and land surface with coverage quite different according to the analyzed variable and geographical area. At the global scale, the most reliable data collection and analysis is currently performed by the Intergovernmental Panel on Climate Change (IPCC), the leading international body for the assessment of climate change established in 1988, by the United Nations Environment Programme (UNEP), and the World Meteorological Organization (WMO), with a mission of properly documenting the current state of knowledge in climate change and its potential environmental and socio-economic impact. In the latest report (IPCC 2014b), it summaries the main evidence regarding recent climate change; reporting for each finding an evaluation of underlying evidence (limited, medium or robust) and agreement (low, medium or high) that supports an assignment of confidence, including the assessed likelihood of an outcome or a result is provided.

According to the IPCC, the globally averaged combined land and ocean surface temperature displays an increasing trend over the period 1880–2012 with a resulting warming of 0.85 °C [0.65–1.06 °C] (in square brackets the range with 90% likelihood of including the value is shown); since 1850, the last three decades were successively warmer than any preceding decades (medium confidence) (Fig. 1a). Approximately 90% of the increase in energy in climate systems between 1971 and 2010 was stored in oceans (1% in atmosphere) resulting in a warming by 0.11 °C [0.09–0.13 °C]. Global sea level rise is estimated at about 0.19 m [0.17–0.21 m] with a rate since the mid-nineteenth

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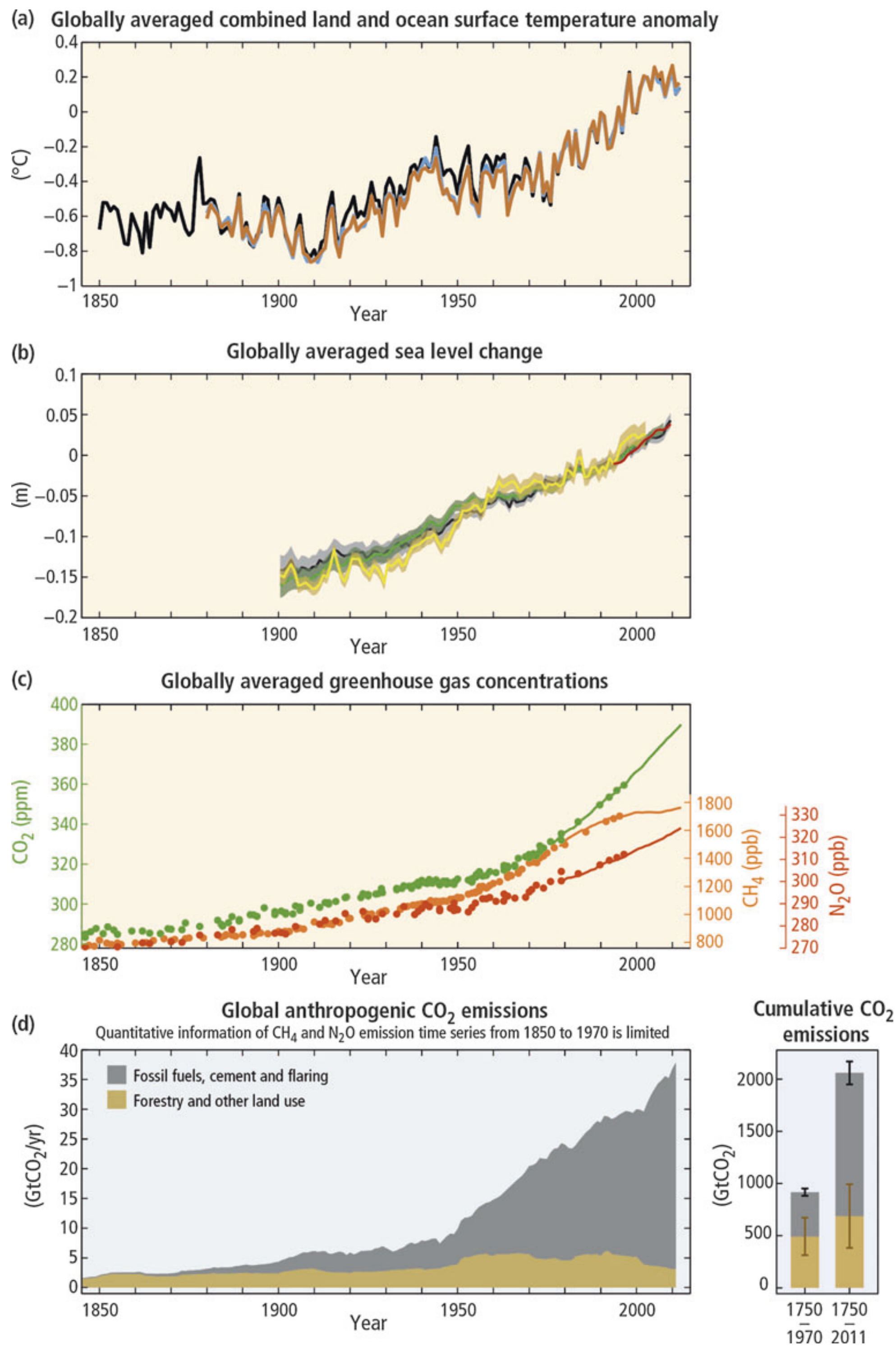


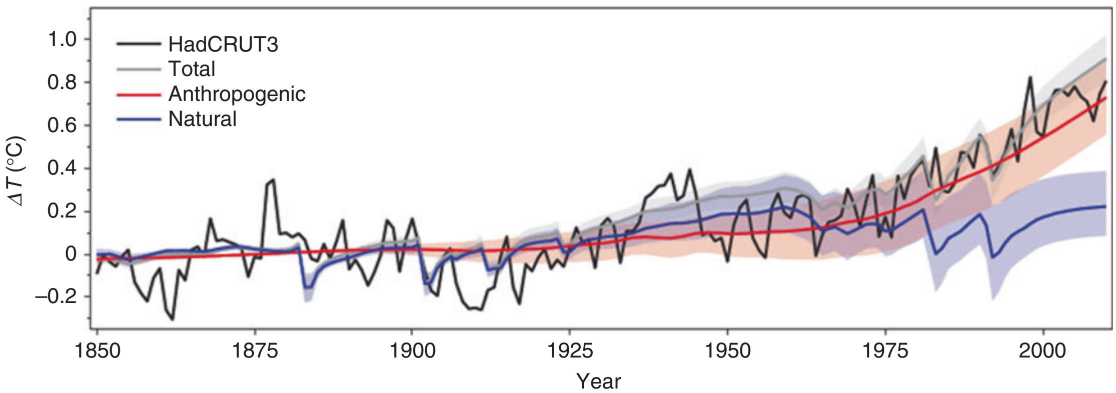
Climate Change, Fig. 1 (**a**) Globally averaged combined land and ocean surface temperature anomalies on annual scale assuming as reference the period from 1986 to 2005. Colors indicate different data sets. (**b**) Globally averaged sea level change on annual scale assuming as reference the period from 1986 to 2005. Colors indicate different data sets. All datasets are aligned to have the same value in 1993, the first year of satellite altimetry data (red). Where assessed, uncertainties are indicated by colored shading. (**c**) Atmospheric concentrations for carbon dioxide (CO_2, green), methane (CH_4, orange), and nitrous oxide (N_2O, red) as provided by ice core observations (dots) or direct atmospheric measurements (lines). (**d**) Global anthropogenic CO_2 emissions by forestry and other land use or from burning of fossil fuel, cement production, and flaring. Related uncertainties in estimations are displayed as bars and whiskers. Source: Figure SPM. 1 from Climate Change 2014: Synthesis Report. Contribution of Working Groups I, II and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Core Writing Team, Pachauri, R.K. and Meyer, L. (eds.)]. IPCC, Geneva, Switzerland century that is quite larger than that assessed in the previous 2000 years (high confidence) (Fig. 1b). Concerning precipitation, an increase is assessed over the mid-latitude land areas of the Northern Hemisphere (medium confidence before and high confidence after 1951), whereas, at other latitudes, negative trends are characterized by lower confidence. The variations in ocean surface salinity are adopted as a proxy of changes in the global water cycle since 1950: increases are assessed in areas where evaporation is prevalent whereas the opposite occurred where precipitation dominates (very likely confidence). Similarly, ocean acidification results from an increase in ocean CO_2 uptake: a decrease in pH of about 0.1 (high confidence) is evaluated with respect to the preindustrial era in ocean surface water. During the period 1992–2011, ice sheet mass in Greenland and the Arctic has decreased (high confidence), reductions in glaciers were detected worldwide (high confidence), and spring snow diminished (high confidence). Regarding the Arctic, sea-ice extent was observed to have reduced in 1979–2012 (3.5–4.1% per decade), whereas in Antarctic, an increase (1.2–1.8% per decade) is assessed for the period 1979–2012. The impacts of such changes vary across the globe. Alterations in hydrological systems with effects on quality and quantity of water resources are recognized with medium confidence. Several impacts on human systems are assumed ascribable to climate change; generally negative effects surpass the beneficial effects on crop yields (high confidence). Finally, climate change could significantly affect the occurrence and severity of weather extreme events and associated impacts. On a global scale, cold days are estimated to decrease with an increase for warm days (very likely); and heat waves have grown in many areas (e.g., Europa and Australia). In this perspective, the increase in heat-related human mortality and a simultaneous reduction in cold-related deaths are associated to climate change with medium confidence. The extent of area where a statistically significant increase in heavy precipitation is greater than those where an opposite trend is seen is recorded (likely). Accordingly, flooding risk is assumed to be increasing in several areas (medium confidence). Extreme sea level events have been probably increasing since 1970.

In order to identify the main drivers of such variations, one could conveniently identify the external or internal (to Earth system) processes recognized primarily as altering the energy conditions at the surface (net radiation or available energy at the surface of the Earth). External forcing concerns the magnitude and distribution of the incident solar radiation on the outermost atmosphere layer (e.g., changes in direct solar radiation input or orbital variations). Internal forcing is related to atmospheric composition and its effects on the Earth's energy balance and features of land surface. They can most appropriately be classified as natural or anthropogenic. Among the former, it is worth noting volcanic activity inducing variations in atmosphere composition due to emission of particulate matter and gases with a main effect of cooling the troposphere. The magnitudes of cooling results are a function of several factors (volcano location, explosive power, sulfur content). Regarding anthropogenic factors, gas emissions (driven largely by economic factors and population growth) play a crucial role. Among the others, carbon dioxide, methane, and nitrous oxide may act as greenhouse gases absorbing outgoing long-wave radiation that is radiated back down to the Earth entailing a surface temperature warming. The current atmospheric concentrations of carbon dioxide, methane, and nitrous oxide are at their highest for the past 800,000 years (Fig. 1c). Similarly, anthropogenically induced variations in land use/land cover, including deforestation processes, have changed albedo and CO_2 storage capacity that may induce changes in the climate system.

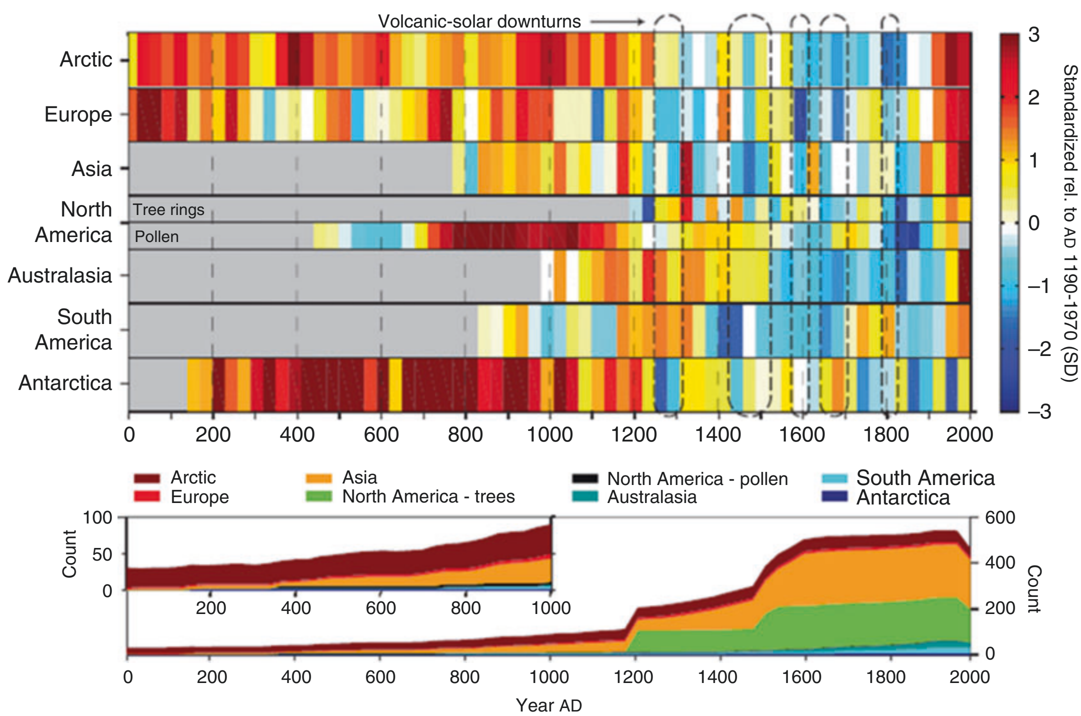
In recent years, in order to discriminate and weigh the human influence on the above-described climate changes, a significant effort has been devoted to reconstruct, through numerical experiments, the trends of global temperature since preindustrial era. The simulations (Fig. 2) prove that, if human factors influencing climate system was removed, solar and volcanic activity would have led to a slightly cooling in the Earth system, whereas the role of other natural variations is too small to explain the experienced warming. However, when the anthropic influences are accounted for, climate models are able to simulate at global scale the warming observed since the end of nineteenth century. Although there are uncertainties associated with climate simulations, this is a strong evidence that humans can substantially alter the climate system. In this perspective, IPCC (2014b) assumes that anthropic actions represent the dominant cause of the observed warming since the mid-twentieth century.

Past Climates: Placing Recent Climate Change in Context

In order to have a clear frame of the specific features of climate changes observed during the last 150 years, it is convenient to extend the observed record farther back in time, placing the recent changes in the global climate system into a broader and proper context. To this aim, longer-term climate change and variability at centennial to millennial scales should be reconstructed. Unfortunately, on such time scales, it is not possible to include instrumental records, but it is necessary to refer to documentary sources (human proxies) or natural proxies. The first include early newspaper reports, harvest dates, crop prices, phenological data or ships' logs. In the second case, information can be indirectly retrieved from tree-ring thicknesses, pollen deposits, beetle remains, and a variety of other indicators. Furthermore, oxygen isotopic composition from ice cores or



Climate Change, Fig. 2 Time series of total anthropogenic (red) and natural forcings (blue) contributions to total simulated (grey) and observed global temperature change (black as provided by HadCRUT3 dataset). The shadowed areas identify the 5–95% uncertainty range. Reference period: 1850–1900 (Figure source: Huber and Knutti 2012)



Climate Change, Fig. 3 Upper part: Temperatures variations (30-year-mean) estimated on seven global regions identified by PAGES 2 k Network; the measures are standardized to have the same mean (0) and standard deviation (1) over the period of overlap among records (ad 1190–1970). Bottom part: Number of individual proxy records by region over the time span. (Source: PAGES 2k Consortium 2013)

shells in oceanic sediments are substantially correlated with global temperature, providing temperature changes back some 2 million years ago. Analysing ocean core isotope, Van Andel (1994) reveals that several cycles of glacial advance and retreat are recognizable in Earth's climate history; the cycles are mainly driven by variations in in Earth's orbital elements. More specifically, during the Last Glacial Maximum (21 ka BP), global sea level and temperature were about 120 m and 4 °C lower than the current. In contrast during the Paleocene–Eocene Thermal Maximum (PETM) (55 million years ago), an increase in temperature of about 5 °C was assessed but with a warming rate significantly slower than those associated to recent climate changes (Porinchu 2017). The investigations performed by PAGES 2 k Consortium (2013) provide a very interesting characterization of temperature trends (in terms of 30 year mean temperature) at a continental scale over the last 2000 years (Fig. 3). Dashed lines identify periods of significant volcanic or solar activity. The analysis displays that the Medieval Warm Period (MWP) or Little

Ice Age (LIA), commonly recognized as global anomalies, were instead limited to some areas or often not simultaneous; MWP occurred in the period AD 800–1100 in Northern Hemisphere, whereas in the Southern Hemisphere, it occurred about AD 1150–1400. On the other hand, the cooling in LIA (second part of sixteenth century) is assumed to be induced by several concomitant causes: reduced solar irradiance, increased volcanic activity, and consistent land use changes due to deforestation. Nevertheless, affected areas experienced the LIA with different severity.

Climate Simulation Chains (CSC)

The warming of the climate system in recent decades is evident from observations that show an increase in global average air and ocean temperatures. Quantitative physical models are required to provide significant projections for the various climatic regions on Earth. Climate projections according to the different IPCC scenarios are usually evaluated using General Circulation Models (GCM). Such scenarios are a possible way in which human society may develop in the future, according to economic and social developments, available resources, and demographic growth. A GCM is a mathematical model used to represent the general circulation of the atmosphere and the ocean, taking into account thermodynamic terms for various energy sources, as well as the laws for mass and momentum conservation. Governing equations are solved using computational grids, which are characterized by spatial resolutions generally near or coarser than 100 km. GCMs are very relevant to study the current climate and to obtain projections on the future climate using the different emission scenarios (IPCC 2014b). IPCC scenarios provide possible and plausible evolution of climate forcing agents, such as greenhouse gases (GHG) and aerosols. These data are used as input for GCMs. The most important advantage from the usage of GCM outputs for practical applications is the definite physical consistency among variables (Robock et al. 1993). However, due to the low values of spatial resolution, they are not adequate for regional climate, to support impact applications and for studies on adaptation strategies to climate change.

Consequently, the generation of climate projections at higher resolution is needed for these goals (Giorgi et al. 2009). The need for local scenarios of climate change for impact studies is widely recognized in the scientific community and led to the development of several methodologies for deriving such information. These procedures are known as "downscaling" and the related interest is confirmed by the establishment of different international initiatives (e.g., WCRP CORDEX). Downscaling techniques have been designed to bridge the gap between the information provided by the GCM modeling community and those required by the impacts research community. A downscaling at the regional scale of the GCM scenarios can be provided with two different families of methods: dynamic or statistical approach. These methods are not alternatives but are generally applied "in cascade." As a first step, a Regional Climate Model (RCM) is used to provide a dynamic downscaling of the GCM. Successively, statistical techniques are applied to the RCM output to further increase the spatial resolution and/or remove the bias of the model output.

RCMs are numerical climate models used to perform an explicit simulation of the physical dynamics of the climate system, for a limited area of consideration. They are nested on the GCM, usually using a one-way nesting procedure. This technique consists of using the outputs from GCM simulations, conveniently interpolated to provide initial and driving lateral boundary conditions for a high-resolution simulation only for a part of the Earth surface, without feedback from the RCM to the driving GCM. It has been shown in many literature works that RCMs are able to provide simulated reliable structure and evolution of synoptic events, realistic regional detail of surface climate, and more detailed information on climate extremes (temperature, wind, precipitation) (Rummukainen 2010), often more important than knowledge of average properties for the modeling of natural hazards or for early warning systems. Moreover, the accuracy of RCMs in reproducing the spatial and temporal features of precipitation grows with increasing model resolution, in particular, the daily precipitation intensity distribution.

RCMs are characterized by spatial resolution generally between 1 km and 50 km, so these values are still not suitable for practical applications. For this reason, statistical downscaling (SD) is used to further increase the resolution, up to hundreds of meters. These techniques are based on statistical models applied to historical data. These methods are able to capture the empirical relationship between variables on a large scale ("predictors") and the local variables ("predictands"), for example, the precipitation at a specific location. Statistical downscaling methods include regression models, weather-pattern classification schemes, weather time-series generators, and combinations of the abovementioned techniques. Statistical downscaling techniques are less expensive computationally than RCM, for this reason, it is easier to provide an ensemble of high-resolution climate scenarios and, therefore, an assessment of the uncertainty in future scenarios. Finally, since RCMs output are generally affected by biases related to systematic model errors due to imperfect conceptualizations, discretization, and spatial averaging, several "bias-correction" techniques have been developed to remove these systematic errors.

Approaches to Accounting for Weaknesses and Uncertainties

In order to provide useful information for impact assessment studies, the uncertainties in climate change projections need to be fully characterized and, where possible, reduced (Knutti et al. 2010). The imperfect knowledge and model description of physical processes represent a critical source of uncertainty when performing climate projections, which tends to increase, as the scale of interest becomes increasingly finer. Due to this uncertainty, different models will generally produce different responses to the same climatic forcing. The reliability of climate projections needs to be assessed by measuring the model performance in reproducing the observed climate conditions in the past.

The sources of uncertainty associated with climate projections are the following (Giorgi et al. 2009):

- GHG emission scenarios
- Global models, regional models, and related parameterization
- Validation dataset reliability

The first source of uncertainty could be explored by performing simulations with different emission scenarios. Uncertainty associated with GCM and RCM could be explored performing the same simulation with different GCMs and/or different RCMs. An optimal way, which could be achieved by a cooperation effort among different research centers, is the execution of a matrix of tests, using different RCMs, each of them forced by different GCMs. A so-called multi-model ensemble could significantly improve the reliability of projections, especially if single elements are weighted with a measure of the model skill. A possible approach is the Reliability Ensemble Averaging (REA) method proposed by Giorgi and Mearns (2002). Moreover, there is an "intrinsic" uncertainty associated with every individual RCM (or GCM), due to its parameterization schemes and tuning parameters. Governing equations are not sufficient to give a complete description of the phenomena that take place in the atmosphere; some phenomena, even if described by those equations, take place on unresolved motion scales, but they have significant impact also on the scale of meteorological impact. In order to improve the quality of the previsions of the model, it is important to include the effects of these phenomena, at least in a statistical way, by means of "parameterization schemes." The uncertainty related to parameterization can be managed performing a sensitivity analysis, aimed to choose the "best" set of tuning parameters. Finally, uncertainty associated with reliability of observational datasets can be managed considering different datasets in the validation phase. With developments in scientific knowledge and computing, demands for gridded climate datasets with fine resolution have increased. High-resolution datasets can provide useful information for initializing numerical models and for validating a model. A number of statistical approaches exist to interpolate climate surfaces and consequently, several gridded datasets and long-time series have been constructed. Still, because of varied developers, datasets are not completely consistent.

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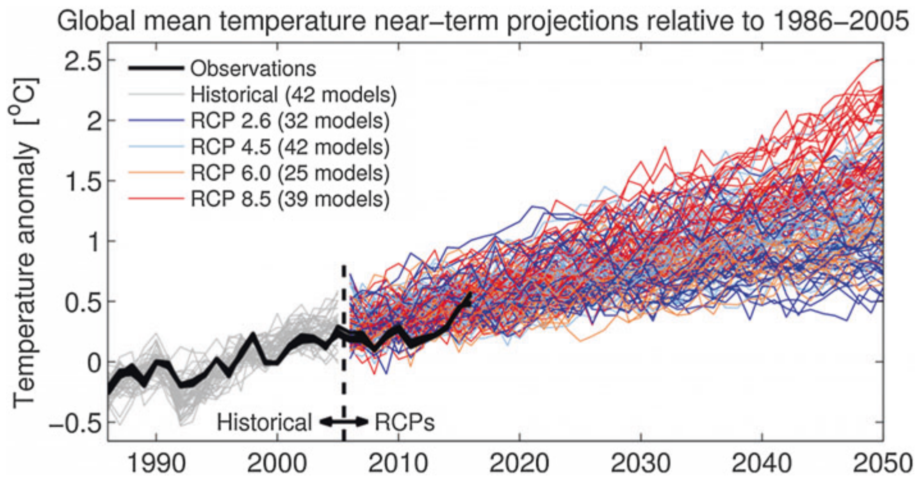
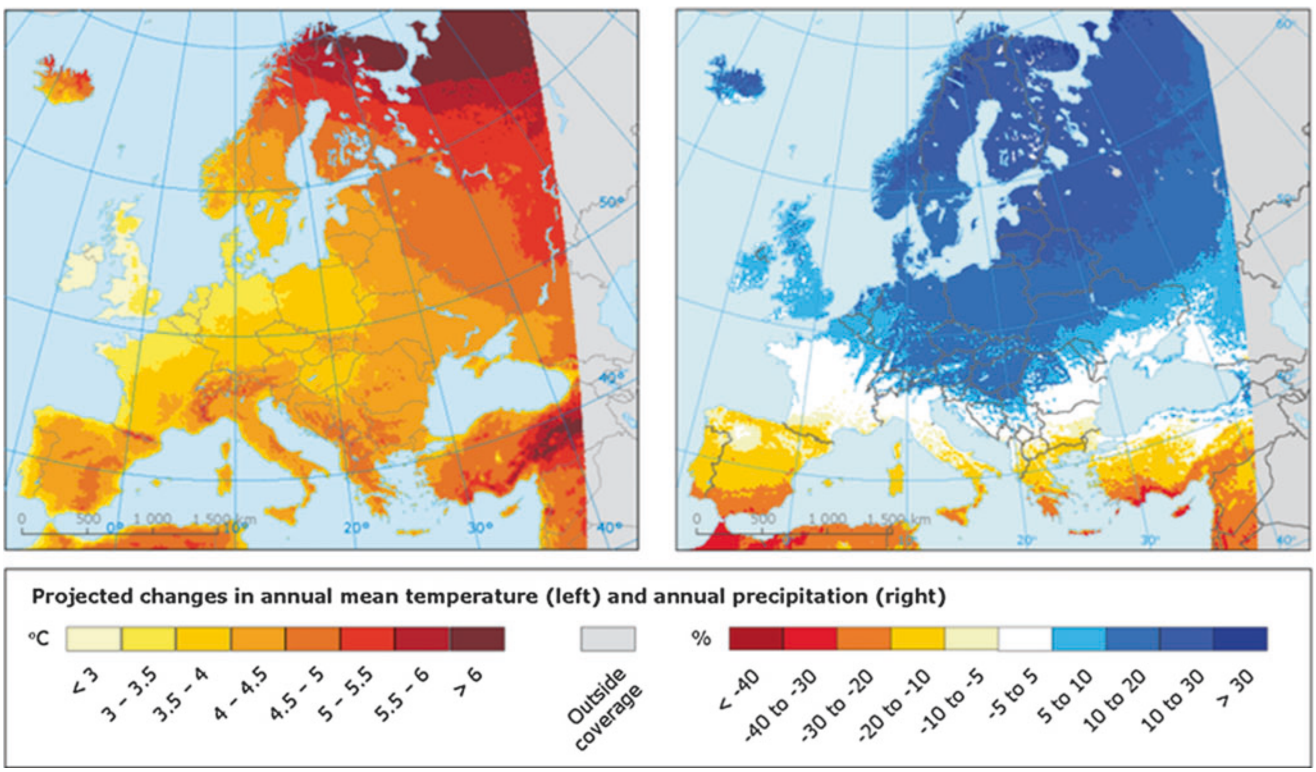
Projections of Future Climate

The Coupled Model Inter-comparison Project (CMIP) is an international effort to advance climate models by comparing multiple GCM simulations to each other and to observations. These comparisons can help our understanding of past and future climate changes and lead to climate model improvements. CMIP has coordinated four large model inter-comparison projects. Most have been extensively used in the various IPCC assessment reports since 1990 (IPCC 2014b). The Fifth Coupled Model Inter-comparison Project (CMIP5) was used in support of the Fifth Assessment Report (AR5) of the IPCC (IPCC 2014b). CMIP5 has represented a collaborative work involving 20 climate-modelling groups over the world (Taylor et al. 2012). The CMIP5 projections of climate change are based on GCMs and are driven by concentration or emission scenarios consistent with the four Radiative Concentration Pathways (RCPs) described in IPCC (2014b). RCPs are based on a range of projections of future population growth, technological development, and social responses. The labels for the RCPs provide a rough estimate of the radiative forcing in the year 2100 (relative to preindustrial conditions). The radiative forcing under RCP8.5 increases along the twenty-first century and reaches a level of about 8.5 W m^{-2} at the end of the century. In addition to this "strong" scenario, two transitional scenarios were defined, RCP4.5 and RCP6, and a weak scenario (RCP2.6) in which radiative forcing touches a maximum value near the middle of the century before decreasing to 2.6 W m^{-2}. The four CMIP5 scenario runs, which provide a range of simulated climate values (characterizing the next few decades to centuries), can be used as the beginning for studying climate change impacts and policy issues of considerable interest and relevance to society. A summary of the global mean temperature near term projections, according with the CMIP5 simulations, is shown in Fig. 4.

Additional experiments were built based on CMIP5. In particular, the CORDEX project produced high-resolution "downscaled" climate data based on the CMIP5 simulations. The WCRP Coordinated Regional Downscaling Experiment (CORDEX) project (Giorgi et al. 2009) was established to provide a global coordination of regional climate

Climate Change, Fig. 4 A summary of the global mean temperature near term projections, according with the CMIP5 simulations (Source: https://creativecommons.org/licenses/by-sa/4.0/)





Climate Change, Fig. 5 Projected changes in annual mean temperature (left) and annual precipitation (right) for Europe based on an ensemble of regional climate model simulations provided by the EURO-CORDEX initiative Projected changes are for 2071–2100, compared to 1971–2000, RCP8.5 scenario (Source: https://creativecommons.org/licenses/by/2.5/dk/deed.en_GB)

downscaling for improved climate change adaptation policy and impact assessment. A set of common domains, encompassing the majority of the populated land areas worldwide plus both the Arctic and Antarctic, is defined within CORDEX for the execution of regional downscaling. CORDEX is essentially designed to provide a framework to evaluate model performances and to design a set of experiments to produce climate projections for use in impact and adaptation studies. The choice of common domains and resolutions was done in order to provide a framework accessible to a broad scientific community with maximum use of results. Figure 5 shows the climate change projections for Europe based on an ensemble of regional climate model simulations provided by the EURO-CORDEX initiative (European branch of CORDEX). Projected changes are for 2071–2100, compared to 1971–2000, based on the average of a multi-model ensemble forced with the RCP8.5 high emissions scenario.

One of the problems that emerged from CORDEX was the heterogeneity in terms of availability of simulations across different domains. While for some domains (e.g., Europe), large ensembles of model simulations are available, for others, e.g., South America, Australia, only a few experiments have been conducted. In order to match with needs for specific regions, a CORDEX core framework has been defined (CORDEX-CORE) conceived to coordinate simulations that could provide fine-scale climate information. In particular, for each region a matrix of experiments was designed to cover as much as possible different dimensions of the uncertainty space (Gutowski et al. 2016).

Test Case for Geohydrological Hazards: Landslide Impacts

Evaluating the effects of climate change on landslide occurrence and evolution is becoming a common exercise to many, especially to people working in the world of mass media (Picarelli et al. 2016). Unfortunately, a reliable prediction of the problems that may be encountered in the near future is a very complex issue because of the uncertainty that characterizes any involved discipline and, still more, the ensemble of disciplines as a whole. In contrast, the natural growth of any branch of knowledge is providing new instruments for, at least, drawing rational scenarios capable to account for the major variables involved. This goal can be generally achieved:

(i) Investigating the correlations between variations in atmospheric forcing and in frequency and magnitude of landslide events adopting data over past periods
(ii) Forecasting the potential trends through climate projections and a quantitative analysis of their effects on the slope-atmosphere system

The past-based frequency studies deal mainly with the evaluation of historic scientific data covering significant time intervals (e.g., through event catalogues and precipitation records) and the identification of clues left by still recognizable past landslide events. In this sense, changes in demography, land use and cover, monitoring techniques, and obviously climate over the last few decades deeply affect the quality and reliability of past-based frequency analysis, making it hard to quantify hazard levels at a large scale. A safer path is represented by susceptibility studies that provide only "steady-state" pictures of target areas and they are not sufficient to assess landslide intensity and magnitude variations. Moreover, detailed databases/inventories of observations, that represent the most useful source/tool for quantifying changes in past landslide occurrence and defining relationships for the future, are often not available, or if available, the information stored is not always reliable and often inconsistent with the other catalogues. For all these reasons, even if most of the assessments based on the past-events analysis bring attention on a broad range of possible impacts of climate change on landslide activity, their relations result to be still weak and links uncertain.

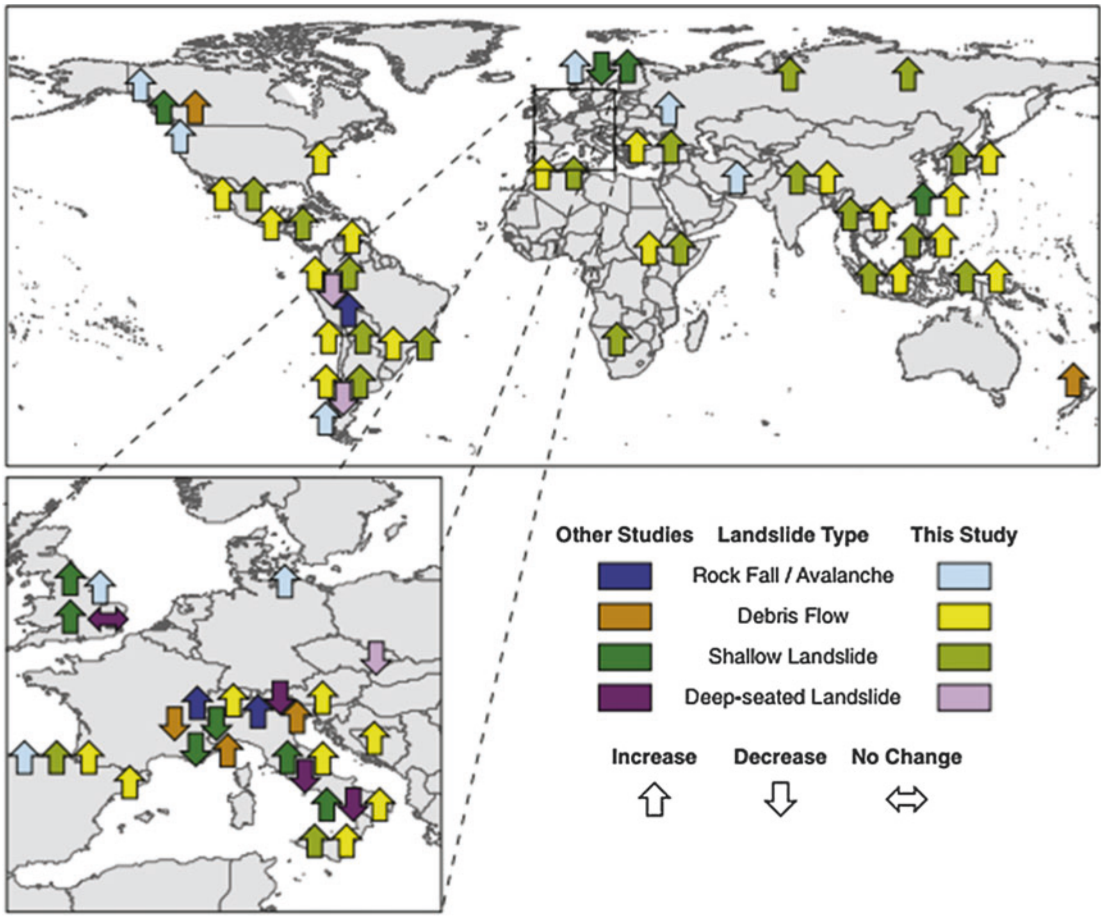
The second category is more complex as it considers different skills and multidisciplinary approaches. Figure 6 shows the variations in frequency or activity of four landslide types driven by the projected climate change according to primary literature on the expected behavior of estimated variations in weather patterns. Paying specific attention to the European continent, a general decrease in abundance/activity of deep-seated landslides and of an increase in rock falls, debris flows, and shallow-landslides are highlighted.

In this sense, only climate projections can indicate some features of trends in landslide occurrences and evolution. It is expected that the projected increase in surface temperature and more intense and frequent rainfall events could increase shallow landslides, including rock falls, debris flows, and debris avalanches, as well as ice falls and snow avalanches in high mountain areas. In mountain environments, probably the most affected areas in the projected general warming, an expected increase of the thawing processes will affect rock slope stability conditions and favor higher infiltration amounts within fine/coarse soils; contextually, the "mutation" of snowfalls in rainfall precipitation will likely favor the inception of debris flows or, more generally, shallow-landslides.

In any cases, the geohydrological features of the involved soils seem to be a key factor. In this sense, Dixon and Brook (2007) state that "landslides with a shorter antecedent period, or those that exhibit a strong response to relatively short-term rainfall events, could be more vulnerable to the predicted increase variability of winter rainfall and so could experience a shorter return period for slope movements. However, slope instability caused by longer periods of antecedent rainfall may occur less often in response to drier summers with increased evapotranspiration."

Forecasting the potential trends through climate projections and a quantitative analysis of their effects on the slope-atmosphere system require combining different elements (well-defined climate simulation chains, CSCs, modern powerful sensors to measure the atmospheric variables, and powerful numerical codes capable to simulate the complex slope-atmosphere interaction) to obtain useful and suitable simulation chains (SC).

Such SC are generally defined by two macro-components. The first macro-component features the atmospheric forcing. It follows the CSCs described elsewhere. Despite the increasing performance of climate simulations, the coarse horizontal resolutions (also for RCMs) could induce imperfect representations of atmospheric physics (e.g., diurnal convective cycle)



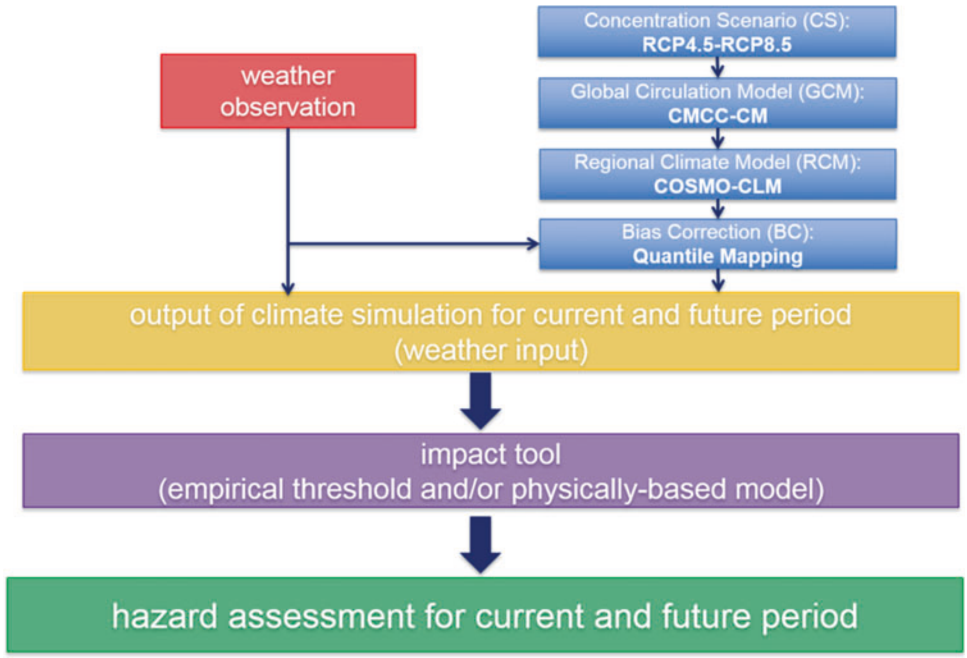
Climate Change, Fig. 6 Map shows general areas of expected variations in the abundance or activity of four landslide types, driven by the projected climate change. Dark colors are projections from the literature and light colors are projections from this study (from Gariano and Guzzetti 2016)

preventing a direct use of atmospheric variables in impact models. To reduce this error assumed as systematic, the scientific community is involved in proposing numerous bias correction (BC) techniques able to "adjust" model output towards observations in a post-processing step (Maraun 2016): the delta change method, local intensity scaling, multiple linear regressions, linear scaling, and quantile mapping. Regardless of the considered technique, the BC can be performed, provided that adequate observations (generally over 30 years) of atmospheric forcing of interest (primarily precipitation and temperature) are available.

The second macro-component is represented by the impact models adopted for the hazard assessment. In this sense, empirical threshold (e.g., rainfall thresholds for the initiation of landslides like intensity-duration rainfall curve; extreme value approaches like GEV) and physically based models (e.g., models which "translate" weather forcing into hydrological variables related to slope stability) can be considered using as input climate simulation data over current and future time-horizons at the slope scale or over an extended area. The impact model performance increases with the input parameters (only precipitation for rainfall threshold; precipitation, evapotranspiration, soil hydraulic and thermal properties, soil mechanical properties, and geomorphological features for more complex physically based tool). Keeping in mind this state-of-the-art, two well-documented landslide test cases are considered in the following to investigate the influence of climate change (CC) on different representative soil deposits in slope referring to deep and shallow rainfall-induced landslides. The analyses are performed through the SC of Fig. 7.

Deed Rainfall-Induced Landslides: The Orvieto Test Case

The hydrological cycle is strongly modified by the CC with a direct impact on the piezometric regime of natural slopes and on their stability conditions. Such an issue clearly emerges considering the deep rainfall-induced landslides (DRILs) in fine-grained deposits: their activity is essentially governed by the continuous interaction between soil and atmosphere (primarily though precipitation and evapotranspiration) that



Climate Change, Fig. 7 Simulation Chain (SC) adopted for Orvieto and Nocera test cases

yields fluctuations in pore water pressure affecting the operational shear strength along the slip surface. The relation between weather conditions and piezometric regime in fine-grained deposits is strongly influenced by their low hydraulic conductivity, which generally delays the precipitation effects on pore pressures. In this sense, the experience based on several well-documented cases suggests an evident connection between landslide activity and rainfall accumulated over some months.

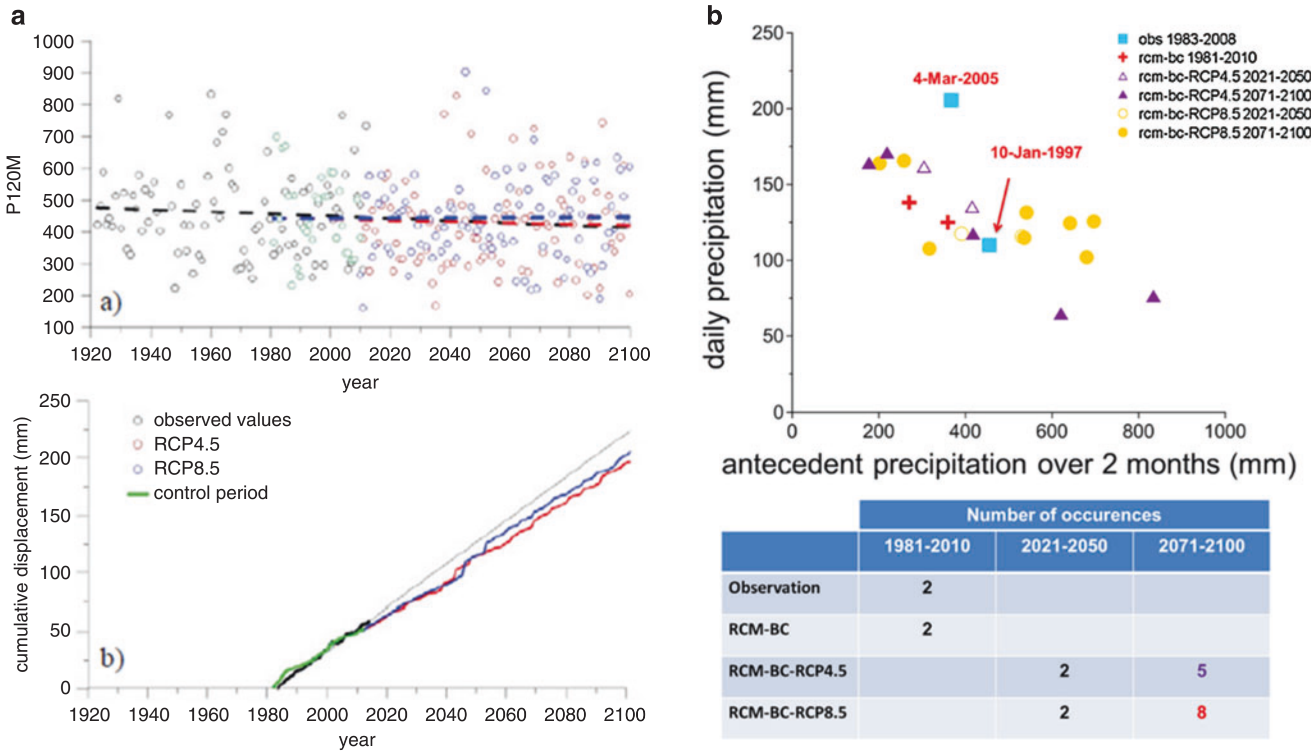
For DRILs assessment in CC studies, a very exhaustive test case is represented by the DRILs of the Orvieto slope (Rianna et al. 2014; Picarelli et al. 2016). Orvieto is an historical town near Rome rising on top of a 50 m thick tuff slab with subvertical lateral cliffs overlying over-consolidated clay. These are stiff and intact, but the shallowest part of the deposit is jointed and fissured. The clay rich slopes around the hill are blanketed by an irregular debris cover produced by disruption of volcanic material and slope movements.

Since 1982, inclinometers and piezometers have recorded slope movements and pore pressure in seven verticals located on the northern part of the hill. The exceptionally long time-history allows a view on both short and mid-term variations in displacement rates related to changes in pore pressures and rainfall cumulated over extended periods. The monitoring activity indicates that deep movements occur at an average displacement rate of 2–6 mm/year with yearly reactivation except for particularly dry years.

This test case is investigated in a CC perspective addressing all the different tasks of the SC of Fig. 7. In this sense, the authors assume these inputs:

- *Weather input*: four 30-year datasets of precipitation and temperature at daily resolution; the first dataset corresponds to observation data over 1981–2010 obtained by a meteorological station of the Hydrological Service of Region Umbria active since 1920 with few short periods of incompleteness; the other datasets derive from the CSC formed by the GCM CMCC-CM dynamically downscaled through the RCM COSMO-CLM simulation at resolution of 8 km in the configuration optimized by CMCC for Italy (Bucchignani et al. 2015) adopting as forcing for climate models the RCP 4.5 and 8.5; these datasets are considered assuming the period 1981–2010 as reference for historical condition and 2071–2100 as indicative for future conditions.
- *Bias correction*: non-parametric approach based on the observed and simulated empirical quantiles (Villani et al. 2015) for precipitation and temperature.
- *Interpretative tool*: empirical threshold which couples the effect of cumulative precipitation values with slope displacements; in this sense, the observed annual displacement, δ, was related to the maximum yearly cumulative rainfall over 120 days, R120; over the monitoring period, such threshold provides a cumulative displacement approximately equal to the actual value (calibration step).

The interpretative tool can be adopted to assess the potential variations in shallow rainfall-induced landslides (SRILs) (Fig. 8a) when the interpretative tool is forced by bias-corrected climate datasets: for an historical period, it allows one to verify the reliability of the entire SC (validation step),



	Number of occurences		
	1981-2010	2021-2050	2071-2100
Observation	2		
RCM-BC	2		
RCM-BC-RCP4.5		2	5
RCM-BC-RCP8.5		2	8

Climate Change, Fig. 8 Assessing the potential expected variations in DRILs (**a**) and SRILs (**b**) for the Orvieto and Nocera Inferiore test case, respectively (Sources: Modified from Rianna et al. (2014) for (**a**) and Rianna et al. 2017 for (**b**))

whereas for the future it reveals an evident reducing trend in the rate of movement according to the RCPs investigated (future projection step). In this sense, such behavior can be directly related to the recognized decreasing trend in the maximum yearly cumulative rainfall over 4 months that result from weather modifications in the landslide area. Local climate changes should then be responsible for a substantial slow deceleration of the landslide movement.

Shallow Rainfall-Induced Landslides: The Nocera Inferiore Test Case

A very exhaustive hotspot for impact applications developed in the last years is represented by the shallow rainfall-induced landslides (SRILs) affecting the pyroclastic slopes of the Campania Region in South-Western Italy. Over the past years, such slopes were frequently a theatre to SRILs resulting in casualties and damages to infrastructures and buildings and well documented by the high exposure and demographic pressure of the territory (e.g., Sarno in 1998).

The Campania pyroclastic soils are the product of intense activity of different eruptive centers (Roccamonfina, Phlegrean Fields and Somma-Vesuvius Mt) in the area. This activity generated loose air-fall deposits characterized by alternating layers of non-plastic ash (silty sand) and pumice (gravely sand) with differences in texture and cover thickness due to the distance from the eruptive centers and wind directions during the eruptions.

These soils are generally in an unsaturated condition continuously modified by the interaction fluxes between soil and atmosphere. The unsaturated state entails stability also for slopes higher than the friction angle due to the positive effect of suction. The coupled effect of particularly wet periods (during which infiltration contributions largely exceed evapotranspiration losses) followed by heavy and persistent rainfall events on 1–2 days scale can induce suction (and related shear strength) decreases such as to trigger SRILs.

For these phenomena, the potential effect of CC is currently a challenging issue: indeed, while temperature increase could lead to increase in evapotranspiration losses (supporting slope stability), the variation in expected rainfall patterns (increase of cumulative values for event and of precipitation inter-arrival period) could have totally different effects on actual infiltration values and then soil water content.

Among the different geomorphological contexts of the Campania Region, attention was paid to the slopes forming the Lattari Mts, around the Nocera Inferiore municipality. Since 1960, this area was affected by several SRILs (1960, 1972, 1997, and 2005) that occurred in the wettest period of the year (November–March) and entailing casualties and service interruptions/disruptions especially on the Naples-Salerno highway.

This case study represents a very exhaustive hotspot since climate projections were considered combining differently all the elements of the SC (Fig. 7) to bring out the different levels of uncertainty that characterize the transition from climate projections to hazard assessments (Reder et al. 2016; Rianna et al. 2017).

For instance, the main findings obtained adopting the most performing SC are reported (Rianna et al. 2017). The major features of the SC are:

- *Weather input*: six 30-year datasets of precipitation and max/min temperature at daily resolution; the first dataset corresponds to observation data over 1981–2010 obtained by joining measurements from two weather stations located near Nocera and assessing its consistency through homogeneity tests; the other datasets derive from the CSC formed by the GCM CMCC-CM dynamically downscaled through the RCM COSMO-CLM simulation at resolution of 8 km in the configuration optimized by CMCC for Italy (Bucchignani et al. 2015) adopting as forcing for climate models the RCP 4.5 and 8.5; these datasets are considered assuming the period 1981–2010 as reference for historical condition and 2021–2050 and 2071–2100 as indicative for future conditions respectively for near- and long-time horizons.
- *Bias correction*: non-parametric approach based on the observed and simulated empirical quantiles (Villani et al. 2015) for precipitation and max/min temperature.
- *Interpretative tool*: physically based model which schematizes both liquid and vapor water flow by accounting for hydrothermal coupling and incorporates evaporation as both boundary and internal phenomenon. Such a model, implemented in the FEM code VADOSE/W 2007 and set in a 1D time saving configuration (thickness = 2 m), allows one to turn atmospheric boundary conditions (precipitation and potential evaporation) into hydrological soil variables related to slope stability (suction and water storage).

The interpretative tool, forced using the observation data, returns precisely the higher water storage peaks at the SRILs actually that occurred (January 1997 and March 2005) allowing a water storage threshold to be fixed minimizing the number of false alarms (calibration step). Such thresholds can be adopted to assess the potential variations in SRILs (Fig. 8b) when the interpretative tool is forced by bias corrected climate datasets. For the historical period, it allows one to verify the reliability of the entire SC (validation step), whereas for the future it reveals how increased hazardous conditions could be expected with different features according to the time horizon and RCPs investigated (future projection step). Specifically, the number of occurrences increases from the short to the long-time horizon and increases from RCP4.5 to RCP8.5.

Response Strategies

The concepts at the base of response strategies include keywords such as mitigation, adaptation, and geoengineering.

According with the IPCC, mitigation represents “a human intervention to reduce the sources or enhance the sinks of GHGs.” In a general overview about response strategies in a CC context, these issues can be achieved by combining different elements – such as increasing the energy efficiency, using low-carbon energy technologies (renewable energy), enhancing carbon sinks through, for example, reforestation and preventing deforestation – capable of limiting directly or indirectly CC.

Regarding adaptation, IPCC states that it is “the process of adjustment to actual or expected climate and its effects” reducing potential damages, taking advantage of opportunities, coping with consequences. Future CC without efforts to limit GHGs would, in the long-term, be likely to exceed the capacity of natural, managed, and human systems to adapt. Geoengineering refers to a “broad set of methods and technologies that aim to deliberately alter the climate system in order to alleviate the impacts of climate change. Most, but not all, methods seek to either (1) reduce the amount of absorbed solar energy in the climate system (Solar Radiation Management) or (2) increase net carbon sinks from the atmosphere at a scale sufficiently large to alter climate (Carbon Dioxide Removal). Scale and intent are of central importance. Two key characteristics of geoengineering methods of particular concern are that they use or affect the climate system (e.g., atmosphere, land or ocean) globally or regionally and/or could have substantive unintended effects that cross national boundaries. Geoengineering is different from weather modification and ecological engineering, but the boundary can be fuzzy” (IPCC 2014a).

Focusing on the geohydrological impacts and specifically on landslides, Picarelli et al. (2016) detect general rules to reduce landslide risk in a changing context. Also in this case, risk reduction is recommended to achieve through mitigation efforts and adaptation strategies configured at different temporal and geographical scales, and adopting a mix of structural (hard) and non-structural (soft) measures.

Hard mitigation measures are usually designed according with a reference return period (RT) determined using a time-series (e.g., maximum yearly precipitation) under “steady-state” assumption. In a changing context and more specifically in a changing climate context, such assumption could be not valid: engineering structures should be designed according with time series considering current and potential future variations of the selected design event. Consequently, hard measures could be more expensive to implement and maintain and they are suggested for areas at elevated risk for which they can entail a significant risk reduction. This is the case of areas affected by rapid landslides.

Soft measures are usually proposed according to current landslide hazard and risk assessment on the basis of reviews of historical landslide events. They are managed using plans at the local and national scale, defined, however, neglecting or underestimating the risk posed by specific landslide types and, in particular, those that are expected to potentially increase in response to CC. Effective forms of soft measures are probably monitoring and early warning. The current monitoring networks provide limited homogeneous datasets capable of coupling atmospheric and hydrological observations for landslide assessment and risk evaluation requiring a strong improvement. Such limitations include blindness to snowfall at lower terrain elevations, low prevalence of rain gauges in elevated areas and in small mountain catchments where debris flows occur, inaccuracy for high intensity rainfall events, and scarcity in soil hydrological observations (moisture and pore pressure).

The improvement of current capacity of landslide early warning systems entails a significant effort from multiple viewpoints. In this sense, adequate behaviors of people are mandatory. This can be achieved through targeted education and dissemination efforts.

Summary and Conclusion

The IPCC (2014a) defines the climate change as a change in the state of the climate that can be identified by changes in the mean and/or the variability of its properties and that persists for an extended period, typically decades or longer. In this sense, the increasing availability of weather and remote sensing data allows one to detect changes that occurred in climate system from the mid-nineteenth century up to the present. During these decades, the warming of the climate system is evident. The impacts of such changes are different: alterations in hydrological systems with effects on quality and quantity of water resources, impact related to the human activity (i.e., increasing of greenhouse gases), variations in occurrence and severity of weather extreme events and associated impacts (i.e., increase in heat waves).

Despite the remarkable technological progress of the last century, the recording of climate-related variables involves only a very small fraction of the Earth's climate history; such a gap is reduced by investigating the ancient clues left by people and nature with the aim to hone the recent changes in the global climate system into a broader and proper context.

The characterization of the evolution of the Earth's climate history and its variation represents the starting point for climate studies about the potential variations in the future. Such studies require future climate projections obtained for the various climatic regions on Earth by adopting quantitative physical models. In this perspective, it is important to use terms such as "potential" or "expected" since climate variations in the future are assessed based on projections and obviously can be affected by large uncertainties. To provide useful information for impact assessment studies and for defining impact policies by decision-makers, the uncertainties in climate change projections need to be fully characterized and, where possible, reduced. The sources of uncertainty associated with climate projections are many. The first source is related to the GHGs emission scenarios and could be explored by performing simulations using different scenarios. The second includes the Global and Regional models and their parameterization; in this sense, uncertainty associated with GCM and RCM could be explored performing the same simulation with different GCMs and/or different RCMs whereas the uncertainty related to parameterization could be managed performing a sensitivity analysis, aimed to choose the "best" set of tuning parameters. Finally, the last uncertainty is associated with reliability of observational datasets, and it could be managed considering different datasets in the validation phase.

Keeping in mind climate variability and climate simulation chains and their uncertainties, the effects of CC can be appreciated referring to geohydrological impacts and specifically to landslide occurrence and evolution. This goal can be achieved investigating the correlations between variations in atmospheric forcing and in frequency and magnitude of landslide events adopting data over past periods and forecasting the potential trends through climate projections and a quantitative analysis of their effects on the slope-atmosphere system. The past-based frequency studies deal with the evaluation of historic scientific data covering significant time intervals and the identification of the clues left by still recognizable past landslide events. Forecasting the potential trends through climate projections and a quantitative analysis of their effects on the slope-atmosphere system require combining different elements in a simulation chain. Such SC are generally defined by two macro-components: the former features the atmospheric forcing; the latter includes the impact models adopted for the hazard assessment. Referring to two types of landslide movement (deep for fine-grained soil and shallow for coarser-grained soil), the discussion of two cases demonstrated that the potential CC could determine a moderate slowing down of active slides in clay rich environments and a substantial worsening of slope stability conditions in unsaturated granular soils.

Climate change and its impact at different scales are a challenge for people. Since one of the main causes of CC is due to anthropogenic activity, human efforts are crucial in managing and reducing the on-going variations of climate patterns. In this sense, the term “mitigation” and “adaptation” are having more and more prominence at different scales (from the local to the global) in the context of policy. People have to appreciate all these topics.

Cross-References

▶ Avalanche
▶ Characterization of Soils
▶ Chemical Weathering
▶ Clay
▶ Coast Defenses
▶ Coastal Environments
▶ Cohesive Soils
▶ Collapsible Soils
▶ Databases
▶ Desert Environments
▶ Desiccation
▶ Environments
▶ Erosion
▶ Expansive Soils
▶ Factor of Safety
▶ Failure Criteria
▶ Glacier Environments
▶ Hazard
▶ Hazard Assessment
▶ Hydrology
▶ Infrastructure
▶ Landslide
▶ Mass Movement
▶ Modelling
▶ Mountain Environments
▶ Noncohesive Soils
▶ Permafrost
▶ Physical Weathering
▶ Risk Assessment
▶ Run-Off
▶ Sea Level
▶ Sinkholes
▶ Soil Laboratory Tests
▶ Stabilization
▶ Subsidence
▶ Tropical Environments
▶ Vegetation Cover

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Coal

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Definition

Coal is a brownish-black to black, combustible, naturally occurring sedimentary rock formed from fossilized plants and vegetable matter consisting of amorphous carbon with organic and inorganic compounds.

Characteristics

Coal occurs in seams in series of alternating sedimentary sequences known as cyclothems. Coal deposits occur in sedimentary basins around the world from Devonian to Quaternary in age. Coal is used to generate electricity (thermal or steam coal) or to produce iron and steel (coking coal) (Fig. 1).

Coal originates in fluvial, deltaic, or coastal barrier settings, where subsidence, sediment supply, and water level changes take place. There are two coal groups: humic coals comprise mixtures of coarse macroscopic plant remains and are bedded. The four lithotypes of humic coal are vitrain (black, glassy, vitreous), clarain (bright luster), durain (dull), and fusain (soft, friable). Sapropelic coals contain microscopic plant debris and appear isotropic. Coal contains inherent constituent macerals (organic equivalent of minerals in rocks) and inorganic minerals (referred to as ash or the mineral residue following combustion). Humification of peat, diagenesis, and coalification converts peat to form a different coal rank. Brown coal refers to low-rank coals, including lignite and sub-bituminous. Black coal or hard coal refers to higher rank coals like bituminous and anthracite.

Coal, Fig. 1 Exposure in an open pit mine, exposing a historical, shallow, abandoned room and pillar mine, Sheffield, Yorkshire, United Kingdom (Photo. Dr. L. J. Donnelly)

Coal varies from occurrences or small partings millimeters thick to seams hundreds of meters thick. Coals are also variable in lateral continuity, rank, maceral content, quality, and engineering properties (Scott et al. 2010). Rank is controlled by burial and tectonic history, whereas the general properties of coal are influenced by the depositional environment. The quality and physical and chemical properties of coal determine its potential uses. Proximate coal analysis provides moisture, volatile content, fixed carbon, and ash. Ultimate analysis gives the chemical content of coal in terms of carbon, hydrogen, oxygen, nitrogen, and sulfur. When evaluating coal analyses, it is important to understand "on what basis" the data are presented. The industrial uses of coal and performance in a furnace are determined by combustions, caking tests, and coking tests. The physical properties of coal are also important for commercial uses and include the coal's density, hardness, grindability, abrasiveness, size distribution, and float-sink tests (Thomas 2013).

Coal seams may be discontinuous due to post-depositional tectonic deformation. These include seam splitting, washouts, floor rolls, thickness variations, quality variations, faults, folds, mineralization, and igneous intrusions. Cleat (joints) enables coal to break into blocks with three roughly perpendicular faces. Mineralization, often calcite, ankerite, or pyrite, could be present on some cleat faces. Coal is susceptible to physical weathering although chemical weathering can occur on coal samples due to oxidation and changes in moisture and temperature. The strength of coal is less than 20 MPa; however, the strength of a coal sample and modulus of elasticity decrease with increasing sample size (Bell 1983). Where shallow, abandoned coal seams exist, these must be appropriately investigated to identify the coal mining hazards and their associated geotechnical risks (CIRIA 2017, Bell and Donnelly 2006).

Cross-References

► Acid Mine Drainage
► Borehole Investigations
► Classification of Rocks

▶ Engineering Properties
▶ Environmental Assessment
▶ Facies
▶ Faults
▶ Geohazards
▶ Geotechnical Engineering
▶ Ground Preparation
▶ Groundwater Rebound
▶ Hazard Assessment
▶ Induced Seismicity
▶ Mine Closure
▶ Mining
▶ Mining Hazards
▶ Sedimentary Rocks
▶ Site Investigation
▶ Subsidence
▶ Voids
▶ Zone of Influence

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Coast Defenses

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Definition

Hard or soft engineered structures, or combinations of these, designed to form effective coastal defense solutions.

Introduction

Coastal defenses are measures taken to protect coastlines from erosion and other damage or to prevent flooding caused by the combined effect of sea waves, extreme tides, and storm surge. They are designed to work by controlling wave action, coastal currents, and sediment movement on beaches. Each coastal defense method has its own advantages and disadvantages, which should be carefully studied and weighed before adopting one into a given site (Horikawa 1978).

Types of Coastal Defenses

There are two main types of coastal defenses: hard engineered structures and soft engineering solutions (Coastal Engineering Manual 2008). Often, hard structures are combined with soft engineering solutions to form effective coastal defense schemes. Hard structures can be seawalls, offshore breakwaters, and groynes. Most hard structures are constructed of concrete or rock. They alter the existing dynamics of the beach and can have high impact on the coastal environment and landscape. A seawall is a structure constructed along the shoreline to prevent wave action on the beach which removes sediment from the coast resulting in excessive damage. Seawalls are also used to control coastal flooding from wave overtopping. Offshore breakwaters are constructed parallel to the beach, farther away from the shoreline. They diminish incoming wave energy thus reducing sediment movement and beach erosion. Groynes are constructed mostly perpendicular to the shoreline. They are used as coastal defenses when beach erosion occurs mainly as a result of sediment movement along the beach (Figs. 1 and 2).

Soft engineering solutions can be beach replenishment (Reeve and Karunarathna 2017), growing coastal vegetation and beach draining. In beach replenishment, sand or other suitable material is added to the beach to replace lost sediment from erosion due to wave and tidal actions. Growth of coastal vegetation is used to control sand movement due to wind and wave activities. This method is adopted mainly to prevent the erosion of shorefaces and coastal dunes. Beach draining removes excess water from the beach which reduces sand movement thus increasing beach stability.

Advantages and Disadvantages

All coastal defense solutions have advantages and disadvantages. The primary advantages of hard engineered structures are that they can be effectively designed to provide protection against coastal erosion and flooding and enhance beach building. The disadvantages are that they are large expensive structures to build and maintain, interfere with the natural environment, can be aesthetically unattractive and can limit access to the beach. Also, over time, they can induce beach erosion in surrounding areas. Soft coastal defense solutions have the advantages of being less expensive, more environmentally friendly, and less intrusive. They encourage beach access and can be aesthetically pleasing. Disadvantages are that they may need frequent

Coast Defenses, Fig. 1 Seawall at Swansea Bay, Swansea, United Kingdom

Coast Defenses, Fig. 2 Rock Groyne, Hambatota, Sri Lanka

maintenance and may not work at sites where wave activities are very strong. Advantages and disadvantages of coastal defense options should be carefully studied before adopting them into a specific site (Kamphuis 2002). In recent times, mainly considering environmental sustainability, soft engineering solutions are preferred as coastal defenses.

Cross-References

▶ Abrasion
▶ Aeolian Processes
▶ Angle of Repose
▶ Armor Stone
▶ Artificial Ground
▶ Beach Replenishment
▶ Bedrock
▶ Biological Weathering
▶ Climate Change
▶ Coastal Environments
▶ Crushed Rock
▶ Current Action
▶ Erosion
▶ Floods

▶ Foundations
▶ Gabions
▶ Geotechnical Engineering
▶ Hazard
▶ Land Use
▶ Landforms
▶ Levees
▶ Marine Environments
▶ Nearshore Structures
▶ Retaining Structures
▶ Risk Assessment
▶ Rock Coasts
▶ Sand
▶ Sea Level
▶ Sediments
▶ Site Investigation
▶ Tsunamis

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Coastal Environments

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Synonyms

Coastal zones

Definition

Coastal environments are those bordering the shoreline on both the seaward and landward sides, which is the geometric place where land, sea, and atmosphere meet and therefore encompasses a variety of subenvironments.

Introduction

Shorelines extend along the five continents for approximately 440 million km, crossing the Equator, Tropics, and Arctic circles, but this only partially explains the variety of coastal environments. Postglacial sea level rise (approx. 120 m) submerged many continental landforms, with headlands, bays, and beaches, and then migrated landwards and either welded to the continent or formed barrier islands.

Further, rivers have long fed the coast with sediments produced by erosion within their watershed forming coastal plains and deltas; at the land/water interface beaches, accumulations of loose material varying in grain size from mud to boulders are being continuously reshaped by waves, winds, and subject to tides.

Inherited from the eustatic rise and reshaped by wave erosion and by weathering, rock coasts (see "▶ Rock Coasts") tremendously increase coastal variability, contributing to the scenic value of such environments. Whereas cliff erosion is an irreversible process, rock coast formation is active in tropical areas, where fringing reefs, platform reefs, atolls, and barrier reefs can grow by several millimeters per year.

Waves

Waves and the energy contained within them are a pivotal factor characterizing and affecting coastal environments. They have a variety of terms and are caused by several independent generating forces (wind, tide, earthquakes, under water landslides, explosions, etc.).

(a) Winds are the main originators (with a wave period shorter than 30 s (see elsewhere)) blowing on the sea surface, they accumulate transferred energy (E), which is proportional to deep sea wave height (H):

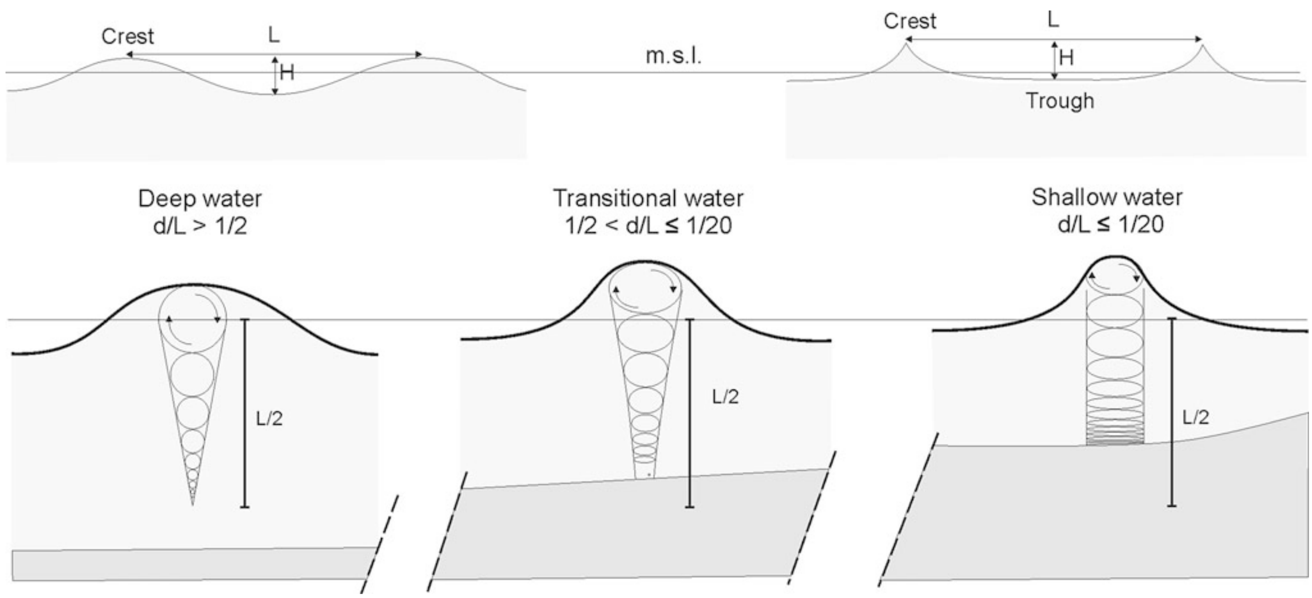
$$\mathrm{E} = 1/8\rho g H^2$$

where ρ is the water density and g gravity acceleration.

In a deep water wave, water particles move in circular orbits, with a decreasing radius with depth, so that at a water depth 1/2 of the wavelength motion is practically insignificant (Fig. 1).

These waves move from the generation area at a speed defined as:

$$\mathrm{C}_0 = \sqrt{1,56\mathrm{L}_0}$$



Coastal Environments, Fig. 1 Water orbits and wave profile deformation while shoaling

where L_0 is the wave length in deep water; the longer the wave length the faster it moves.

Approaching the coast, water depth decreases and water particle motion is subject to friction, that is, the orbits flatten so that at the seabed they produce a forward and backward movement.

Waves slow down, but their period (the time taken for successive peaks/troughs to pass a stationary object) cannot change and reduced velocity implies shorter length waves. As waves enter shallow water, they "peak up" or increase in height together with a decreased wave length resulting in a steepened wave (steepness = H/L; maximum 1:7). If the crest angle decreases, the wave becomes unstable with the crest moving more rapidly than the water beneath it, so it collapses and the wave breaks. The water depth at which this occurs (h_b) is generally considered proportional to the wave height at breaking (H_b), with $\gamma = 0.78$ and $H_b = \gamma h_b$ and $C = \sqrt{g.d}$.

(b) Tsunamis waves are generated by seismic activity or submarine landslides; they have very long wavelengths and periods and can be extremely destructive.
(c) Tidal waves caused by astronomical forces are formed as a function of the periodic movement of the Moon around the Earth (the Sun plays a very small part).

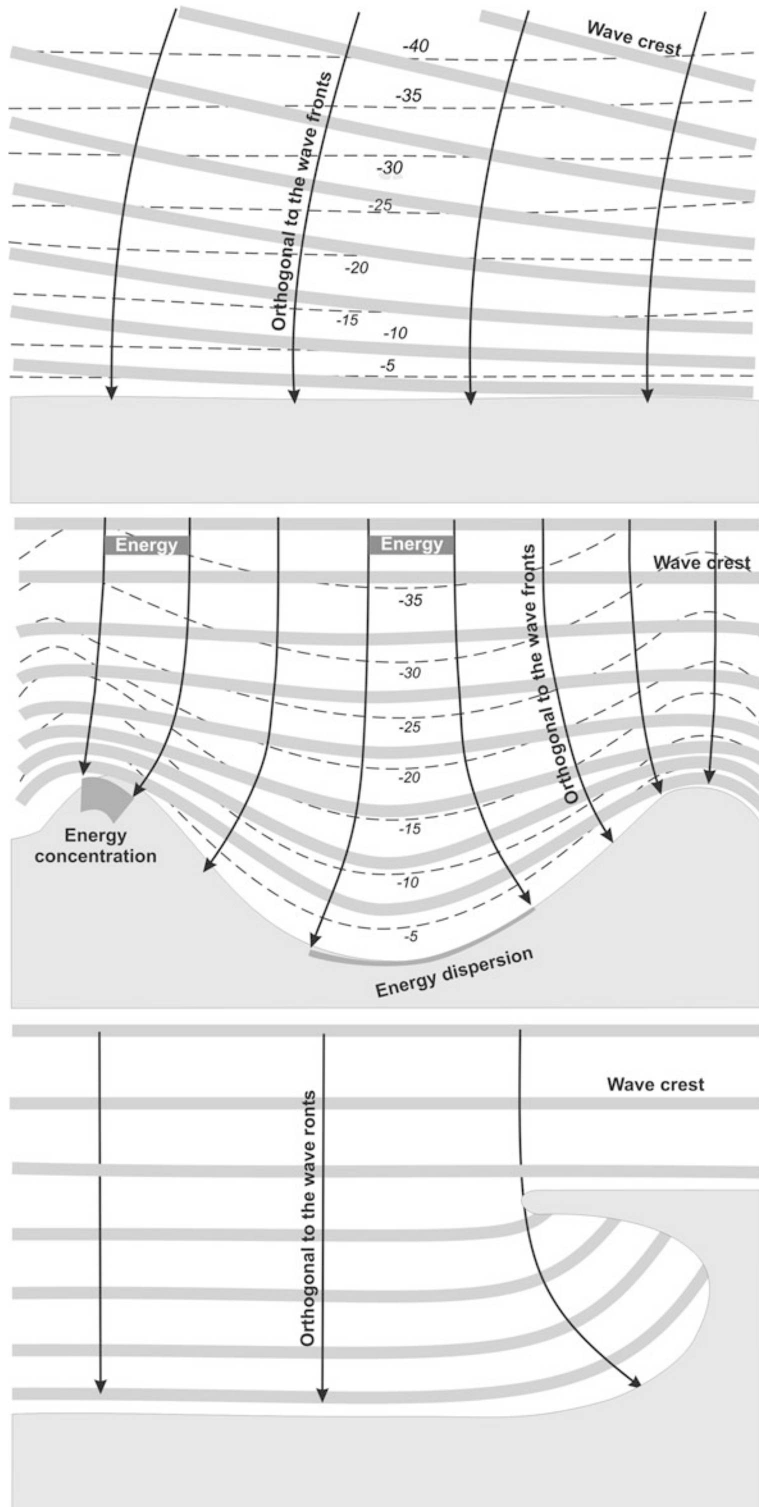
When waves approach obliquely with respect to the coast, the part nearest the coast slows down earlier than the remaining part due to bottom friction; in this manner waves rotate to fit the coast (wave refraction; Fig. 2 top). If the near-shore has a mild slope or waves are long, their rotation can complete otherwise they arrive at the shore with a certain angle. The long-shore component of the energy, mostly near the breaker zone, is responsible for sediment transport along the coast.

For the same process, headlands and bays receive different wave energy, due to concentration or dispersion while shoaling (Fig. 2 center). Waves passing through a straight, or through the gap between two detached breakwaters, generate circular waves (wave diffraction). After bypassing headlands and anthropogenic structures stretching into the sea (e.g., jetties and breakwaters), waves enter the sheltered area with a circular planform (Fig. 2 bottom).

Beaches

The bulk of the world's beaches consist of sediments delivered by rivers to their mouth and moved longshore by wave-induced currents. River input is modified by human activities (e.g., river dams, changing land use patterns, riverbed quarrying, land reclamation), all operating with different intensities, and sediments can be trapped by coastal structures in their longshore movement. With more than 70% of the world beaches eroding and 50% currently under threat because of excessive development (Pilkey and Cooper 2014), the interest of geoscience in this environment is pronounced.

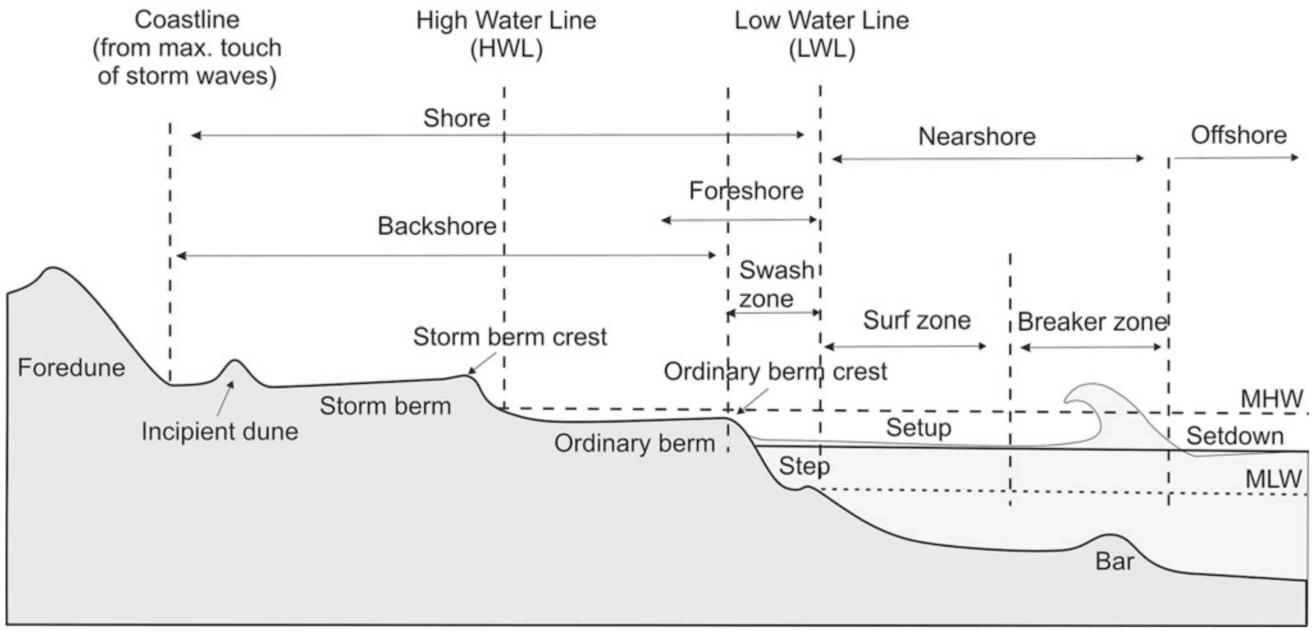
Beach profile, in the emerged and submerged part, depends on grain size and wave energy. The coarser the sediments the steeper is the profile; high energy waves



Coastal Environments, Fig. 2 Offshore oblique waves refracting while shoaling in a linear (top) and in an embayed coast (center); wave refraction on the leeside of an obstacle (bottom)

produce gentler slopes, that is, storm waves flatten the beach, whereas low energy waves build up the beach, or steepen the gradient. Following wave characteristic changes, beach profiles modify with cross-shore sediment displacement.

After breaking, water in the upper layers moves towards the beach and a return flux runs near the bottom, which moves sediment offshore and these tend to accumulate beneath the breaker line forming a bar (Fig. 3).



Coastal Environments, Fig. 3 Beach profile and terminology

During storms, when higher waves break in deeper water, bars form far from the beach, but this material returns onshore with milder constructive waves.

In the swash zone, wave uprush transports grains, a part of which is abandoned on a ridge because backwash is less rapid and part of the water infiltrates into the sediments. The higher the waves, the higher is the ridge; milder storms produce lower ridges welding to the former. In this manner, the beach develops a stepped profile, with each step slowly deepening landward, but steepening seawards.

In a prograding beach, storm berm crests can grow intercepting wind-driven sediments forming foredunes; flanking one another these linear and parallel rises form beach-ridge or foredune-ridge plains, where interdune swales and ponds can be present. On river deltas, they converge to the apex and the innermost trace the old coastlines. The proposed beach profile model is valid for microtidal environments, but when tidal range is significant, wave-induced processes shift cross shore; the most evident result being a low-tide terrace and the absence of features such as berm crests and steps.

The basic framework for the morpho-dynamic states of a beach is inherited from earlier states (Short and Wright 1983), each state distinguished by a different association of morphology, circulation, and behavior. Different morphological types relate to stages in erosional or accretional sequences and the two extreme types are: *Dissipative* (flat, shallow beaches with relatively large subaqueous sand storage) and *Reflective* beaches (steep beaches, with small subaqueous sand storage). The surf scaling parameter (ε) divides them:

$$\varepsilon = 2\pi a/gT \cdot \tan^2\beta$$

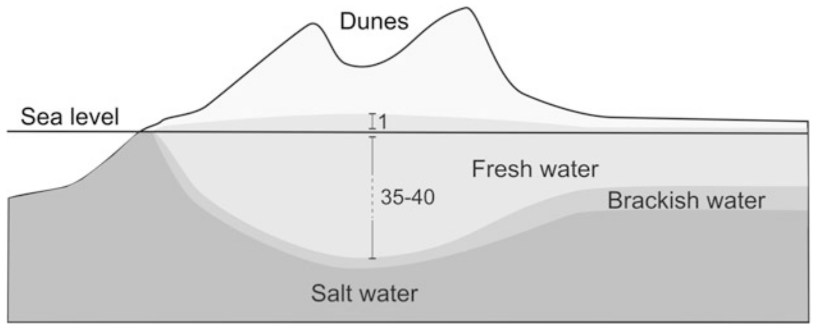
where a = wave amplitude, T = wave period, g = gravity component, β = beach slope.

Reflection dominates when $\varepsilon < 2.5$, and for $\varepsilon > 2.5$ waves begin to plunge dissipating energy, and at $\varepsilon > 20$ spilling breakers occur, surf zone widens, and turbulent dissipation of incident wave energy increases.

The dissipative extreme has high ε values across the surf zone and beach face (30 to >100), spilling breakers, and surf zones across which bores decay progressively to become smaller and smaller.

Coastal Dunes

Offshore winds take the finer sand fractions from backshore surfaces and deposit them against obstacles or vegetation, forming incipient dunes, which can further evolve into foredunes (Fig. 3). Since vegetation lives at a specific distance from the shoreline, dunes form parallel to the coast, independent from the prevailing wind direction. Dune growth is a feedback-regulated process, since the higher the dune the stronger the wind on the crest, which in turn erodes its own deposits. In arid environments, or where sand accumulates faster than vegetation grows, dunes can move landward, possibly losing their linear disposition and forming barchans



Coastal Environments, Fig. 4 Dunes as fresh water reservoirs (Ghyben-Herzberg relation)

or seifs. They can climb cliffs (clifftop dunes) and overpass promontories.

Dunes are inherent to any natural sand beach, but high dune formation needs strong offshore winds, fine sand sources, and a high tidal range; in middle latitudes, dunes are better developed on coasts that are exposed to westerly winds. Along Arctic coasts, dune formation is limited given coarse glacial deposits, wet soil, and land-based winds. In the Tropics, it is the vegetation growing on the beach that opposes wind ablation.

Unvegetated dunes can move inland, obstructing communication lines and burying villages, as occurred during the Middle Ages when Via Julia, in Wales, was abandoned. Wind fences are often built to stabilize the sand, generally constructed with wood lathes. Planting, when possible, is a more effective way of dune stabilization and marram (*Ammophila arenaria* or *brevigulata*) grass is frequently used.

Dunes act as water reserves since freshwater is lighter than saltwater: for each meter of water table elevation above sea level 40 m of water will be present in the aquifer below; per the Ghyben-Herzberg relation (Fig. 4). Coastal development negatively impacts dunes, especially flattening or cutting, thus reducing this fresh water resource.

Coastal Plains and River Deltas

When rivers enter the sea, they deposit their bedload, which is reworked by waves and distributed longshore to form beaches and coastal plains. The latter can grow due to alluvial deposits because of floods and river course migration. Uneven sedimentation can give rise to coastal lakes and marshes, whose formation is favored by subsidence (see "▶ Subsidence") characterizing recently deposited materials.

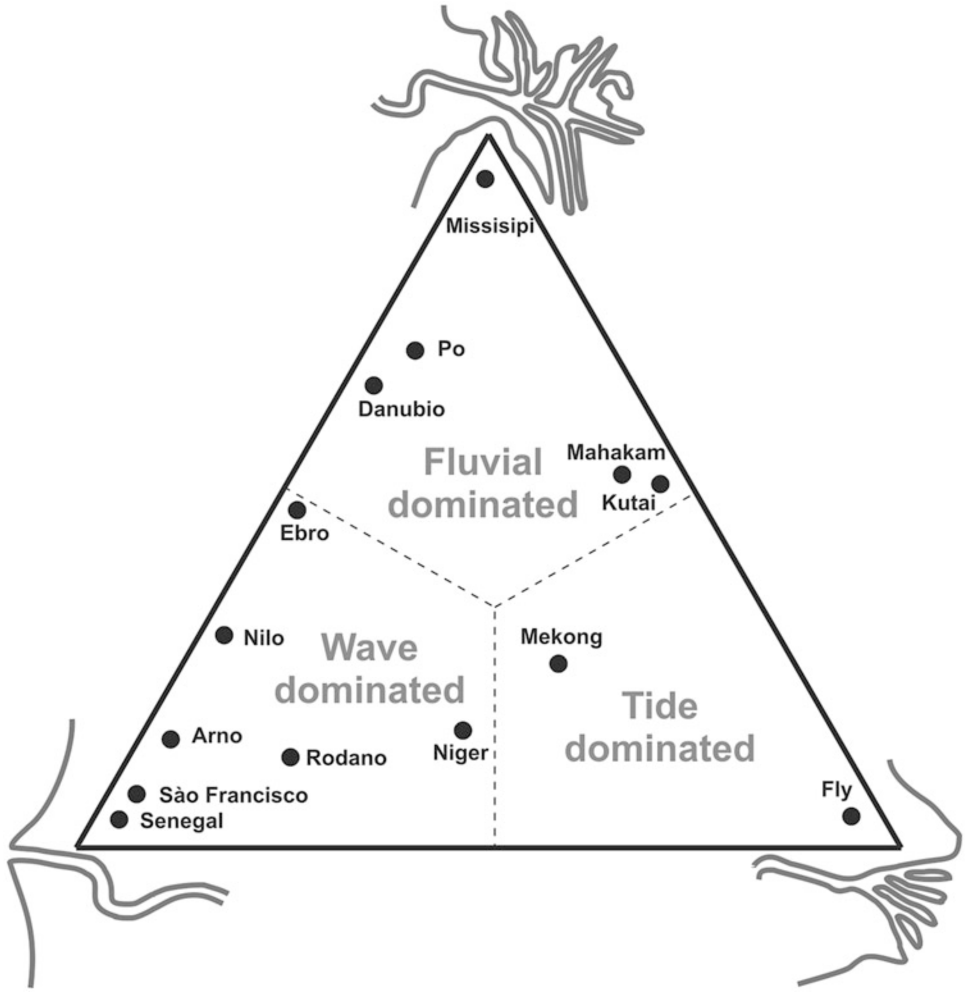
If sediment supply is greater than what waves can move longshore, part of will remain to construct a delta that protrudes into the sea. Delta shape depends on river input, wave energy, and tidal processes, and given these three parameters, a classification is proposed by Galloway (1975) (Fig. 5): wave dominated, fluvial dominated, and tide dominated. The first have cuspate shapes, more or less prominent as a function of the ratio of river input versus longshore transport; the second shows several branches cut by distributaries; and the latter are funneled shaped with elongated sand islands continuously reshaped by ebb and flood tides.

Given an almost constant wave energy and direction over the last few centuries, delta accretion and erosion has been mostly modulated by changing river input and, consequently, by changing soil erosion rates. Providing that most deltas began to form only after sea level stabilization (approx. 6,000 year BP), human impact on their evolution is evident.

Delta progradation accompanied socio-economic development, since humans started to cut forests to expand pasture and agriculture including use of the timber. Furthermore, dam construction, hill slope stabilization, riverbed quarrying, and river diversion have filled coastal lagoons reducing overall coastal sediment input. Consequently, beach erosion first started at river mouths, especially at deltas. Forest cutting in developing countries, which triggers additional soil erosion, reduces coastal accretion or stability since reservoirs built along a river course prevents coarse sediments from reaching the coast (Syvitski et al. 2005).

In the Nile delta, the Rosetta promontory retreated more than 70 m/year following the Aswan dam construction. Elsewhere, over the past 60 years, construction of large reservoirs and soil-conservation practices within the Yellow River basin have reduced sediment flux to the sea by some 90%. The Niger delta is severely eroding with subsidence induced by oil extraction a relevant factor. In Colombia, coastal erosion was approximately 80 m/year on the western side of the Magdalena river delta after the construction of a jetty in 1936, which delivered sediments at the head of a submarine canyon (Correa et al. 2005).

Estuaries are not immune from similar processes: from 1921 to 1965, the average cumulative subsidence in Shanghai (Yellow River estuary) was 1.76 m; through water extraction limitations and artificial recharge, this process has slowed down, but large building construction made subsidence rates jump over 10 cm/year in the city (Wang et al. 2012).



Coastal Environments, Fig. 5 River delta classification after Galloway (1975, modified)

River deltas host a relevant part of the world population, e.g., Ganges, Indo Brahmaputra, Nile, and coastal erosion, increased by land subsidence, severely threatens their future.

Barrier Islands, Spits, and Lagoons

Sediments (sand and/or gravel) bypassing headlands can settle downcoast to form emerged bars connected to the mainland; their tip is frequently bent due to wave diffraction. If the bar geomorphologically welds to the coast, a lagoon forms. Bay barriers close embayments.

Barrier islands are similar deposits, but detached from the coast to which they run almost parallel in a sequence of elongated islands. They border 13% of the world coast and are present in all the continents except Antarctica. In the United States, they run for more than 4,300 km from Maine to Texas, in the North Sea they protect the coast of Germany and The Netherlands, and in the Arctic they occur in the Hudson Bay and on the Barents Sea; others are found in Nigeria, Gabon, Ceylon, Vietnam, and Australia; in the latter where Younghusband Peninsula stretches for 130 km.

Two conceptual models have been proposed for their formation: onshore sediment migration following Holocene sea level rise, with bar emergence (Shepard 1963), and detachment of one or more spits which migrated longshore (Otvos 1970).

Gaps separating islands guarantee water exchange through tidal inlets, where submerged ebb (offshore) and flood (inshore) deltas forms, the latter with a wave dominated shape and the former a tide dominated one. Being flat and narrow overwash can occur during severe storms and hurricanes, as along the US Atlantic coast, this process being part of barrier landward translation.

Coastal Erosion

Most of the world beaches are eroding, and many of these were accreting until the late nineteenth and early twentieth century. Natural and anthropogenic causes join in this process, the latter being dominant. Those factors mentioned for river delta erosion add to the interventions on the coast, which further alter the beach sediment budget. Harbors and jetties interrupting longshore transport are responsible for downdrift erosion. Sand quarrying on the beach to obtain cheap building material is an additional cause of the sedimentary deficit, and this generally illegal activity occurs due to the continuous development of the coastal zone.

Although most beaches are fed by rivers, soft cliff erosion makes sediment available for beach construction; in this case, cliff protection can trigger down coast beach erosion.

Coastal erosion works both on the dry beach, i.e., shoreline retreat, and on the nearshore, with profile lowering to the depth of closure, thus producing a milder sloping profile. When river deltas are involved, their deeper parts do not erode and isobaths often trace the old landforms even when the shoreline is rectified.

Sea level rise is a natural factor contributing to beach erosion, even if produced by human-induced global warming. Bruun (1962) proposed a rule – still under revision – for which the beach profile adapts to sea level rise elevating the same amount within the depth of closure; if sediments are not available (e.g., from the river input), they are eroded from the upper part of the profile, inducing a more severe erosion than that produced by mere flooding. With recent sea level rise (approximately 20 cm in the last century), this component of beach erosion is minor, except on low lying areas, but under the future 2100 year scenario (IPCC 2014), with up to 1 m in sea level rise, this will be the dominant process in many coastal sectors.

Considering that most coastal plains are subsiding and that this process extends offshore, especially where oil and gas extraction is carried out, relative sea level rise will be more harmful.

Severe beach erosion is documented in most countries, except where tectonic uplift is intense and/or isostatic rebound is high. Boulder clay cliffs at Holderness, UK, for example, have been retreating about 2 m/year, and since the eleventh century, 26 villages have disappeared (Sheppard 1913). US Atlantic beaches are eroding mainly because of inlet stabilization via jetty construction. Climate change has been noticeably contributing to beach erosion, for instance, glacier retreat in Iceland deposited outwash sediments inside a moraine dammed lake instead of reaching the coast and thus is indirectly responsible for beach erosion of some 8 m/year (Jóhannesson and Sigurðarson 2005).

Conclusions

The areas bordering shorelines are very dynamic environments that are subject to waves, tides, sea level changes, cliff instability, river sediment input, and increasingly human intervention. Since humans preferentially gather along most shorelines through community settlements, industries, communication, and transportation corridors, etc., a sound knowledge base of the processes affecting coastlines is imperative to ensure prolonged and effective sustainable development.

Cross-References

▶ Abrasion
▶ Aeolian Processes
▶ Angle of Repose
▶ Armor Stone
▶ Artificial Ground
▶ Beach Replenishment
▶ Bedrock
▶ Biological Weathering
▶ Climate Change
▶ Crushed Rock
▶ Current Action
▶ Dams
▶ Erosion
▶ Floods
▶ Fluvial Environments
▶ Foundations
▶ Gabions
▶ Geotechnical Engineering
▶ Groundwater
▶ Hazard
▶ Land Use
▶ Landforms
▶ Levees
▶ Marine Environments
▶ Nearshore Structures
▶ Reservoirs
▶ Retaining Structures
▶ Risk Assessment
▶ Rock Coasts
▶ Sand
▶ Sea Level
▶ Sediments
▶ Site Investigation
▶ Subsidence
▶ Tsunamis

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Coefficient of Uniformity

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Definition

The coefficient of uniformity is a crude shape parameter and is a dimensionless ratio of the diameters of grains of *sediment* or particles of *soil* used to distinguish between well-graded and poorly graded coarse-grained soil using results from *laboratory tests* of *grain size* distribution.

Context

Coarse-grained soil particles have grain sizes larger than 0.075 μm in engineering usage (ASTM 2011), which is the grain size at the boundary between *silt* and *sand*. The formula for the coefficient of uniformity, C_u, is

$$C_u = \frac{D_{60}}{D_{10}} \tag{1}$$

where D_{60} is the diameter of grains in a soil sample which corresponds to the size with 60% by mass of the range of grain sizes being smaller and 40% being larger and D_{10} is the diameter with 10% by mass being smaller and 90% being larger. Typically, the diameters are measured in mm, but the ratio must be dimensionless. A poorly graded soil in engineering has a concentration of particles in a small range of grain sizes and is considered to be uniform. Conversely, a well-graded soil in engineering has a large range of grain sizes with no major concentration of grain sizes. In geology, however, the terms poorly graded and well graded are reversed and use of a dimensionless sorting coefficient, S_o, tends to be preferred:

$$S_o = \sqrt{\frac{D_{75}}{D_{25}}} \tag{2}$$

where D_{75} is the diameter with 75% of the soil mass having smaller grain sizes and D_{25} is the diameter with 25% of the soil mass having smaller sizers. Poorly sorted soils are well graded and have larger S_o values.

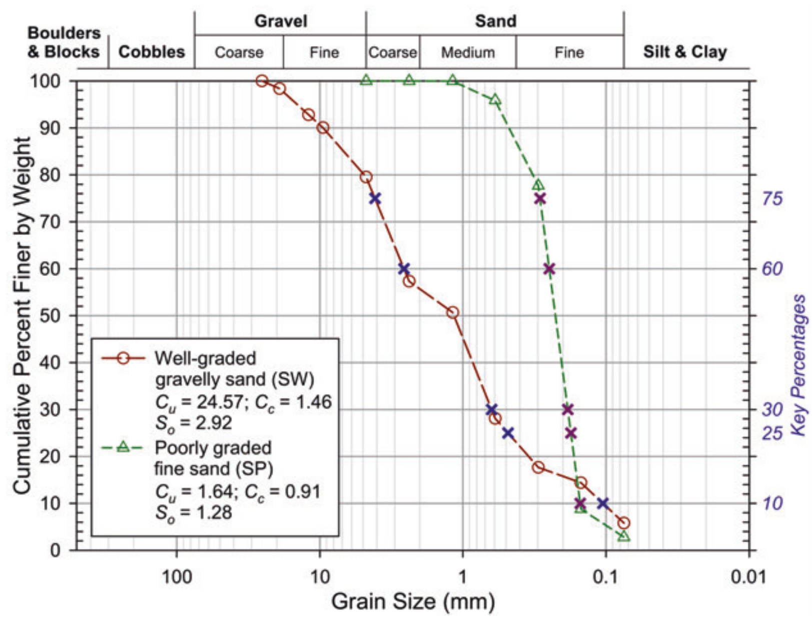
A companion parameter to the coefficient of uniformity is the coefficient of gradation, C_c:

$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} \tag{3}$$

where D_{30} is the diameter with 30% of the soil mass having smaller grain sizes.

The Unified Soil Classification System uses both coefficient of uniformity and coefficient of gradation in the definition of well-graded and poorly graded sand and gravel (Fig. 1). A well-graded sand (SW) or *gravel* (GW) meets the definition of sand or gravel and has $C_u > 4$ and $1 \leq C_c \leq 3$, whereas a poorly graded sand (SP) or gravel (GP) meets the definition of sand or gravel and has $C_u \leq 4$ and $1 > C_c > 3$.

The coefficient of uniformity is an important parameter in engineering geology of relevance to other properties such as *unit weight*, *compressibility*, and *shear strength*.



Coefficient of Uniformity, Fig. 1 Grain-size distribution graph plotted as cumulative percent finer by weight retained on standard US sieves (ASTM 2009). One sample is well-graded gravelly sand from a *Holocene alluvial-fan* deposit, whereas the other sample is poorly graded fine sand from an active sand *dune*; both samples were from locations in the Mohave Desert, California, USA

Cross-References

- ▶ Aeolian Processes
- ▶ Aggregate Tests
- ▶ Boulders
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Coastal Environments
- ▶ Fluvial Environments
- ▶ Gradation/Grading
- ▶ Infiltration
- ▶ Percolation
- ▶ Sand
- ▶ Sediments
- ▶ Shear Strength
- ▶ Silt
- ▶ Soil Laboratory Tests
- ▶ Soil Properties

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Cofferdam

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Synonyms

Coffer; Temporary dam

Definition

A temporary watertight enclosure built for specialized construction below water level.

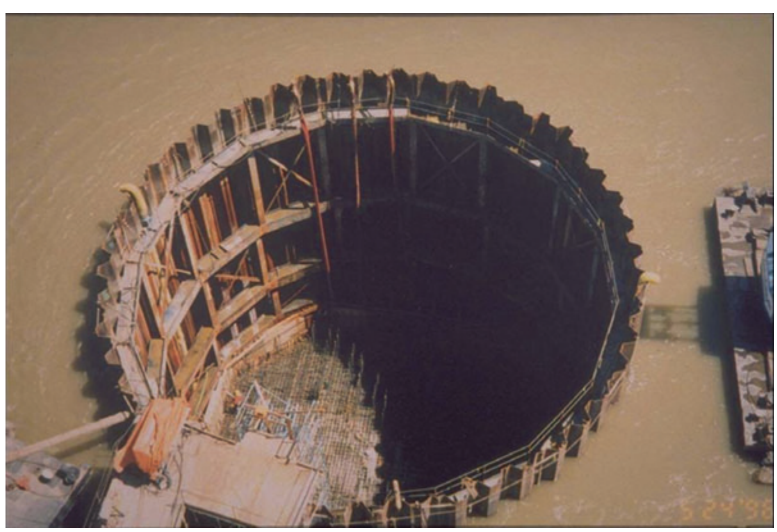
Introduction

Construction in water is a difficult and dangerous job that requires a dry working surface. Cofferdams are one type of temporary structure designed to keep water and/or soil from the execution of construction at a site, so that the permanent facility/structure can be constructed in water (Anderson 2001). A cofferdam should have waterproof walls more than 1 m higher than the maximum water level to ensure that water does not enter the opposite side. Cofferdam design and construction involve the consideration of the structure, local soil and water conditions, often construction offshore, and the possibility of severe weather during construction. The hydrostatic force of the water and the dynamic force due to currents and waves must be considered in the design.

Cofferdams can be classified by their material as earthen, rock-fill, single-walled, double-walled, braced, and cellular cofferdams https://theconstructor.org/water-resources/types-of-cofferdams-construction-details/13807/:

- Earthen cofferdams are used at locations with a water depth less than 3 m and low current velocity. They are built using the locally available soil. The slope is usually 1:1 or 1:2. Rubble stones are pitched on the waterside to protect the embankment.
- Rock-fill cofferdams are preferentially built at locations involving rock. The dam height is up to 3 m, and the slopes are at 1:1.5 to 1:1.25. Rubble stones are also pitched on the waterside to protect the dam from wave action.
- Single-walled cofferdams are commonly used at bridge construction sites and can prevent water intrusion for depths more than 6 meters. They are constructed with wooden or timber sheets placed into the river bed along the perimeter of the area of construction. Steel or iron sheets are also driven into the river bed and placed inside the wooden or timber sheets at equal distances.
- Double-walled cofferdams are typically used when the construction site area is large and water depth is high. Use of single-walled cofferdams becomes uneconomical due to increasing supports. The difference between single wall and double wall cofferdams is that two walls provide extra stability and can hold water as high as 12 m.
- Braced cofferdams are used when driving piles into the bed under the water body is difficult. Two piles are driven into the bed and are laterally supported with the help of wooden cribs installed in alternate courses to form pockets. The empty pockets are filled with stone and earth. The wooden framework of the cofferdam is prepared on ground and then floated to the construction site. The layers of loose material overlying the impervious bedrock are dredged first. Crib is then sunk into position to fit the variation of the surface of the bedrock. The cofferdam is removed after the concrete structure is completed above the water level.
- Cellular cofferdams are used where the water depth is more than 20 m and is the common type of cofferdam used during construction of dams, locks, weirs, etc. (Fig. 1). They are constructed by driving straight web steel sheet piles, all arranged to form a series of interconnected cells that are filled with clay, sand, or gravel to make them stable against various forces. The cells are constructed in various shapes and styles to suit the requirements of the site.

Enclosed cofferdams are commonly used for construction and repair of oil platforms, bridge piers, and other support structures built within or over water. For dam construction, two cofferdams are usually built, one upstream and one downstream of the proposed dam, after an alternative diversion tunnel or channel has been provided for the river flow to bypass the dam foundation area. These cofferdams are typically a conventional embankment dam of both earth and rock-fill, but concrete or some sheet piling also may be used. Typically, upon completion of the dam and associated



Cofferdam, Fig. 1 Cellular (circular) cofferdam structures, the downstream coffer is removed, and the upstream coffer is flooded as the diversion is closed and the reservoir begins to fill. Depending upon the geography of the dam site, in some applications, a "U"-shaped cofferdam is used in the construction of one half of a dam. When complete, the cofferdam is removed, and a similar one is created on the opposite side of the river for the construction of the other half of the dam (Fell et al. 2014).

A cofferdam is also used on occasion in the shipbuilding and ship repair industry, when it is not practical to put a ship in dry dock for repair or alteration. An example of such an application is certain ship lengthening operations. In some cases a ship is actually cut in two while still in the water, and a new section of the ship is floated in to lengthen the ship. Torch cutting of the hull is done inside a cofferdam attached directly to the hull of the ship; the cofferdam is then detached before the hull sections are floated apart. The cofferdam is later replaced, while the hull sections are welded together again. As expensive as this may be to accomplish, use of a dry dock may be even more expensive.

Cross-References

- ▶ Dams
- ▶ Levees
- ▶ Marine Environments
- ▶ Nearshore Structures
- ▶ Retaining Structures
- ▶ Sea Level
- ▶ Water

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Cohesive Soils

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Definition

Cohesive soils are fine-grained, low-strength, and easily deformable soils that have a tendency for particles to adhere. The soil is classified as cohesive if the amount of fines (silt and clay-sized material) exceeds 50% by weight (Mitchell and Soga 2005). Examples of cohesive soils include sandy clay, silty clay, clayey silt, and organic clay.

Characteristics

Cohesive soils have significant cohesive strength and exhibit plasticity. Cohesion between soil particles comes from three major sources, cementation, electrostatic and electromagnetic attraction, and primary valence bonding and adhesion (Mitchell and Soga 2005). The structure of clay in cohesive soil has a great influence in the engineering behavior of soils. The structure of soil refers to the geometric arrangement of soil or mineral particles and depends on genetic, chemical, mineralogical characteristic, as well as past stress conditions of the soil. Interparticle force also influences the soil structure. For cohesive soils, interparticle force is much higher than in noncohesive soils. Most natural clay has highly oriented and dispersed structure due to tectonic activity, by sliding, or by construction activities such as compaction (Terzaghi et al. 1996). Most commonly used characteristics for cohesive soils are boundaries of fine-stratification, grain-size distribution, consistency limits, maximum and minimum density, specific gravity, organic matter, moisture content, dry density, porosity, permeability, void ratio, compression index, and shear strength. The quality of cohesive soil samples is critical for the best geotechnical information and for planning the safe and economic design of structures. Some disturbance sources such as *in-situ* stress, mechanical disturbance, and rebound are difficult to avoid when obtaining an undisturbed sample. Erodibility of cohesive soils is attributed to *in-situ* condition and properties related to soil history (Kimiaghalam et al. 2016).

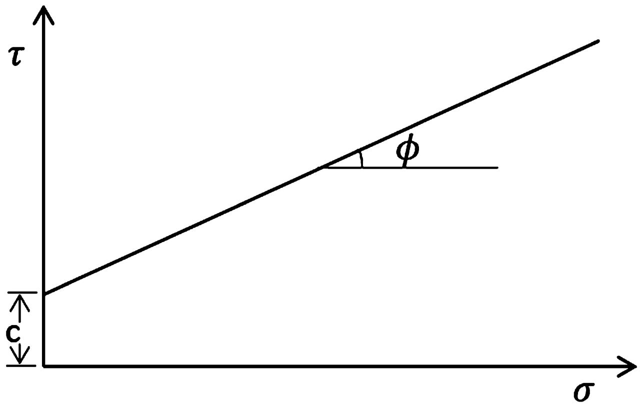
Cohesion and internal friction are strength parameters of soils. The stress-independent component of shear strength is intrinsic cohesion, and the stress-dependent component is the angle of internal friction (Holtz et al. 2011). Coulomb's equation is used to determine the shear strength of soil (Eq. 1, Fig. 1):

$$\tau = c + \sigma \tan\phi \tag{1}$$

where, τ is the shear strength, c is the intrinsic cohesion, σ is the applied normal stress, and ϕ is the angle of internal friction. In this equation, when ϕ is 0, $\tau = c$, and when c is 0, $\tau = \sigma \tan\phi$.

Mohr-Coulomb failure criterion is the commonly used strength criterion applied to soils (Holtz et al. 2011) and is expressed as:

$$\tau_{ff} = c + \sigma_{ff} \tan\phi \tag{2}$$



Cohesive Soils, Fig. 1 Graphic representation of Coulomb's equation

where, τ_{ff} is shear stress on the failure plane at failure and σ_{ff} is normal stress at failure.

The shear strength of cohesive soils is divided into three groups: undrained shear strength, drained shear strength, and drained residual shear strength (Day 2012). Undrained shear strength refers to a shearing condition where water neither enters nor leaves the cohesive soil. Undrained shear strength tests are frequently used in design analysis. Drained shear strength tests allow pore pressure to dissipate completely during the shearing condition. Drained residual shear strength tests are performed for cohesive soils to find the remaining shear strength after a considerable amount of shear deformation has occurred.

Cross-References

- ▶ Characterization of Soils
- ▶ Clay
- ▶ Expansive Soils
- ▶ Mohr-Coulomb Failure Envelope
- ▶ Noncohesive Soils
- ▶ Normal Stress
- ▶ Shear Strength
- ▶ Soil Properties

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Collapsible Soils

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Definition

Soils that have the potential to collapse generally possess porous textures with high void ratios and relatively low densities. At their natural moisture content these soils possess high apparent strength but they are susceptible to large reductions in void ratio on wetting, especially under load. In other words, the metastable texture collapses as the bonds between the grains break down as the soil becomes saturated (Culshaw et al. 2018).

Collapse of soils is controlled both microscopically and macroscopically, and both aspects need to be understood if the controls on collapse are to be determined. When collapse takes place, there is a rearrangement of soil particles resulting in densification. Collapse typically takes place rapidly as the soil passes from a metastable condition to a normally consolidated one.

Jefferson and Rogers (2012) defined collapsible soils as: "*…soils in which the major structural units are initially arranged in a metastable packing through a suite of different bonding mechanisms.*" These structural units include both individual primary minerals (non-clay) and "peds" comprising individual primary minerals with clay mineral coatings and/or clay "bridges" to other particles.

This definition refers to soils that collapse because of their inherent geological and geotechnical characteristics and not to soils which might "collapse" into a void or partially filled void beneath them (e.g., in karst areas caused by dissolution or areas that have been undermined).

Types of Soil Susceptible to Collapse

The most extensive geographically distributed collapsible soils are loesses that are often associated with former and existing glacial environments. Loess consists of generally angular grains that are predominantly of silt size and well sorted. They are derived from continental areas where silty source material was produced by glacial action prior to transportation by wind and subsequent deposition. It is estimated that loess covers between 10% (Jefferson et al. 2001) and 15% (Dibben 1998) of the land surface of the Earth. The main distribution is in the Northern Hemisphere

Collapsible Soils, Table 1 Some geotechnical properties of loessic brickearth soils from south east England. The figure in brackets indicates the number of samples tested (where available) (From Culshaw et al. 2018)

Property	South Essex[a]	Allington, Kent[b]	Allington, Kent (gull-fill)[c]	Ospringe, E of Faversham Kent[d,e]	Pegwell Bay, Kent[f]	Pegwell Bay, Kent[g]	Ford, NE of Canterbury, Kent[g]	Pine Farm Quarry, E of Maidstone, Kent[g]	Reculver, E of Herne Bay, Kent[g]	Northfleet, Kent[g]	Sturry, NE of Canterbury, Kent[g]
Natural moisture content (%)	13–21	–	16–24	15–20 (9)	2–10 (9)	–	–	–	–	–	–
Particle density ($Mg\ m^{-3}$)	2.61–2.77	–	2.61–2.62	2.60–2.71 (8)	–	2.69	2.70	2.70	2.68	2.70	2.69
Bulk density ($Mg\ m^{-3}$)	1.78–2.25	–	1.71–2.04	–	1.55–1.78 (9)	–	–	–	–	–	–
Dry density ($Mg\ m^{-3}$)	1.43–1.99	–	1.38–1.70	–	1.52–1.65 (9)	1.64–1.73	1.49	1.48	1.62	1.61	1.69
Void ratio	0.57–0.82	–	0.54–0.90	–	0.63–0.77 (9)	0.55–0.64	0.81	0.82	0.65	0.68	0.59
Porosity (%)	36–45	–	35–48	–	–	36–39	45	45	39	41	37
Grain-size distribution (%)											
Sand	4–54	12–31 (3)	5–17	5–20 (9)	12–27 (9)	–	–	–	–	–	–
Silt	26–84	67–86 (3)	78–86	43–70 (9)	51–70 (9)	>65	>65	>65	>65	>65	<65
Clay	4–42	<3 (3)	5–14	19–39 (9)	14–22 (9)	–	–	–	–	–	–
Plastic limit (%)	17–24	23–25 (2)	18–23	20–24 (9)	18–21 (11)	17–21	17–20	21–22	19–21	19–20	21-23
Liquid limit (%)	27–64	28–29 (2)	31–34	33–39 (9)	26–32 (11)	28–33	31–45	30–32	32–33	31–33	41–46
Plasticity index (%)	7–40	4–5 (2)	9–16	9–17 (9)	8–12 (11)	11–14	11–28	9–11	12–13	12–13	20–25
Activity	–	–	–	0.23–0.68 (9)	0.48–0.59 (2)	–	–	–	–	–	–
Coefficient of collapsibility	−0.009–0.038	–	−0.0003–0.029	–	–	–	–	–	–	–	–
Angle of friction	11–36	–	–	–	–	–	–	–	–	–	–
Calcium carbonate content (%)	0–16.5 (12)	7.9–8.3	<0.5	–	0–19 (9)	16.2	12.7	14.0	6.0	9.4	–

[a]Northmore et al. (1996)
[b]Lill (1976)
[c]Bell et al. (2003)
[d]Northmore et al. (2008)
[e]Non-calcareous brickearth only
[f]Fookes and Best (1969)
[g]Derbyshire and Mellors (1988)

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from the Central Plains of the USA, across northern Europe and Central Asia to north east China. Non-glacial environment loess is also known (e.g., in desert environments in North Africa, Australia, and elsewhere) but may be less likely to collapse because of the mineralogy, microfabric, and stress history.

In situ loess can often be divided into three layers in terms of relative collapsibility:

- A surface crust that will only collapse if an additional load is applied
- A collapsible layer
- A layer that has collapsed as a result of overburden pressure

Other soils susceptible to collapse include:

- Soils derived from weathered granite (Haskins et al. 1998) and other weathered bedrock (Pereira et al. 2005)
- Some superficial kaolinite deposits (Assadi 2014)
- Tropical red clay soils derived from volcanic ash deposits (Northmore et al. 1992)
- Rapidly deposited and then desiccated debris flow materials (such as some alluvial fans) (Waltham 2009)
- Quick clays (sensitive glaciomarine clays) (Rankka et al. 2004). Quick clays were involved in the Rissa landslide in Norway (Gregersen 1981)
- Some colluvium in semi-arid areas (for example, White and Greenman 2008)
- Cemented, high salt-content soils such as some sabkhas (for example, El-Ruwaih and Tourma 1986)
- Non-engineered fills (compacted dry of optimum) and waste materials such as fly ash (e.g., Bell et al. 2012)

Properties of Collapsible Soils

According to Jefferson and Rogers (2012), the main geotechnical and microfabric characteristics required of most collapsible soils are:

- *an open, metastable structure*
- *a high voids ratio and low dry density*
- *a high porosity*
- *a geologically young or recently altered deposit*
- *a soil with inherent low inter-particle bond strength*

In addition to these, favorable conditions for porous granular soils to collapse include:

- *a well sorted soil, where grains connect together via two points;*
- *a sub-angular shape and rough texture for silt;*
- *a low degree of saturation (structure-based) and hence a high apparent cohesion;*
- *a prolonged application of load smaller than fragmentation load.*

Culshaw et al. (2018) summarized some geotechnical properties of a number of loessic soils from south east England (Table 1). Most of the properties were reasonably consistent considering that the samples were from an area of roughly 4000 km^2. The main identifying property is the silt content that is usually greater than 50% and often greater than 65%. Consequently, plasticity and dry density tend to be low whereas porosity is high.

Risk Reduction for Buildings and Services

The potential for soils to collapse is important, geotechnically, with regard to the effect on foundations and services if unrecognized during the site investigation and if their presence is not incorporated into the building design. Around the world, there are numerous examples where buildings have collapsed and their infrastructure disturbed by a failure to recognize the presence of such soils (e.g., the collapse of

Collapsible Soils, Fig. 1 Derelict house in Torbay, south west England, situated on loessic Brickearth, which has suffered foundation failure probably due to the leaking of a drain or downpipe (© NERC)

Collapsible Soils, Fig. 2 Failure of a building located directly north of Bratislava in Slovakia where uncontrolled drainage led to collapse of the underlying loess soil

apartment blocks built on loess in Volgodonsk, Russia (Fort 2017)). Figure 1 shows a house in Torbay, south west England, built on collapsible loess that has suffered structural damage following a failure of the drainage system beneath the corner of the house. Similarly, Fig. 2 shows the failure of a house located directly north of Bratislava in Slovakia where uncontrolled drainage led to collapse of the underlying loess soil.

Popescu (1992) identified four steps that need to be taken to reduce risk:

- The use of geological and geomorphological information to identify the presence of potential collapsibility
- Classification of the degree of collapsibility, including the use of indirect correlations
- Quantification of the degree of collapsibility using laboratory and/or *in situ* testing
- Engineering options to improve collapsible soils

Jefferson et al. (2005) discussed a series of methods for treating collapsible ground to different depths (Table 2). In general terms, the available approaches are to use very stiff, raft foundations and rigid superstructures (that are expensive), use a flexible foundation and superstructure to accommodate movement, use piles to avoid the collapsible layer by driving through it, or control/alter the ground conditions. Apart from the foundation and superstructure themselves, the methods include pre-wetting, compaction, piling, grouting, chemical methods, and heat treatment.

Collapsible Soils, Table 2 Methods of treating collapsible loess ground (After Jefferson et al. 2005)

Depth (m)	Treatment method	Comments
0–1.5	Surface compaction with vibratory rollers, light tampers	Economical but requires careful site control, for example, limits on water content
	Pre-wetting (inundation)	Can effectively treat thicker deposits but needs large volumes of water and time
	Vibroflotation	Needs careful site control
1.5–10	Vibrocompaction (stone columns, concrete columns, encased stone columns)	Cheaper than conventional piles but requires careful site control and assessment. If uncased, stone columns may fail with loss of lateral support on collapse
	Dynamic compaction; rapid impact compaction	Simple and easily understood but requires care with water content and vibrations produced
	Explosions	Safety issues need to be addressed
	Compaction pile	Need careful site control
	Grouting	Flexible but may adversely affect the environment
	Ponding/inundation/pre-wetting	Difficult to control effectiveness of compression produced
	Soil mixing lime/cement	Convenient and gains strength with time. Various environmental and safety aspects; the chemical controls on reactions need to be assessed
	Heat treatment	Expensive
	Chemical methods	Flexible; relatively expensive
>10	As for 1.5–10 m, some techniques may have a limited effect	(see above)
	Pile foundations	High bearing capacity but expensive

Summary

Collapsible soils are those that are susceptible to a rapid reduction of volume when wetted under load as the soil passes from a metastable condition to a normally consolidated state. The most common type of metastable soil is loess, which is of wind-blown origin and consists of angular grains of silt size that are well sorted. The silt is produced by glacial action before being moved by wind action and redeposited. However, there are a number of other types of soil that are susceptible to collapse particularly some young soils that are recently altered. These include soils derived from weathering of granite and some tropical red clay soils derived from volcanic ash deposits. Some nonengineered fills may also collapse.

To reduce the risk posed by collapsible soils, their presence needs to be identified using geological and geomorphological information, their degree of collapsibility needs classifying and quantifying by geotechnical testing, and then appropriate engineering mitigation measures need to be applied (Stumpf 2013).

Cross-References

- ▶ Atterberg Limits
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Cohesive Soils
- ▶ Compaction
- ▶ Cut and Fill
- ▶ Desert Environments
- ▶ Dewatering
- ▶ Expansive Soils
- ▶ Geochemistry
- ▶ Ground Motion Amplification
- ▶ Hydrocompaction
- ▶ Karst
- ▶ Liquefaction
- ▶ Loess
- ▶ Noncohesive Soils
- ▶ Quick Clay
- ▶ Residual Soils
- ▶ Sinkholes
- ▶ Soil Properties

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Compaction

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Synonyms

Densification; Soil stabilization

Definition

Compaction or densification is reduction in the volume of voids in a soil mass caused by rearrangement of soil particles by mechanical means.

Introduction

Compaction is used as a method of stabilizing soils, that is, improving their properties. Compaction is required when soils are used as a construction material in applications such as structural fill, highway and railroad embankments, earth dams and levees, cover and liner material for sanitary landfills, foundation material, and reclamation of mine waste embankments.

Compaction improves almost all desirable properties of soils. It reduces detrimental settlements, increases soil strength and improves its stability, improves bearing capacity, reduces permeability, and reduces volume changes due to frost action, shrinking, and swelling.

Compaction is measured in terms of dry density (ρ_d), which is defined as the weight of solids (mineral particles) per unit volume. In the field or laboratory, the bulk or wet density (ρ) and water content (ω) are measured first and the dry density is calculated using the following equations:

$$\rho = M_t/V_t \quad (1)$$

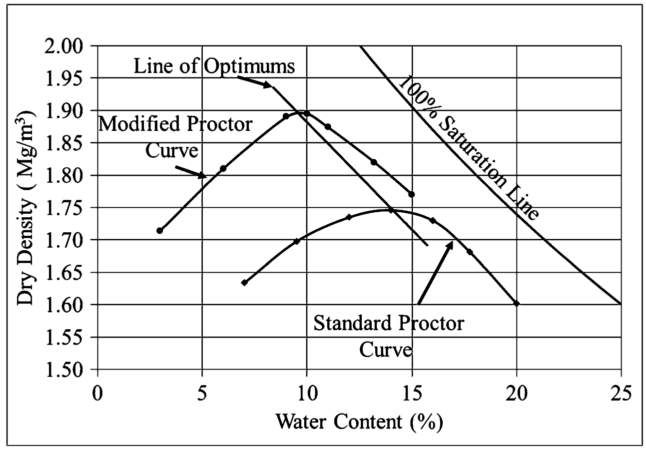
$$\rho_d = \rho/1 + \omega \quad (2)$$

where:

M_t = total mass of soil and V_t = total volume of soil.

Factors Affecting Compaction

According to Proctor (1933), who developed the procedures for compaction, the degree of compaction a given soil can achieve depends on three factors: (1) water content, (2) compactive effort, and (3) soil type (coarse-grained versus fine-grained; grain size distribution; amount and type of clay minerals). Figure 1 shows the effect of water content and compactive effort on dry density. The curves in Fig. 1 are known as the compaction curves. A series of samples at different water contents are tested to establish the compaction curves in Fig. 1. The lower curve shows the results of a standard Proctor test. The peak point of the curve defines the maximum dry density (MDD) and optimum water content (OWC) for the soil tested. The curve demonstrates that, for a given soil and a given compactive effort, a certain amount of water, known as the OWC, is required to achieve the MDD. The curve also shows that the dry density first increases with increasing water content, up to the point of OWC, because, initially, the addition of water facilitates particle rearrangement, resulting in an increase in density. Beyond the OWC, the water causes the soil particles to repel each other, resulting in a drop in dry density. The upper curve in Fig. 1 shows the results of a modified compaction test that involves a higher compactive effort. The curve shows that for a given soil, an increase in compactive effort increases MDD and decreases OWC.

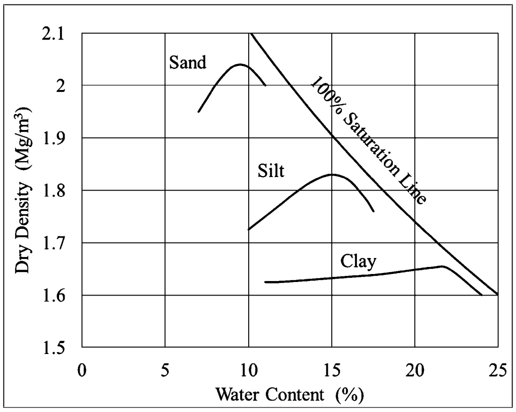


Compaction, Fig. 1 Compaction curves for standard and modified Proctor tests, showing the effect of increasing water content and compactive effort on dry density

Figure 1 also shows the theoretical curve representing the line of 100% saturation. The following equation can be used to establish theoretical curves representing different degrees of saturation (Holtz et al. 2011):

$$\rho_d = \rho_\omega S/[\omega + (\rho_\omega/\rho_s)S] \quad (3)$$

where: ρ_d = dry density, ρ_ω = density of water (1 Mg/m^3/62.4 lb/ft^3), ω = water content (in fraction; e.g., 0.5 for 50%), and S = degree of saturation (in fraction; e.g., 0.1 for 10%). The right sides of the compaction curves in Fig. 1 approach



Compaction, Fig. 2 Effect of soil type on maximum dry density and optimum water content

100% saturation line but never reach it because it is not possible to remove air completely from the voids in the soil.

The line of optimums in Fig. 1 is the line drawn through the peak points of the compaction curves.

Figure 2 shows the effect of soil type on the degree of compaction achievable, using the same compactive effort. It is clear from the figure that well-graded sand and gravel can achieve higher values of MDD at lower values of OWC as compared to high plasticity clays. The strong cohesive bonds in high plasticity clay make it difficult to rearrange the soil particles, even at higher values of OWC. Additionally, granular soils consisting of angular particles can be compacted to higher densities as compared to those consisting of rounded particles.

Typical Values of MDD and OWC

The MDD values for different soils range from 1.3 to 2.4 Mg/m^3(80 to 150 lb/ft^3) with typical values falling between 1.6 to 2.0 Mg/m^3 (100–125 lb/ft^3). The OWC can range from 5% to 40% with typical values being 10% to 20% (Holtz et al. 2011).

Laboratory Tests

In the laboratory, static, vibratory, impact, and kneading methods can be used to compact soils, with the impact method being the most common. The impact test uses a hammer to compact soil in a steel mold in the form of layers. The American Society of Testing and Materials (ASTM) has standardized both the standard Proctor test (ASTM D 698) and the modified Proctor test (D 1557). The specifications are as follows:

Standard Proctor Test

Mold volume = 944 cm^3 (0.033 ft^3)
Rammer weight = 2.49 kg (5.5 lb)
Height of hammer drop = 30.5 cm (12 in)
No. of soil layers = 3
No. of blows/layer = 25
Compactive effort = 600 kN-m/m^3 (12,400 ft-lbf/ft^3) (*ASTM D 698;* ASTM 2010)

Modified Proctor Test

Mold volume = 944 cm^3 (0.033 ft^3)
Rammer weight = 4.53 kg (10 lb)
Height of hammer drop = 45.7 cm (18 in)
No. of layers = 5
No. of blows/layer = 25
Compactive effort = 2700 kN-m/m^3 (56,000 ft-lbf/ft^3) (*ASTM D1557,* ASTM 2010)

Relative Density

The void ratio (e) of a soil is defined as the ratio of the volume of voids to the volume of solids in a mass of soil. The void ratio of a granular soil will be minimum (e_{min}) in its densest state and maximum (e_{max}) in its loosest state. The actual density of a granular soil ranges between these two states. Relative density (D_r), defined by the following equation, is used to indicate the state of compaction of a natural granular soil with a void ratio of e:

$$D_r = [(e_{max} - e)/(e_{max} - e_{min})] \times 100\ (\%) \qquad (4)$$

In terms of maximum dry density ($\rho_{d\ max}$) and minimum dry density ($\rho_{d\ min}$) values, compared to the existing dry density (ρ_d), the relative density can be calculated by:

$$D_r = [(\rho_d - \rho_{dmin})/(\rho_{dmax} - \rho_{dmin})] \times 100\ (\%) \qquad (5)$$

The maximum and minimum dry density or void ratio values can be determined by using ASTM methods D 4253 and D 4254, respectively (ASTM 2010). Based on D_r, a granular soil can be classified as very loose ($D_r < 15\%$), loose ($D_r = 15$–35%), medium dense ($D_r = 35$–65%), dense ($D_r = 65$–85%), and very dense ($D_r > 85\%$) (Holtz et al. 2011). The engineering properties of a granular soil depend on the relatively density. Therefore, laboratory tests should be performed at the same relative density as the *in situ* value.

Field Methods of Compaction

Compaction Equipment and Procedures

In the field, the soil for compaction purposes is excavated from a borrow area using power shovels, draglines, scrapers, and bulldozers. Once transported to the construction site, the soil is spread by bulldozers and graders, in layers 0.33–0.66 m (1–2 ft) thick, known as "lifts." The minimum lift thickness should be at least twice the maximum particle size in the material. Depending upon the natural water content of the soil, the soil is either dried or wetted to bring its water content close to the OWC. The soil layer is then compacted using rollers. The choice of roller depends on the type of soil being compacted. The number of times a roller goes back and forth over the soil layer to achieve the desired density is referred to as the "passes." Commonly used rollers include smooth-wheel rollers, pneumatic or rubber-tired rollers, vibratory rollers (smooth-wheel rollers equipped with a vibratory device), sheepsfoot rollers, tamping foot rollers, and mesh rollers. The smooth-wheel and rubber-tired rollers are suitable for compacting most soils, vibratory rollers are best for granular soils, sheepsfoot and tamping foot rollers that simulate a kneading action are best for compacting cohesive soils, and mesh rollers are most suited for compacting rocky soils and gravels. Details about percent coverage and applied pressures by different types of rollers can be found in Holtz et al. (2011).

Factors that control the degree of compaction include the mass and size of the roller used, the soil characteristics (soil type, initial density, initial water content), lift thickness, number of passes, towing speed, and vibrator frequency in the case of vibratory rollers.

Dynamic compaction and vibro-compaction methods can be used to compact thick, loose, *in situ* deposits of granular soils. In dynamic compaction, a heavy weight (10–40 tons) is repeatedly dropped on the soil from varying heights (10–40 m/33–132 ft) by a crane (Holtz et al. 2011). The depth of influence is given by the following equation (Lukas 1995):

$$D = n\,(W \times H)^{1/2} \tag{6}$$

Where: D= depth of influence (m), n = an empirical coefficient (0.35–0.5, with an average of 0.5), W = weight dropped (megagrams), and H = drop height (m).

The details of dynamic compaction method can be found in Menard and Broise (1975), Leonards et al. (1980), Lukas (1980, 1995), and Holtz et al. (2011).

The vibro-compaction method, used for sand, gravel, and mine spoils, consists of inserting into the soil a device that generates vibration and jets of water. The spacing between vibro-centers ranges from 1 m (3.3 ft) to 3 m (10 ft) and the depth of influence ranges from 10 m (33 ft) to 20 m (66 ft) (Holtz et al. 2011).

Compaction Specifications and Quality Control

Compaction specifications can be either "end product specifications" or "method specifications." For most earthwork projects, end product specifications, including relative compaction (RC) and desired water content, are used. Relative compaction is defined as:

$$\text{Relative compaction (RC)} = \left(\rho_{d\ \text{field}}/\rho_{d\ \max}\right) \times 100\ (\%) \tag{7}$$

A RC value of 95–98% is usually specified with the desired water content being within ±2%.

In method specifications, the type and weight of the roller, the lift thickness, and the number of passes are specified by the project engineer. In this case, the contractor is not responsible for the end product.

In order to ensure if the compacted soil meets the specifications, field tests are performed to measure density and water content. A hole is excavated in the compacted soil, the excavated soil is weighed, the hole volume is measured using either the sand cone or balloon or oil methods (Holtz et al. 2011), and the bulk density is computed and converted to dry density. Additionally, nondestructive methods involving nuclear techniques are frequently used for monitoring the quality of compaction.

Compaction Water Content Versus Soil Properties

Engineering properties of compacted soils, especially fine-grained soils, depend on the compaction water content. Considering the desired properties, engineers can choose one of the three options: (1) compact dry of OWC, (2) compact wet of OWC, and (3) compact at OWC. Fine-grained soils, compacted dry of OWC, usually exhibit brittle behavior, higher strength, higher permeability, and flocculated structure (clay minerals randomly oriented), whereas those compacted wet of OWC are more flexible, exhibiting plastic behavior, but have lower strength, lower permeability, and a more oriented structure. Compacting soils near or at OWC provides the best compromise of all desired properties. If the water content is much higher than the OWC, the soil may be difficult to compact and a rapid decrease in strength may occur due to pore pressure buildup. In such a case, increasing the compactive effort can do more harm than good. Furthermore, density values of granular soils are more sensitive to changes in compaction water content than those of cohesive soils (Fig. 2).

Summary

Compaction is densification of soils by mechanical means, such as rollers. The degree of compaction is measured in terms of dry density. There are three factors that influence the dry density that can be achieved by compaction: (i) water content of soil, (ii) compactive effort or the amount of energy

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transmitted to the soil, and (iii) soil type (grain size distribution, grain shape, plasticity characteristics, etc.). For a given soil and a given compactive effort, the maximum dry density is achieved at a water content known as the optimum water content. The compaction curve established in the laboratory is used to describe specifications for field compaction. Relative density of a soil is usually used to determine the extent of compaction required. The soil in the field is compacted in the form of layers using different types of rollers. Smooth-wheel and rubber-tired rollers are good for all soil types, vibratory rollers are best for granular soils, and sheepsfoot rollers are best for cohesive soils. Dynamic compaction and vibrocompaction methods can be used to compact thick deposits of *in situ* granular soils. Compaction improves all desirable properties when soils are used as highway subgrades and embankments, earth dams and levees, and as a structural fill for foundations.

Cross-References

- ▶ Artificial Ground
- ▶ Atterberg Limits
- ▶ Casagrande Test
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Cohesive Soils
- ▶ Collapsible Soils
- ▶ Compression
- ▶ Cone Penetrometer
- ▶ Consolidation
- ▶ Density
- ▶ Desiccation
- ▶ Deviatoric Stress
- ▶ Dewatering
- ▶ Dynamic Compaction/Compression
- ▶ Expansive Soils
- ▶ Field Testing
- ▶ Fluid Withdrawal
- ▶ Geostatic Stress
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- ▶ Ground Pressure
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- ▶ Liquid Limit
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- ▶ Saturation
- ▶ Shear Strength
- ▶ Shear Stress
- ▶ Soil Field Tests
- ▶ Stabilization
- ▶ Strain
- ▶ Strength
- ▶ Subsidence
- ▶ Voids

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Compression

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Synonyms

Compressibility; Consolidation; Deformation; Settlement; Volume reduction

Definition

Reduction of volume primarily through rearrangement of soil grains in a soil mass from shear and sliding at inter-particle contacts, disruption of particle aggregations, grain crushing, and expulsion of water from the voids of the soil.

Introduction

Soil is a multiphase medium consisting of mineral grains which enclose voids that may be filled with gas, liquid, or a combination of both. For most soils, the gas is air and the liquid is water. When stress is applied to a soil sample, its volume decreases. Loads giving rise to stresses can be due to weight of foundations, superstructure, chemical and moisture environments, and temperature. The effects of stress changes are generally the most important and the most studied. Change in volume of the soil mass may be due to: (a) compression of the solid soil particles, (b) compression of liquid/water and air within the voids, or (c) escape of water and air from the voids.

Both the arrangement of particles and particle groups and the forces holding them in place are important. Seepage flow is a function of the rate of loading which can be either drained or undrained. At small effective stresses, the soil is loose whereas high stresses give rise to dense structure. But if the pore pressure is constant, the changes in total and effective stresses will be the same in both cases, and for relatively incompressible soil grains, the change in the volume of the soil is equal to the volume of water expelled from the voids.

The solid particles and the pore water are relatively incompressible and therefore, under the loads usually encountered in geotechnical engineering practice, they will not undergo appreciable volume changes. Therefore the decrease in volume of a saturated soil mass when subjected to stress increase is due almost entirely to an escape of water from the voids (Terzaghi et al. 1996). Some compression also occurs through shifting of position by the soil particles by rolling and sliding under the influence of the applied load. This aspect of compressibility of a soil mass depends on the rigidity of the soil skeleton. That rigidity, in turn, depends on the structural arrangement of the soil particles, and, in fine-grained soils, on the degree to which adjacent particles are bonded together.

As compression occurs in soils, the pore water escapes. The escape of water, according to Terzaghi (1943), takes place in accordance with Darcy's law. This slow escape of water from the pores, gradual compression, and gradual pressure adjustment is called consolidation. When compression takes place in unsaturated soils by mechanical means such as rolling or tamping, it is termed compaction, and mostly air, rather than water, is driven out from the soil pores.

Coarse-Grained Soils

In granular soils, most of the compression takes place during construction of the project and the after-effects are therefore much smaller than in fine-grained soils. However, the impact on foundation engineering depends on the sensitivity of the structure to small differential settlements. But volume changes under high pressures may be substantial in granular materials as reported by Pestana and Whittle (1995) in their study of the compressibility of three sands under high pressure. At low stress levels, the compressibility of sand depends on initial density. However, at higher stress levels, yielding is observed, and the compression curves for a given sand at different initial densities merge into a unique compression line. Particle crushing is the primary cause of the large volumetric strains that occur along the normal compression line. The yield stress is related to particle tensile strength (McDowell and Bolton 1998; Nakata et al. 2001). At a pressure of 700 kPa (100 psi), a compression of 3% is common, and values as high as 6.5% have been measured.

Fine-Grained Soils

Natural clay has normally undergone a natural process of consolidation, having originally been deposited in water and then gradually compressed by the weight of the material deposited above it. The soil is said to be fully or partially consolidated depending on whether or not a state of equilibrium has been reached under the existing overburden pressure. Some clay deposits are overconsolidated, that is, they have been compressed at some time in their geologic history by superimposed loads, such as the ice sheets of the Pleistocene period, or have consolidated because of free draining conditions, as in some lodgement tills, by capillary suction or by lowering of groundwater table. It is also possible for some of the original overburden pressure, which caused the deposit to consolidate, to be removed in the cause of geological history, e.g., by erosion. This will also give rise to overconsolidation.

Total Compression

The total compression of soil deposit under load is composed of three components, that is,

$$\text{Total Settlement} = S_e + S_p + S_s \tag{1}$$

where

$S_e = \textit{Elastic Deformation (immediate settlement)}$
$S_p = \textit{Primary Consolidation}$
$S_s = \textit{Secondary Compression}$

Elastic deformation is due to the deformation of soil and rock grains and the compression of air and water in the void; it is fully recoverable. But primary consolidation by drainage of water and air from the voids allowing compression of soil skeleton is inelastic, time dependent, and only partially recoverable. While secondary compression is due to creep movements – plastic adjustment of soil fabric under a constant effective stress – this is also inelastic, time dependent, but unrecoverable.

The Elastic Deformation of Soils

Computation of immediate settlement assumes that the soil is an elastic, isotropic, and homogeneous mass. These assumptions are fairly valid for cohesionless soils and soils with large modulus. Several methods use an elastic modulus value and a general elasticity equation to calculate settlements in granular soils and immediate settlements in clayey soils. The general equation for displacement at the center of an applied surface loading is of the form:

$$s = \frac{qBI}{E_s} \cdot \left(1 - v^2\right) \tag{2}$$

where

q = applied surface stress ($\Delta\sigma_v$ at $z = 0$)
B = width of the loaded area
I = displacement influence factor
E_s = equivalent elastic modulus
v = Poisson's ratio of the soil

The influence factor (I) depends on surface area geometry, layer thickness, degree of homogeneity, and relative rigidity of the loaded area. Tables and charts for the Influence Values are available (Coduto et al. 2016; Adeyeri 2015) but these can also be generated on a simple spreadsheet using stress distributions. The following parameters are required to evaluate the equivalent elastic modulus (E_s): (1) Poisson's ratio; (2) shear wave velocity, V_s; and (3) modulus degradation value, E/E_o. A value of 0.1 for Poisson's ratio may be assumed for drained loadings in granular materials at working load levels (Sabatini et al. 2002).

Process of Consolidation

When a load is applied to a saturated clay-water system and the pore water is not allowed to escape, virtually all the applied pressure is transferred to the pore water as excess pore pressure. If drainage is permitted, the resulting hydraulic gradients initiate a flow of water out of the clay mass, and the mass begins to compress as the applied pressure is gradually transferred to the soil skeleton which, in turn, causes a reduction in the excess pore pressure. This process of compression, fluid flow, and pressure transfer is known as consolidation (Terzaghi 1943). If the rate of compression is controlled solely by the resistance to the flow of water under induced hydraulic gradients, the process is referred to as primary consolidation. The mechanism of compression may be accompanied by fracturing in coarse-grained soils and in fine-grained soils by compression and swelling of the clay particles.

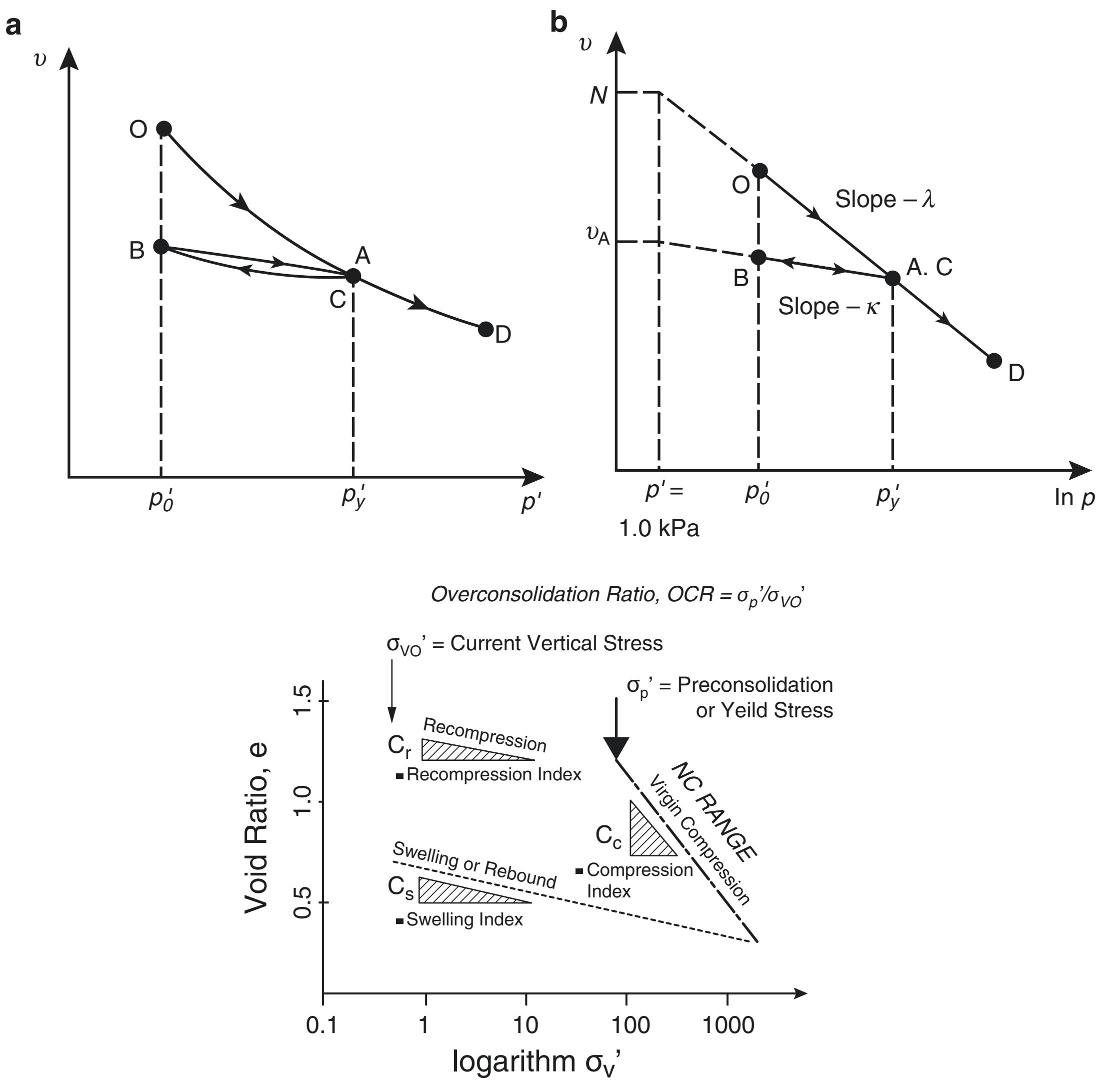
Consolidation Test Analysis

Consolidation is time-dependent. The rate of consolidation or excess pore pressure dissipation is a function of the permeability of the soil. The analysis of this process is usually based on Terzaghi's theory of one-dimensional consolidation which assumes that the squeezing out of pore water is in accordance with Darcy's law (Terzaghi et al. 1996). The general consolidation properties of cohesive soils are typically evaluated in the laboratory using the one-dimensional consolidation test. The most common laboratory method is the incremental load oedometer (ASTM D-2435). Consolidation tests often require high-quality undisturbed samples using thin-walled tubes, piston samplers, or other special samplers for meaningful quantitative predictions of field behavior.

The test results are usually shown in a graph of void ratio (e) versus applied vertical effective stress (σ_v') in the oedometer, either on arithmetic (a) or logarithm of stress (b, c) plot as shown in Fig. 1. The loading begins at O. When stress is removed from the consolidated soil at A, the soil will rebound to B, regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve BC, defined by the recompression index and will continue to D joining the virgin curve at C. The soil which had its load removed is considered to be *overconsolidated*. The highest stress that it has been subjected to is termed the preconsolidation stress. The *overconsolidation ratio (*OCR) is defined as the highest stress experienced divided by the current overburden pressure/stress.

Each of the compression indices (Fig. 1c) is defined by the change in void ratio per log cycle stress ($= \Delta e/\Delta \log\sigma_v'$) for the respective ranges of recompression (C_r), virgin compression (C_c), and swelling or rebound (C_s). An alternate version of presenting consolidation test results is using a plot of vertical strain as the ordinate axis, whereby $\varepsilon_v = \Delta e/(1 + e_o)$. In this case, the compression indices are reported as the recompression ratio, $C_{r\varepsilon} = C_r/(1 + e_o)$, and compression ratio, $C_{r\varepsilon} = C_c/(1 + e_o)$.

Normally the range of soil void ratio is about 0.5–4.0 (Lambe and Whitman 1969). Although the range of pressures of interest in most cases (up to a few hundred kilopascals) is relatively small on a geological scale, the void ratios encompass virtually the full range from fresh sediments to shale. The reduction in volume during consolidation leads to densification, the process which is accompanied and influenced by mechanical and chemical changes in the soil. In general, the void ratio–effective pressure relationship is



Compression, Fig. 1 Isotropic compression and swelling (© Definition of Cc, Cr, Cs, and σp′ Sabatini et al. 2002)

related to grain size and plasticity (Lambe and Whitman 1969). According to Meade (1964), particle size and shape, which together determine specific surface area, are the most important factors influencing both the void ratio at any pressure and the effects that physicochemical and mechanical factors have on consolidation and swelling. Particle size and shape are direct manifestations of composition, with increasing colloidal activity and expansiveness associated with decreasing particle sizes.

The rate of loading and time have significant effects on the equilibrium void ratio–effective stress relationship, especially for sensitive structured clays, and also on the measured preconsolidation pressure (Mitchell and Soga 2005). The preconsolidation pressure decreases as the duration of load application increases and as the rate of deformation decreases (Leroueil et al. 1990). The higher values of apparent preconsolidation pressure associated with the faster rates of loading reflect the influences of the viscous resistance of the soil structure.

Values of compression index, C_c, can vary from less than 0.2 to as high as 17 for specially prepared sodium montmorillonite under low pressures, although values less than 2.0 are usual. The compression index for most natural clay is less than 1.0, with a value less than 0.5 in most cases. The swelling index, C_e, is less than the compression index, usually by a substantial amount, as a result of particle rearrangement during compression that does not recur during expansion. After one or more cycles of recompression and unloading accompanied with some irrecoverable volumetric strain, the reloading and swelling indices measured in the pre-yield region become nearly equal. Swelling index values for three clay minerals, muscovite, and sand are listed in Table 1. For undisturbed natural soils, the swelling index values are usually less than 0.1 for nonexpansive materials and more than 0.2 for expansive soils.

Compression, Table 1 Swelling index values for several minerals (Olson and Mesri 1970)

Mineral (1)	Pore fluid, adsorbed cations, electrolyte concentration, in gram equivalent weights per liter (2)	Void ratio at effective consolidation pressure of 100 psf (5 kPa) (3)	Swelling index (4)
Kaolinite	Water, sodium, 1	0.95	0.08
	Water, sodium, 1×10^{-4}	1.05	0.08
	Water, calcium, 1	0.94	0.07
	Water, sodium, 1×10^{-4}	0.98	0.07
	Ethyl alcohol	1.10	0.06
	Carbon tetrachloride	1.10	0.05
	Dry air	1.36	0.04
Illite	Water, sodium, 1	1.77	0.37
	Water, sodium, 1×10^{-3}	2.50	0.65
	Water, calcium, 1	1.51	0.28
	Water, sodium, 1×10^{-3}	1.59	0.31
	Ethyl alcohol	1.48	0.19
	Carbon tetrachloride	1.14	0.04
	Dry air	1.46	0.04
Smectite	Water, sodium, 1×10^{-1}	5.40	1.53
	Water, sodium, 5×10^{-4}	11.15	3.60
	Water, calcium, 1	1.84	0.26
	Water, sodium, 1×10^{-3}	2.18	0.34
	Ethyl alcohol	1.49	0.10
	Carbon tetrachloride	1.21	0.03
Muscovite	Water	2.19	0.42
	Carbon tetrachloride	1.98	0.35
	Dry air	2.29	0.41
Sand			0.01–0.03

Overconsolidation Ratio

The maximum preconsolidation stress delineates the region of semi-elastic behavior (corresponding to overconsolidated states) from the region of primarily plastic behavior (associated with normal consolidation). The degree of preconsolidation is expressed using the overconsolidation ratio, where OCR $= \sigma_p'/\sigma_{vo}'$. Other important parameters obtained from the consolidation test include the constrained modulus ($D = \Delta\sigma_v / \Delta\varepsilon_v = 1/a_v$), and time-dependent parameters: coefficient of consolidation (c_v) and coefficient of secondary compression (C_α). The preconsolidation stress can be estimated from the $e - \log\sigma_v'$ relationship using the Casagrande (1936) graphical technique or by the strain-energy method which involves plotting the cumulative strain energy (i.e., the product of stress times strain) for each load increment in a laboratory consolidation test. The point where the strain energy plot exhibits a large incremental increase represents the preconsolidation stress for the soil.

If we have a soil whose preconsolidation pressure is equal to the overburden, such a soil is said to be normally consolidated. If the preconsolidation pressure is greater than the overburden (i.e., $\sigma_p' \gg \sigma_o'$), we have a case of overconsolidation or preconsolidation. Usually when

$$\text{OCR} = 1 \text{ the soil is normally consolidated}$$

$$\text{OCR} > 1 \text{ the soil is overconsolidated}$$

$$\text{OCR} < 1 \text{ the soil is underconsolidated}$$

Underconsolidated ($\sigma_p' < \sigma_o'$): The soil has not yet reached equilibrium under the present overburden owing to the time required for consolidation. Underconsolidation can result from such conditions as deposition at a rate faster than consolidation, rapid drop in the groundwater table, insufficient time since the placement of a fill or other loading for consolidation to be completed, and disturbance that causes a structure breakdown and decrease in effective stress.

Normally Consolidated ($\sigma_p' = \sigma_o'$): The soil is in effective stress equilibrium with the present overburden effective stress. Few deposits are exactly normally consolidated but most are at least very slightly overconsolidated for reasons mentioned under overconsolidated soils. Underconsolidated soil often behaves as normally consolidated soil until the end of primary consolidation, and overconsolidated clay becomes normally consolidated clay when loaded beyond their maximum past pressure (Mitchell and Soga 2005).

Overconsolidated or Preconsolidated ($\sigma'_p > \sigma'_o$): The soil has been consolidated, or behaves as if consolidated, under an effective stress greater than the present overburden effective stress. Overconsolidation in soil can be due to a number of reasons which include: (a) change in total vertical stress: this can be as a result of geologic deposition of soil followed by natural erosion, removal of part of the overburden as a result of excavation, past structure; (b) changes in pore water pressure due to change in water table elevation, artesian pressure, deep pumping, or desiccation due to surface drying or plant life; (c) change in soil structure due to secondary compression (aging); (d) chemical alteration of the soil due to weathering, precipitation, cementing agents, ion exchange; (e) environmental changes such as pH, temperature, and salt concentration; and (f) change in strain rate of loading. An accurate knowledge of the maximum past consolidation pressure is needed for reliable predictions of settlement and to aid in the interpretation of geologic history of such soils. The Casagrande construction method is routinely used for this. If the recompression to virgin compression curve does not show a well-defined break, the preconsolidation pressure may be difficult to determine. Gentle curvature of the compression curve over the preconsolidation pressure range is characteristic of sand, weathered clay, heavily overconsolidated clay, and disturbed clay. Sample disturbance, in fact, has the effect of lowering the value of the preconsolidation pressure in sensitive clay.

Factors Which Affect Compression

Many factors including environmental and compositional factors have some effects on volume change of soils. Mitchell and Soga (2005) have clearly explained the effects of many such factors on soil compression including: physicochemical interactions between particles, physical interactions between particles, chemical and organic environment, mineralogical detail, fabric and structure, stress history, temperature, pore water chemistry and stress path.

Physical and Physicochemical Interactions

Compression is due to particle rearrangements from shear and sliding at interparticle contacts, disruption of particle aggregations, and grain crushing. Therefore the structure of the soil and the forces holding the soil particles together are important. Swelling on the other hand depends strongly on physicochemical interactions between particles, but fabric also plays a role. The physical interactions between particles include bending, sliding, rolling, and crushing. In general, the coarser the gradation, the more important are physical particle interactions relative to chemically induced particle interactions. Particle bending is important in soils with platy particles. Even small amounts of mica in coarse-grained soils can greatly increase compressibility. For example, addition of mica flakes to mixtures of a dense sand having rounded grains can even make the compression and swelling curves look like those of clay (Fig. 1). Usually cross-linking makes soil fabric more rigid, especially clay containing platy particles. Particles and particle groups act as struts whose resistance depends both on their bending resistance and on the strengths of the junctions at their ends. According to van Olphen (1977), cross-linking is important even in “pure clay” systems. The importance of grain crushing increases with increasing particle size and confining stress magnitude. Particle breakage is a progressive process that starts at relatively low stress levels because of the wide dispersion of the magnitudes of interparticle contact forces. The number of contacts per particle depends on gradation and density, and the average contact force.

The chemical environment influences surface forces and water adsorption properties, which, in turn, increase the plasticity and compressibility of soils, and this explains why expansion of pyrite minerals in some shale and other Earth materials as a result of oxidation caused by exposure to air and water has been the source of significant structural damage of some projects (Bryant et al. 2003). Also compacted expansive soils with flocculent structures may be more expansive than those with dispersed structures. Seed et al. (1962) in their study of the effect of structure and electrolyte concentration of absorbed solution on swelling of compacted clay reported that at pressures less than the preconsolidation pressure, the soil with a flocculent structure was less compressible than the same soil with a dispersed structure and that increased electrolyte concentration in the water imbibed by a compacted clay resulted in reduced swelling. Usually a change in the pore solution chemistry that depresses the double layers or reduces the water adsorption forces at particle surfaces reduces swell or swell pressure. And the amount of compression or swelling associated with a given change in stress usually depends on the path followed.

Cementation Bonding

Shearing of coarse-grained soils at the same void ratio but with different initial fabrics gives rise to different volume changes. These volume changes develop from different methods of sample preparation and are manifested by differences in liquefaction behavior under undrained loading. This is reflected in the work of Cuccovillo and Coop (1999) on natural intact cemented calcarenite sand. The initial compressibility before yielding of the sand is stiff due to cementation. If the cementation is stronger than the particle crushing strength, the compression line will lie to the right of the normal compression line of the uncemented reconstituted sand. However, if the cementation is weaker than the particle crushing strength, the compression curve will merge gradually with that of the uncemented sand before yielding (Cuccovillo and Coop 1999). This highlights the importance of relative strengths of cementation bonding and particles on the compression behavior of structured soils.

Consolidation Theory

The process and theory of one-dimensional consolidation under saturated and unsaturated conditions as proposed by Terzaghi (1943) and Fredlund et al. (2012), respectively, are well-explained by Macciotta (2016). The theory was the first organized body of knowledge which explained the physics of the compression process and provided an analytical method for predicting the magnitude and time rate of compression. The theory was based on a number of simplifying assumptions and hypothesis including assuming that the rate of volume decrease is controlled totally by hydrodynamic lag. The governing equation for saturated conditions (Terzaghi 1943; Adeyeri 2015) is

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \tag{3}$$

where

u = the excess pore pressure
t = time
z = distance from a drainage surface
c_v = the coefficient of consolidation

The coefficient of consolidation is given by

$$c_v = \frac{k_h(1+e)}{a_v \gamma_w} \tag{4}$$

where

k_h = the hydraulic conductivity
a_v = the coefficient of compressibility which is given as

$$a_v = \frac{\Delta e}{\Delta \sigma'} = \frac{e_1 - e_2}{\sigma'_2 - \sigma'_0} \tag{5}$$

The more commonly used consolidation parameter in geotechnical engineering is the coefficient of volume compressibility, m_v. The coefficient of volume compressibility is the volumetric strain in a clay element per unit increase in stress and represents the compression of the soil per unit original thickness due to a unit increase of the pressure. It is given by

$$m_v = \frac{\Delta e}{(1+e_0)} \frac{1}{\Delta \sigma'} \tag{6}$$

The units of m_v are the inverse of pressure (m^2/kN). It depends on the stress range of the linear part of the $e - \log\sigma$ curve. C_v is the same for any stress range on the linear part of the $e - \log\sigma$ curve.

Solutions of Eq. 3 for different boundary conditions can be found in standard geotechnical engineering textbooks (Adeyeri 2015; Coduto et al. 2016) in terms of a dimensionless depth z/H (where H is the maximum distance to a drainage boundary) and a dimensionless time factor $T = c_v t/H^2$ for different boundary conditions. One analytical solution in the form of a Fourier series (Taylor 1948) is given as

$$u = 1 - \sum_{m=0}^{\infty} \frac{2}{M} e^{-M^2 T_v} \tag{7}$$

where

$$M = \pi\,(2\,m + 1)/2$$

According to Taylor (1948), the following approximation is possible.

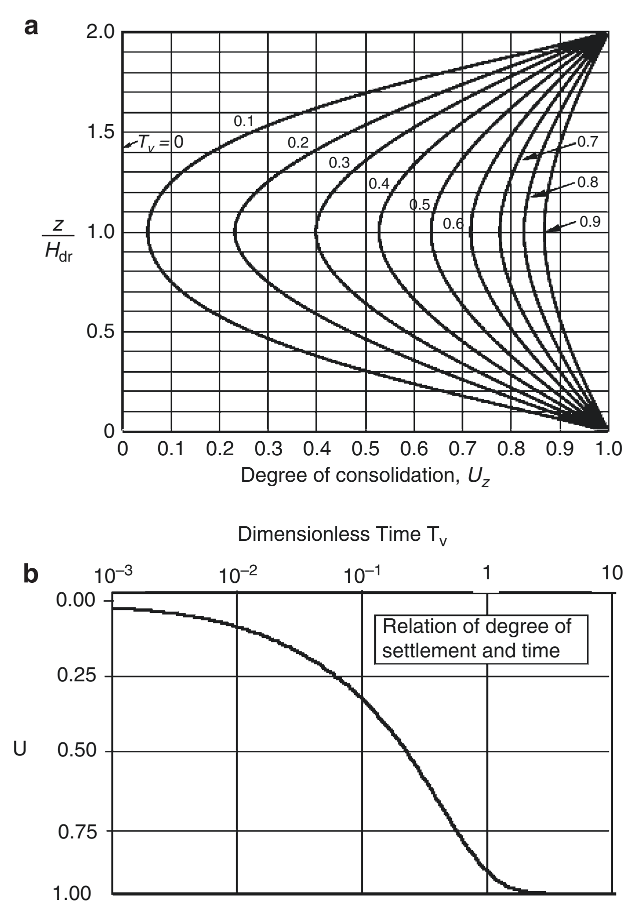
$$T_v = \frac{\pi}{4} U^2 \qquad \text{when} \quad U \leq 60\% \tag{8a}$$

$$T_v = 1.781 - 0.933\log(100 - U\%) \qquad U > 60\% \tag{8b}$$

The graphical solution for u = $f(z/H, T)$ for a layer of thickness 2H that is initially at equilibrium and subjected to a rapidly applied uniform surface loading is shown in Fig. 2a. The average degree of consolidation U over the full depth of the clay layer as a function of T for this case is shown in Fig. 2b.

Consolidation Settlement

For a saturated clay soil, the settlement or compression on the application of load is due to change in the void ratio of the soil



Compression, Fig. 2 Solution to the one-dimensional equation. (**a**) Distribution of excess pore water pressure as a function of dimensionless time and depth for a doubly drained clay layer and (**b**) average degree of consolidation as a function of time factor mass since it is assumed that the soil mineral grains are relatively incompressible. In one-dimensional consolidation, the settlement of the soil will therefore be equal to change in the thickness of the soil layer. There are two ways to compute the settlement, namely using coefficient of volume compressibility and $e - \log\sigma$ methods.

Using the coefficient of volume compressibility, the amount of vertical settlement ΔH that a homogeneous clay layer of thickness H_o will undergo when subjected to a vertical stress increase at the surface is given by (Adeyeri 2015; Coduto et al. 2016)

$$\Delta H = \frac{H_o}{(1+e_o)}\Delta e = H_o\frac{a_v}{(1+e_o)}\Delta\sigma = H_o m_v \Delta\sigma \qquad (9)$$

where

e_o = the initial void ratio
Δe = the decrease in void ratio due to the stress increase from σ_o to σ_1
$m_v = \frac{a_v}{(1+e_o)}$ is referred to as the coefficient of volume compressibility

Similarly by using the compression index, it can be shown (Adeyeri 2015) that the settlement of a clay soil of thickness layer H due to a stress increase is

$$\Delta \mathrm{H} = \frac{H_o}{(1+e_o)}\Delta \mathrm{e} = \frac{H_o}{(1+e_o)}C_c\log\frac{\sigma_o+\Delta\sigma}{\sigma_o} \qquad (10)$$

If the clay is normally consolidated, the entire loading path is along the virgin compression line (VCL) and so

$$\delta_c = \sum\frac{C_c}{1+e_0}H\ \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) \qquad (11)$$

However, if the clay is overconsolidated and remains so by the end of consolidation, i.e., $\sigma_1 < \sigma_p$, then

$$\delta_c = \sum\frac{C_r}{1+e_0}H\ \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) \qquad (12)$$

Note that in this case, it is the recompression index that is being used.

But if an overconsolidated clay becomes normally consolidated by the end of consolidation ($\sigma_1 > \sigma_p$), then for this case, the settlement consists of two parts: one due to recompression and the other due to normal consolidation.

$$\delta_c = \sum\frac{C_r}{1+e_0}H\ \log\left(\frac{\sigma'_c}{\sigma'_{z0}}\right) + \frac{C_c}{1+e_0}H\ \log\left(\frac{\sigma'_{zf}}{\sigma'_c}\right) \qquad (13)$$

Secondary Compression

After a sufficiently long time has elapsed, excess pore water pressure in a soil layer, which is undergoing consolidation, approaches zero signifying the end of theoretical consolidation. However, the clay will still continue to decrease in volume slowly if the load is not reduced or removed. This phenomenon, which takes place after the dissipation of excess pore pressure, is referred to as **secondary compression**. Secondary compression is caused by creep, a slow viscous change in the soil–water system, which leads to a gradual change in the void ratio without moisture movement, compression of organic matter, and other processes. As a result of secondary compression, some of the highly viscous water between the points of contact of the soil particles may be forced out.

It is assumed that secondary compression progresses linearly with the logarithm of time (Fig. 3) and in series with primary consolidation (Terzaghi et al. 1996). The secondary compression index is similar to the compression index. It is the slope of the secondary compression part and is defined as follows:

$$C_\alpha = \frac{\Delta e}{\Delta \log t} = \frac{e_1 - e_2}{\log t_2/t_1} \qquad (14)$$

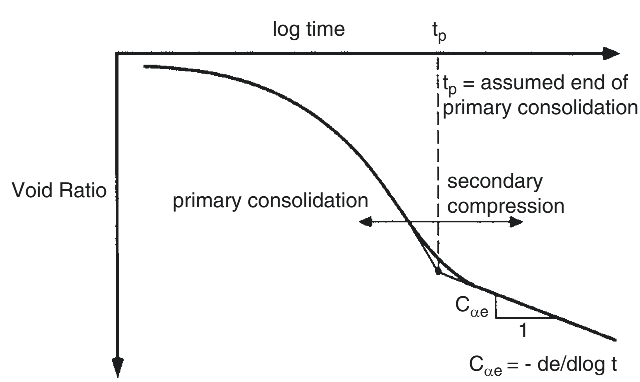
where e_1 and e_2 are the strains at t_1 and t_2, respectively, in the secondary zone of the $e - \log$ curve (Fig. 3).

Secondary compression is then given by the formula (Schiffman et al. 1964)

$$S_s = \frac{H_0}{1+e_0}C_\alpha\log\left(\frac{t}{t_{90}}\right) \qquad (15)$$

where

H_0 = the height of the consolidating medium
e_0 = the initial void ratio
C_a = the secondary compression index



Compression, Fig. 3 e versus log t

Compression, Table 2 Common values of C_α, after Cernica (1984)

Overconsolidated clays	0.0005–0.0015
Normally consolidated clays	0.005–0.03
Organic soils, peats	0.04–0.1

$t =$ the length of time after consolidation considered
$t_{90} =$ the length of time for achieving 90% consolidation

Some common ranges of C_a are given in Table 2.

It should be mentioned that secondary consolidation may not always follow primary consolidation. The two processes may under certain situations occur simultaneously. This is particularly so during the long years of settlement of thick layers of clay. In sand, settlement caused by secondary compression is negligible, but in organic soils, it is very significant. For normally consolidated soils, the ratio of the coefficient of secondary compression to the compression index ($C_\alpha/C_c = C_{\alpha\varepsilon}/C_{c\varepsilon}$) is relatively constant for a given soil. On average, the value of C_α/C_c is 0.04 ± 0.01 for inorganic clay and silt. For organic clay and silt, the value averages 0.05 ± 0.01. For peat, the value averages 0.06 ± 0.01.

The generalization of Terzaghi's one-dimensional consolidation theory to three dimensions has been made by many investigators (Biot 1941; Schiffman et al. 1964). At present, there are finite element and finite difference methods for the 3-D consolidation equation incorporating nonlinear stress–stress relationships as well as anisotropic hydraulic conductivity. The hydraulic conductivity can also be treated as a function of void ratio or effective stress (Coussy 2004).

The analysis of compression or volume change is typically done through consideration of a soil mass as a continuum, but the processes that determine it are at the particulate level and involve discreet particle movements required to produce a new equilibrium following changes in stress and environmental conditions. Important aspects of colloidal type interactions involving interparticle forces, water adsorption phenomena, and soil fabric effects have been analyzed and extensively discussed by many investigators, especially Mitchell and Soga (2005) who have also highlighted the reasons why the commonly used constitutive models for soil compression and consolidation may not give suitable representations of actual soil behavior.

Summary

Compression in soils is caused primarily by the rearrangement of soil grains and expulsion of water from the voids accompanied by fracturing of soil grains in coarse-grained soils and compression or swelling of clay particles in fine-grained soils. It may also be as a result of changes in confinement, loading, exposure to water and chemicals, changes in temperature, etc. Soil compression and consolidation under applied stress have been the most studied owing to their essential role in estimation of settlements, and this was one of the first motivations for development of soil mechanics.

The behavior of the soil during isotropic compression and swelling is governed by $e = e_o - C_c \log\sigma$. This equation is for the virgin portion of the compression curve. The state of a soil cannot ordinarily go above the compression curve defined by the above equation. However, it can go below the line during unloading when the soil becomes overconsolidated. The overconsolidation ratio is given as $\mathrm{OCR} = \sigma_p'/\sigma_{vo}'$ where σ_p' is the overconsolidated pressure or yield stress. The total compression is made of three components namely elastic, primary consolidation, and secondary compression components. The commonly used methods of estimating the three components are highlighted.

Cross-References

- ▶ Artificial Ground
- ▶ Atterberg Limits
- ▶ Casagrande Test
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Cohesive Soils
- ▶ Collapsible Soils
- ▶ Compaction
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- ▶ Geotextiles
- ▶ Ground Pressure
- ▶ Hydrocompaction
- ▶ Land Use
- ▶ Landfill
- ▶ Liquid Limit
- ▶ Noncohesive Soils
- ▶ Normal Stress
- ▶ Percolation
- ▶ Plastic Limit
- ▶ Plasticity Index
- ▶ Pore Pressure

▶ Poisson's Ratio
▶ Residual Soils
▶ Saturation
▶ Shear Strength
▶ Shear Stress
▶ Soil Field Tests
▶ Soil Properties
▶ Strain
▶ Strength
▶ Subsidence
▶ Voids

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C

Concrete

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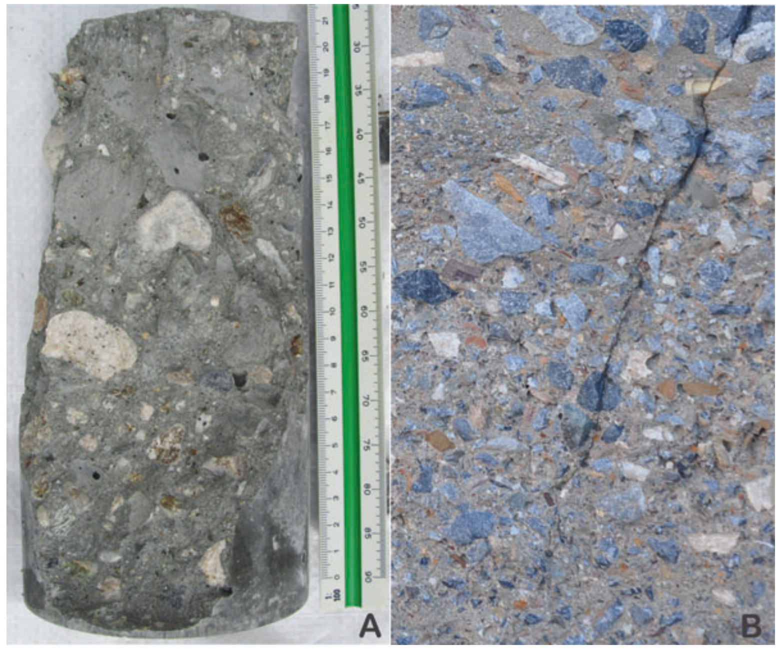
Definition

A general name used to refer to manufactured or synthetic rock material that is formed by cohesion and then solidifies. Concrete has similarities to a natural deposit of well-cemented, clastic, sedimentary rock called conglomerate. Typical concrete constituents are cement, water, mineral aggregates, and chemical admixtures. Bituminous material is the cement in asphalt concrete, typically called "asphalt" or "black top"; however, the most common cement used in what is called "concrete" is Portland cement, a compound made from clay and limestone. Clay is a source of silica, alumina, and iron, which upon wetting will react with calcium oxide derived from high-temperature roasting of crushed and powdered nearly pure calcite limestone ($CaCO_3$). Wetting transforms powdered Portland cement by hydration into a durable strong solid composed of four silica and alumina compounds: tricalcium silicate ($3(CaO){\cdot}SiO_2$), dicalcium silicate ($2(CaO){\cdot}SiO_2$), tricalcium aluminate ($3(CaO){\cdot}Al_2O_3$), and tetracalcium aluminoferrite ($4(CaO){\cdot}Al_2O_3Fe_2O_3$). A small amount of gypsum ($CaS0_4{\cdot}2(H_20)$) is used to control the rate at which cement hardens. *Hydration* is an *exothermic chemical reaction* that generates substantial heat depending on the thickness of the curing mass of concrete.

The Portland cement-water mixture before it hardens is called *paste*; it coats the *aggregate* particles and promotes "workability" of concrete, allowing it to be spread and placed into forms. Concrete mix design utilizes the weight-ratio of water to cement as an index of ultimate *compressive strength* of cured concrete and workability of fresh concrete (USACE 1994). Lower water:cement ratios (<0.40) have higher strengths but less favorable workability, whereas higher water:cement ratios (>0.55) have more favorable workability but lower strengths. Water containing *dissolved elements*, such as sodium, could be deleterious to concrete performance by leaching calcium hydroxide from hardened cement-paste matrix, resulting in strength loss. Water containing calcium may have minor effects on concrete performance, possibly related to air entrainment.

Concrete, Fig. 1 A - Natural Exposure of conglomerate. B - Broken concrete exposing its constituents



Mineral aggregates used in concrete must be durable and strong *subangular* to *angular* particles in the sand and gravel size ranges, called fine and coarse, respectively. Aggregates comprise 60–75% of concrete volume or 70–85% of concrete mass. Durability of coarse aggregate is determined by standardized tests, such as *Los Angeles abrasion*, chemical (sodium and magnesium) soundness, and freezing and thawing. Percentages of fine and coarse aggregates are specified for different concrete applications. Concrete without aggregate is called *neat cement grout*; concrete without coarse aggregate is called *sand-cement grout*.

Chemical admixtures typically are used to modify the properties of cured concrete; ensure quality during mixing, transporting, placing, and curing concrete; and reduce the cost of concrete construction. Admixtures can retard or accelerate the rate of curing, reduce the required amount of water, enhance air entrainment, counteract corrosive effects of on-site soil or groundwater, and reduce shrinkage during curing. Cured concrete has favorable compressive strength but low tensile strength. Many structural applications use *reinforced concrete*, which is placed to engulf steel bars or welded wire mesh. Steel fibers can be mixed into concrete for *shotcrete* applications (Fig. 1).

Cross-References

▶ Aggregate
▶ Aggregate Tests
▶ Alkali-Silica Reaction
▶ Cement
▶ Clay
▶ Gradation/Grading
▶ Grouting
▶ Petrographic Analysis
▶ Shear Strength
▶ Shotcrete
▶ Strength

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Conductivity

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Definition

A measure of a material's ability to transmit a flow across it under given conditions; the flow can be of heat, electricity, magnetism, light, sound and, of especial interest to engineering geology, fluids (water, gas, and oils).

To obtain a measure of conductivity, a defined volume of the material is used, e.g., a disc or column; a volume of shape that has radial symmetry. Imagine a column for the purposes of this measurement; it has three boundaries: one at the top through which the flow of whatever is being measured enters the material, one at the bottom through which whatever is flowing leaves the material, and the sides of the column are its third boundary and for the purposes of this measurement they are controlled so that no flow can cross them; they can be sheathed in some way. The distance between the upper and lower boundaries is (L) in units of length, and the area of each of the upper and lower boundaries is (A) in units of area (British Standard Normative tests 22282 Geohydraulic testing, Parts 1 to 6 (2012)).

Once the boundaries are established, the conditions on them have to be defined, and for ground water that means the water level operating at the top and the bottom of the column. These water levels will both be measured from a common datum and that allows them to be called "heads" as they are then a measure for the work that has been done in raising them above that datum. Flow will only occur if the level at the top differs from that at the bottom; that is, if there is a difference in head across the column, let that difference be called (Δh) in units of length. The potential for the difference in work to generate flow between the top and the bottom of the sample is given by the fraction ($\Delta h/L$) which, being length/length, is dimensionless.

The flow that actually results can be measured as the volume per unit time crossing the end boundaries (Q) per unit area (A). So, in theory (Q/A) should be proportional to ($\Delta h/L$); for example, if (A) and (L) are not changed but (Δh) is increased, (Q) could be expected to increase proportionally. To say by how much this increase will be, it is necessary to introduce a coefficient (K), whose magnitude is based on the slope of the plot of (Q/A) measured against ($\Delta h/L$) (Darcy 1856). That enables the Darcy eq. [$Q = K(\Delta h/L)A$] to be written, but this presumes the intercept of that graph passes through its origin.

The unit of (K) in groundwater is velocity (length/time) where (K) is called ***hydraulic conductivity*** or permeability. Conductivity is very sensitive to fabric and sample disturbance and so best measured using a field arrangement that provides reasonable constraint on boundary conditions. Hydraulic conductivity has the greatest range of value of all geotechnical parameters, and its measured value can easily be in error by orders of magnitude. It is also directional; the value measured is in the direction of flow.

Indicative values of K are given in Table 1.

Conductivity, Table 1 Indicative ranges for the conductivity (K) of rocks and soils to illustrate the great range of values possible and the care needed with the use of this parameter. Velocity of flow = conductivity × hydraulic gradient

Material	cm/s	m/s	Notes
Rocks in-situ[a]			**As in the field**
Limestones	10^{1} to 10^{-4}	10^{-2} to 10^{-6}	Even greater in karst
Fractured rock	10^{-2} to 10^{-6}	10^{-4} to 10^{-8}	Igneous crystalline & metamorphic
Porous rock	10^{-4} to 10^{-8}	10^{-6} to 10^{-10}	Sandstones & similar including volcanic ash
Mudrocks	10^{-7} to 10^{-11}	10^{-9} to 10^{-13}	Shales, mudstones, etc.
Rock material			**As in a hand specimen**
Solid igneous crystalline metamorphic rock	10^{-8} to 10^{-11}	10^{-10} to 10^{-13}	And capable of being lower
Porous rock	10^{-3} to 10^{-7}	10^{-5} to 10^{-9}	Will depend on direction of bedding relative to flow
Mudrocks	10^{-6} to 10^{-13}	10^{-8} to 10^{-15}	
Sediments			**As in engineering soils**
Gravel	10^{2} to 10^{-1}	10^{0} to 10^{-3}	Clean or little silt and clay
Sand	10^{0} to 10^{-3}	10^{-2} to 10^{-5}	
Silt	10^{-3} to 10^{-6}	10^{-5} to 10^{-8}	Will depend on direction of bedding relative to flow
Soft to firm clay	10^{-7} to 10^{-10}	10^{-9} to 10^{-12}	Can be indented with finger pressure
Stiff to hard clay[b]	10^{-10} to 10^{-13}	10^{-12} to 10^{-15}	Either not easy to indent or not indented with finger pressure

[a]The conductivity of rock in-situ generally decreases with depth although karstic limestones and volcanic rocks can provide major departures from such a trend

[b]These can be fissured and possess a much higher conductivity whilst the fissures are open

Cross-References

▶ Darcy's Law
▶ Field Testing
▶ Piezometer

References

British Standard Normative tests 22282 Geohydraulic testing (2012) BS EN ISO 22282-Part 1 General rules. BS EN ISO 22282-Part 2 Water permeability tests in a borehole using open systems. BS EN ISO 22282-Part 3 Water pressure tests in rock. BS EN ISO 22282-Part 4 Pumping tests. BS EN ISO 22282-Part 5 Infiltrometer tests. BS EN ISO 22282-Part 6 Water permeability tests in boreholes using closed systems

Darcy H (1856) Les fontaines publiques de la ville de Dijon. Victor Dalmont, Paris, pp 305–311

C

Cone Penetrometer

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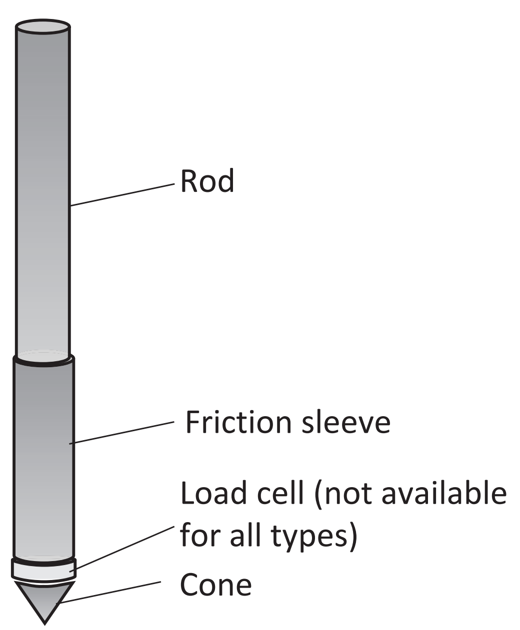
Definition

A **cone penetrometer** is an instrument used to perform a cone penetrometer test (CTP), from which preliminarily geotechnical engineering properties of soils, such as the soil strength, can be evaluated, and the delineation of soil horizons can be interpreted (ASTM D3441 2005; ASTM D5778 2000). There are different types of cone penetrometers, including mechanical, mechanical friction, electrical friction, and piezocone penetrometers. Mechanical cone penetrometers are known for their low cost and easy operation, where as the electrical cone and piezocone penetrometers can potentially extend the range of soil engineering property measurements (NRCS 2012).

Regardless of the cone penetrometer type, the basic components of each typically includes a steel conical tip with a 60° (±5°) apex, a load cell, a steel friction sleeve, and cylindrical rods (ASTM D3441 2005; ASTM D5778 2000) as shown in Fig. 1. The specifications of these components, however, vary among types.



Cone Penetrometer, Fig. 1 A schematic of a simplified cone penetrometer instrument

The original mechanical cone and mechanical friction jacket cone penetrometers are also known as the *Dutch Mantle Cone* and *Begemann Friction-Cone,* respectively, because the cone penetrometer was originally developed in the 1950s at the Dutch Laboratory for Soil Mechanics in Delft to investigate soft soils (Begemann 1953, 1965). The original application of CPT was to evaluate the soil total bearing capacity. A friction sleeve was then added to separate and quantify the two components of the total bearing capacity, that is, the tip friction and friction generated by the rod (Begemann 1965). The introduction of electronic measurements made cone penetrometers more versatile. Additional features, such as *pressure transducer*, *magnetometer*, and *geophone*, were then added to collect pore water pressure data, detect ferrous materials, and estimate the shear modulus and Poisson's ratio through measurements of seismic shear wave and compression wave velocities.

The results from CPT are based on interpretation of empirical correlations (NRCS 2012), rather than direct measurements. The accuracy of the testing results is sensitive to many factors, such as the stiffness and thickness of the soil horizons, the saturation of the soil, the experience and skill of the operators, as well the suitability of the equipment and facilities used. The CPT performs better in saturated homogeneous soil with greater thickness, but is less reliable in unsaturated clayey soils, and has limited penetration in heterogeneous stiffer materials, such as gravel and cemented materials (ASTM D3441 2005; NRCS 2012).

Cross-References

▶ Bearing Capacity
▶ Characterization of Soils
▶ Classification of Soils
▶ Poisson's Ratio
▶ Pore Pressure
▶ Saturation
▶ Shear Modulus
▶ Shear Strength
▶ Soil Mechanics
▶ Soil Properties
▶ Strength

References

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Consolidation

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Definition

1. In Soil Mechanics (Engineering): Time-dependent volumetric change of a soil in response to increased loading, involving squeezing of water from the pores, decreasing volume, and increasing effective stresses
2. In Geology (Scientific): Process or processes whereby loose, soft, or molten Earth materials become firm and coherent (Holtz et al. 2011; Herrmann and Bucksch 2014).

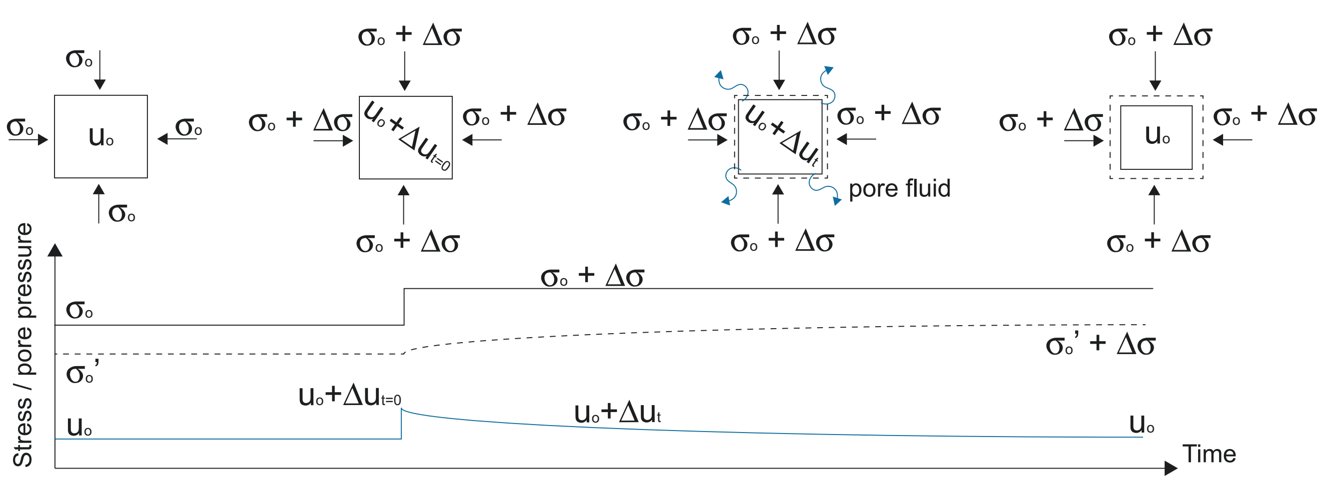
The engineering definition of consolidation is followed here.

Consolidation Process

During consolidation of a fully saturated soil, an isotropic stress state starts when an increase in total pressure ($\boldsymbol{\Delta}\sigma_0$) is applied to a soil volume that was initially at equilibrium under the *in situ* stress state (σ_0) and pore water pressure (u_0). The increase in total stress is assumed to be initially transferred as an increase in pore pressure ($\Delta u_{t=0}$) (Fig. 1). This increase in pore pressure dissipates over time at a rate that is inversely proportional to the soil's hydraulic conductivity. Dissipation of this excess pore pressure is associated with a loss in pore water content, leading to a volume loss and an increase in the dry density of the soil. As the excess pore pressure dissipates, the initial effective stress of the soil (σ_0') increases until it accounts for the increased total stress ($\sigma_0' + \Delta\sigma_0$).

The relationship between soil volume and stress takes the form of a loading curve and a family of unloading (re-loading) curves that depend on the soil stress history. The consolidation rate is solved through a diffusion equation (for excess pore pressure) that depends on the soil void volume and hydraulic conductivity. This equation was proposed and solved initially by Terzaghi (Holtz et al. 2011) for the one-dimensional case (Eq. 1). The concepts of consolidation have been expanded for unsaturated soil conditions (Eqs. 2 and 3) and for the three-dimensional general case (Fredlund et al. 2012).

One-dimensional consolidation under saturated conditions (Terzaghi and Peck 1960):



Consolidation, Fig. 1 Simplified sketch of the soil consolidation process

$$\frac{d_{uw}}{d_t} = C_v \frac{d^2{}_{uw}}{d_z{}^2} \quad (1)$$

$\frac{d_{uw}}{d_t}$ is the change in pore water pressure with time, $\frac{d^2{}_{uw}}{d_z{}^2}$ is the second derivate of pore water pressure with position (depth), and C_v is the coefficient of consolidation.

One-dimensional consolidation under unsaturated conditions (Fredlund et al. 2012):

$$\frac{d_{uw}}{d_t} = -C_w \frac{d_{ua}}{d_t} + C_v^w \frac{d^2 uw}{d_z{}^2} \quad \text{(water phase)} \quad (2)$$

$$\frac{d_{ua}}{d_t} = -C_a \frac{d_{uw}}{d_t} + C_v^a \frac{d^2 ua}{d_z{}^2} \quad \text{(air phase)} \quad (3)$$

C_w and C_a are constants associated with the water (w) phase and the air (a) phase in the unsaturated soil, $\frac{d_{ua}}{d_t}$ and $\frac{d_{uw}}{d_t}$ are the change in pore air pressure and pore water pressure with time, and C_v^w and C_v^a are the coefficients of consolidation with respect to the water phase and air phase. $\frac{d^2{}_{ua}}{d_z{}^2}$ is the second derivate of pore air pressure with position (depth). Equation 2 is a simplified form for the partial differential equation for the water phase during unsaturated consolidation. The simplified form neglects the gravitational component of the hydraulic head and considers that the coefficient of permeability does not vary significantly with space. Equation 3 is a simplified form for the partial differential equation for the air phase during unsaturated consolidation. This simplified form neglects the variation of air transmissivity with space.

Rigorous formulation of three-dimensional, unsaturated consolidation requires simultaneously solving the equilibrium equations and the continuity equations for water and air flow. Details are presented in Biot (1941) and Fredlund (2012).

Cross-References

- ▶ Effective Stress
- ▶ Pore Pressure
- ▶ Saturation
- ▶ Soil Mechanics
- ▶ Stress
- ▶ Voids

References

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Contamination

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Definition

Contamination is the act or process of contaminating or the state of being contaminated.

Interpretation of the term “contamination” depends on its scope. In chemistry, the term “contamination” refers to the mixing of components, contamination of samples, solutions, etc., distorting the results of an analysis. In geology, contamination is the process of changes to the compositions of igneous rocks under the influence of assimilation (capture and processing) of sedimentary and metamorphic rocks that differ from the parental magma composition. Contamination is possible if the temperature of the magma is sufficient for remelting captured fragments (xenoliths) of host rocks.

Radioactive contamination is the deposition of radioactive substances on surfaces, or within solids, liquids, or gases (including in the human body), where their presence is unintended or undesirable, or the processes giving rise to their presence in such places.

In environmental geochemistry, contamination is the presence of a substance where it should not exist or at concentrations above local background levels.

There is a difference between the terms contamination and pollution. Pollution is contamination that results in or can result in adverse biological effects to resident communities. All pollutants are contaminants, but not all contaminants are pollutants. Contamination could be any quantity of a contaminant but pollution means that a quantity of pollutant has reached a level that is hazardous to health or ecosystems.

Cross-References

- ▶ Brownfield Sites

References

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International Atomic Energy Agency (2010) Programmes and systems for source and environmental radiation monitoring. Safety reports series no. 64. IAEA, Vienna, p 234

Cross Sections

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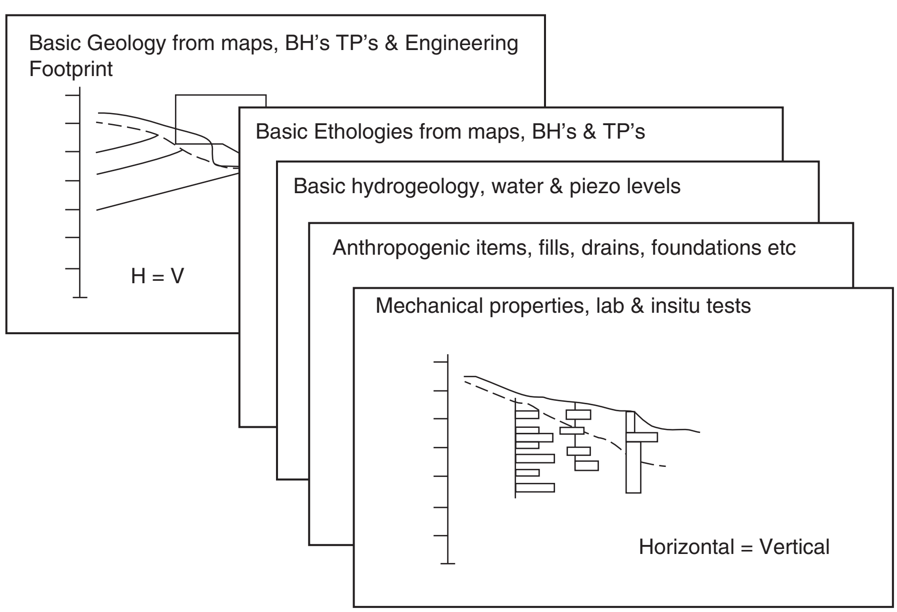
Definition

Illustrations of the geology that lie vertically below the line defining the position of that section on a map.

The line of section can be one straight line or a number of straight lines, each joined to its neighbor but following a different direction from its neighbor, or occasionally, the line can be curved. The vertical scale used in most geological cross sections differs from the horizontal scale so that detail over a vertical distance of tens of meters can be shown over horizontal distances of thousands of meters. This means that distance on these sections can only be measured accurately in the vertical and horizontal direction. Any angle measured between points on such a section is a distortion of the real angle between those points. For engineering purposes, it is therefore advisable to use vertical and horizontal scales that are identical.

The accuracy of sections is usually constrained by boreholes where the vertical profile is known correctly; extrapolations between boreholes and from exposures and surface outcrops enable the sections lengths between the boreholes to be filled. The limits of the structures to be constructed should also be shown at ground level or below ground level as appropriate.

Sections need not only display solid and drift geology; any character of the ground can also be shown, including groundwater and material properties such as strength and permeability at the depths where these have also been measured. Indeed an aspect of sections most valuable to ground engineering is their ability to reveal relationships of properties that were measured independently. Thus a geological section can be used to show the geology at a borehole on the line of section plus the fluid returns with depth from drilling them, and any measurements of strength and permeability *in situ* with depth, all displayed as overlays to the basic solid geology (Fig. 1). To these can be added the location of piezometers and their piezometric level and water tables, and any geophysical measurements made in the holes. Laboratory test results can similarly be displayed on sections, at the depth from which the samples tested were recovered. In these ways a holistic picture of ground at depth and its relationship to the engineering works can be constructed (de Freitas 2009). Numerical models rely heavily on such sections to constrain their design and functionality.



Cross Sections, Fig. 1 Showing how sections for different subjects can be superimposed (From de Freitas 2009). BH = borehole; TP = trial pit

Thought should be given to the scale of a section when used for design purposes as small scale detail which may be significant can be lost at small scales. Similarly thought should be given to the direction of a section; slope instability normally requires a direction parallel to the direction of slope and a section either illustrating or analyzing groundwater flow should be in the plane containing the greatest hydraulic gradient.

Sections constructed in different directions across a given volume of ground form the basis of simply built 3D models for the ground; these are variously called fence diagrams or egg-box diagrams and were much used in mining engineering (Excellent tutorials on U-tube (n.d.)).

Cross-References

- ► Engineering Geological Maps
- ► Exposure Logging
- ► Modelling

References

de Freitas MH (2009) Geology; its principles, practice and potential for Geotechnics. (Ninth Glossop lecture). Q J Eng Geol Hydrogeol 42:397–441. (see pages 409-413)

Excellent tutorials on U-tube; Insert *drawing geological cross sections*. e.g. Oregon State University (OSU ECampus). https://www.youtube.com/watch?v=Vgo8Z63n60g

Crushed Rock

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Definition

Hard natural rock reduced to fragments by mechanical crushing or breaking. (Preparation of ore minerals also requires crushing but that is to secure crushed mineral rather than crushed rock and is not within this definition.)

Synonyms

Crushed stone

Introduction

Hard rock is mechanically crushed or broken mainly for construction aggregates and fill. Sources are primarily limestone, dolomite, sandstone, and igneous rock but sometimes quartzite and metamorphic rocks such as marble and granulite. Sources are widespread where bedrock is at or near the surface and is not significantly weathered or hydrothermally altered. Mineralogical composition, grain size, grain sorting, cementation, compaction, porosity, and weathering state directly affect potential uses. Selection depends on a variety of physical and chemical tests.

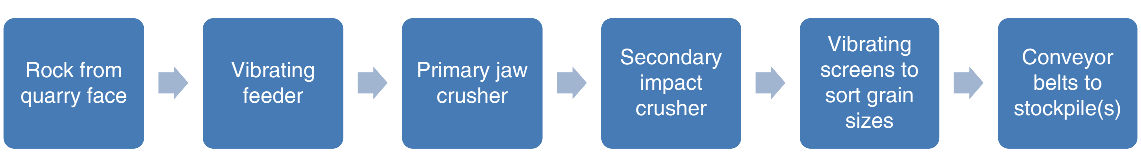
Crushing and Grading

Large blocks brought directly from the quarry blast pile are reduced to a few tens of centimeters or less, in primary crushers then further reduced in secondary crushers. The debris are graded (sieved) to a grain size distribution that meets standards and specifications for the intended use (Fig. 1). For some uses, rock is powdered.

Construction Uses

Crushed rock aggregate and fill are used in construction with, or without, binders. Admixture with cement and other additives makes concrete. Coating with bitumen (asphalt) or cement provides road surfacing material. Uses without binders include railway track ballast, road base courses, surface dressings and material used to reduce lake, stream, and coastal erosion.

Important factors for use in concrete are particle-size distribution, angularity, resistance to impact, volume stability/frost susceptibility, density, and water absorption. Properties of the aggregate affect concrete characteristics (density, strength, durability, thermal conductivity, and shrinkage). The shape, surface texture, and grading of aggregate



Crushed Rock, Fig. 1 The sequence of processing rock into crushed rock particles influence workability and strength of concrete. They should have low porosity (less than 1%) to reduce the amount of water used. The crushed rock should be clean (with limits on clay or other weak rocks, silt, and dust) and not contain deleterious impurities (e.g., mudstone, pyrite, coal, mica) that would reduce strength and durability. It should be resistant to attack by alkali-silica reaction (Kazi and Al-Mansour 1980).

Aggregates for load-bearing road pavements should be strong, durable, resistant to crushing, impacts, abrasion, polishing (skid resistant), stripping (tearing away from the binder), and chemical and weathering damage. These restrictions are obtained from high-quality sandstone and igneous rock. Base courses can be constructed using slightly lower quality rock. Railway track ballast must be strong, clean, and angular with a high resistance to abrasion and attrition and is mainly sourced from hard igneous rocks.

Lower quality permeable material is used as free draining rock fill and for pipe bedding and drains. Impermeable material is used to raise or level construction sites, or for hard standings and tracks (Smith and Collis 2001).

Other Uses

Some crushed rocks, especially sandstones, can be used as filtering media. Other uses depend on chemical suitability of limestone or dolomite. These include powders and fillers in plastics and paper, high purity limestone or dolomite for industrial and chemical processes, furnace fluxes, flue-gas desulphurization, fertilizers, and reduction of soil acidity (Rooney and Carr 1971).

Summary

Hard rock is mechanically crushed or broken mainly for construction aggregates and fill but limestone and dolomite have a range of other uses. Selection of suitable stone depends on mineralogical composition, grain size, grain sorting, cementation, compaction, porosity, weathering state, and the intended use.

Cross-References

- ► Aggregate
- ► Aggregate Tests
- ► Alkali-Silica Reaction
- ► Concrete
- ► Cut and Fill
- ► Gradation/Grading
- ► Igneous Rocks
- ► Limestone
- ► Metamorphic Rocks
- ► Sedimentary Rocks

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Current Action

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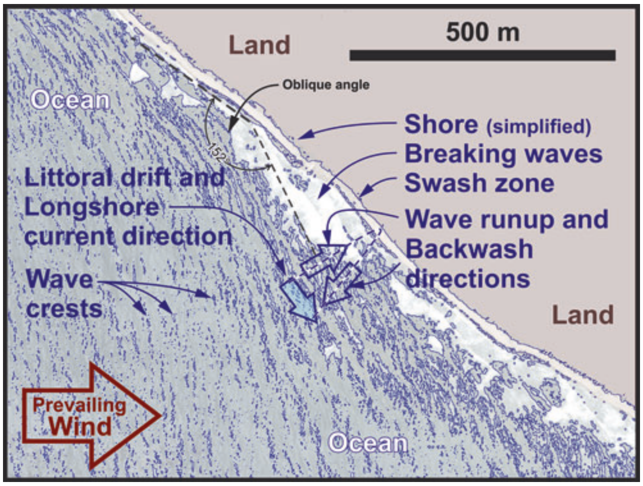
Definition

Current action refers to the effects of water moving in oceans or lakes.

Unlike water flowing under the influence of gravity down a river channel, movement of water in oceans and large lakes is caused by tidal cycles, wind, and temperature and density differences in the water (NOAA 2018). Gravitational attraction of the Earth to the Sun and the Moon, and its rotation about its axis, causes predictable cyclic rise and fall of large bodies of water. The rise and fall of tides results in movement of water that is most noticeable where it encroaches on coastlines, estuaries, bays, and harbors as the tides “come in,” and recede from them as the tides “go out.” Tidal currents are called “flood” or “ebb” in the rising and falling of tides, respectively. Because the tidal range is relatively uniform at any particular place along the coast, the reversing currents create equilibrium landforms, until some major natural or human-caused event disturbs the shape of the tidal channels or shoreline.

Wind blowing across open water in oceans and large lakes imparts shear stress on the surface of the water, which causes the near-surface water to move in the direction the wind is blowing. The resistance of the water to move creates irregularities on a still water surface that become ripples and then waves. The size and wave length of the waves depends on the water depth, wind speed, and wind duration, as well as the



Current Action, Fig. 1 Computer graphics sketch of wave crests in a coastal ocean environment illustrating wave fronts arriving at oblique angles to the shoreline. Schematic vectors of wave run-up and backwash result in a net movement of water and sand in what are called the longshore current and littoral drift

distance of open water across which the wind is blowing, which is known as fetch (USACE 2008). Waves begin to slow as water depth becomes shallower than a critical depth. Ultimately, waves break on the shore dissipating substantial energy as that happens. Waves running up a wide, gently sloping beach tend to dissipate energy gradually, whereas a wave with a similar character breaking against a rocky cliff or an engineered breakwater dissipates energy abruptly. It is common for waves to move toward the shoreline at an oblique angle, which causes waves to run up a beach, crossing the swash zone at an angle and flowing back perpendicular to the beach or at a similar oblique angle (Fig. 1). The effect of waves approaching shore at an oblique angle is a net movement of water along the beach which creates and sustains a longshore current. The waves that create and maintain the longshore current also move sand in the same general direction in a process known as littoral drift.

Differences in water temperature and in water density can create slow-moving currents in otherwise still water. Temperature differences occur where general circulation of tropical water encounters general circulation of arctic water. The general circulation results from the continuous tidal action, enhanced or disrupted from time to time by major storms and even an occasional tsunami. Density differences occur in response to temperature, but they also can be related to differences in concentration of sodium chloride where freshwater dilutes seawater. Such currents, called thermohaline (for the NaCl mineral halite), occur in both shallow and deep levels in the ocean and move much slower than either tidal currents or wave-induced surface currents.

Cross-References

- ▶ Beach Replenishment
- ▶ Coastal Environments
- ▶ Hydraulic Action

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Cut and Cover

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Synonyms

Bottom-up tunneling; Cut and cover tunneling; Shallow-depth tunneling

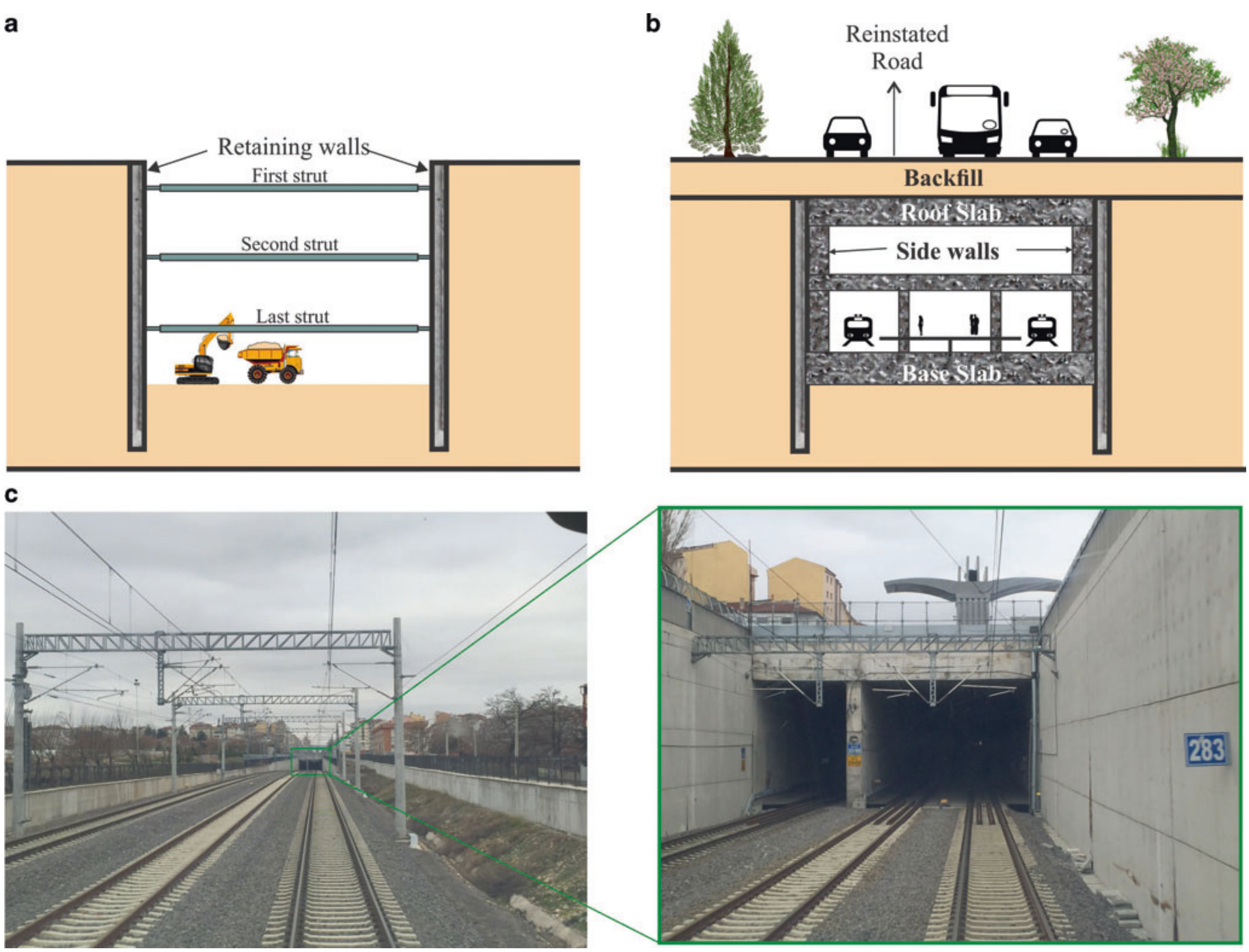
Definition

Cut and cover is a tunnel construction technique, preferred at a shallow depth, in which excavation can be economically performed from the surface and the trench is subsequently covered with backfill after installation of all components for tunnel structures.

Characteristics

The excavatability and strength characteristics of the ground surrounding the tunnel, *in situ* stresses, depth of groundwater, length and diameter of tunnel, main purpose of tunnel, and particularly the depth of the designed tunnel are important controlling factors on selecting the most appropriate tunnel construction method. Generally, tunnels are classified into three different groups based on construction technique. These are (1) cut and cover tunnel known as an advanced engineering technique mainly for tunnel construction in urban and interurban areas (Mouratidis 2008); (2) bored tunnel, which is usually excavated in a circular or horseshoe cross section by drilling and blasting, using a sequential excavation method or a tunnel boring machine; and (3) immersed tube tunnel (e.g., Istanbul underwater Bosphorus immersed tube tunnel, Turkey). The existence of soft and weak ground conditions and shallow depth are significant factors in designing cut and cover tunneling. The cut and cover tunnel is usually constructed as a rigid frame structure inside a supported trench because of the lack of adequate space at the project site. These double box-type tunnels have a rectangular cross section and are generally larger than circular tunnels (Debiasi et al. 2013). In addition, Bobet et al. (2008) specified that box-shaped tunnels show more stable conditions against some ground failures such as shear displacements resulting from the intersection of an active fault with the tunnel, liquefaction of fine grained sandy soils, slope instability, tectonic uplift, and subsidence (Debiasi et al. 2013).

It is well known that a cut and cover tunnel is cheaper, more appropriate, and a practical construction method for depths of up to 35–45 ft (10.7–13.7 m) in comparison with underground tunneling (Wilton 1996). Based on previous studies and worldwide completed tunnel projects, it may be concluded that the depth of a cut and cover tunnel that exceeds a depth of 100 ft (about 30.5 m) is very rarely preferred. As stated by Wilton (1996), many urbanization-dependent shallow-depth tunnels (e.g., sewer, vehicular and rapid transit tunnels) have been constructed based on the principles of the cut and cover method. In addition, the cut and cover method is also utilized in approaching sections to mine tunnels, construction of tunnel portals and all other transportation, aqueduct, and utility tunnels projected in



Cut and Cover, Fig. 1 **(a)** Installation of retaining walls, excavation and installation of struts; **(b)** construction of underground structures, placing backfill materials, and restoration activities; and **(c)** an example for cut and cover tunneling (Eskişehir High Speed Train Station, Turkey) flat terrain. Typical activities required for cut and covers tunneling sequentially involve traffic control, relocation utilities, support adjacent structures, controlling groundwater if required, installation of temporary decking, installation and then bracing of the ground wall support system, excavation, construction of permanent structure, backfill, restoration of utility problems and problems related to decking removal, and finally repave street (Wickham and Tiedemann 1976). This tunneling method is currently designed in engineering projects in two different approaches: bottom-up (cut and cover) and top-down (cover and cut) constructions.

For bottom-up construction methods, the retaining wall (such as concrete bored pile wall, concrete diaphragm wall, or a steel sheet pile wall) is installed to provide enough stability of sidewalls for initiating excavation. The ground excavating operation is continued and then required numbers of struts are placed to support the tunnel and adjacent structures until reaching the final level. If there is extremely unfavorable geotechnical characteristics, ground stabilization should be considered during excavation to prevent any slope stability problem (Mouratidis 2008). The installed struts are removed one by one after construction of a base slab, side walls, and finally the roof slab. These construction processes are completed by placing backfill materials and restoration activities (Fig. 1). As observed from above, the construction of the tunnel structure is completed before covering in the bottom-up type of procedure. However, as stated by Mouratidis (2008), the conventional bottom-up method induces significant disruption to traffic especially in overcrowded cities. Therefore, with a top-down construction method, the priority is given to the "covering" phase to avoid traffic problems.

Cross-References

- ▶ Cut and Fill
- ▶ Deformation
- ▶ Drilling
- ▶ Excavation
- ▶ Extensometer
- ▶ Foundations
- ▶ Instrumentation
- ▶ Lateral Pressure
- ▶ Normal Stress
- ▶ Pressure
- ▶ Retaining Structures
- ▶ Strength
- ▶ Stress
- ▶ Tunnels

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Cut and Fill

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Definition

Earthmoving works undertaken to even out topography by flattening hills and slopes and depositing the spoil in depressions or on slopes.

Cut and fill works are often carried out in road, railway, canal, housing constructions and mining, etc. (Fig. 1). Natural sites are usually undulating, not level, and must be modified before any construction can begin. Thus, the cut and fill process is, if necessary, one of the first construction processes to take place on each development site.

Earth material removed from rises and hills is emplaced in valleys or on lower parts of side slopes (Mitamura et al. 2011). The aim is to balance material removed from cuts with the materials that are to avoid the costs of taking excess material elsewhere. Also, large volumes of fill are required in large-scale coastal reclamation projects and may be supplied by removal from neighboring mountains or hills (Fig. 2).

After the earthmoving works have been completed, various problems may occur. These include:

- Slope movements due to weak rock masses and joint systems exposed on the excavation slopes
- Land subsidence in landfill if compaction is insufficient
- Landslides in fill slopes if drainage is insufficient, including movements on the unconformity between fill and natural strata (known in Japan as the Jinji Unconformity – see Fig. 1)

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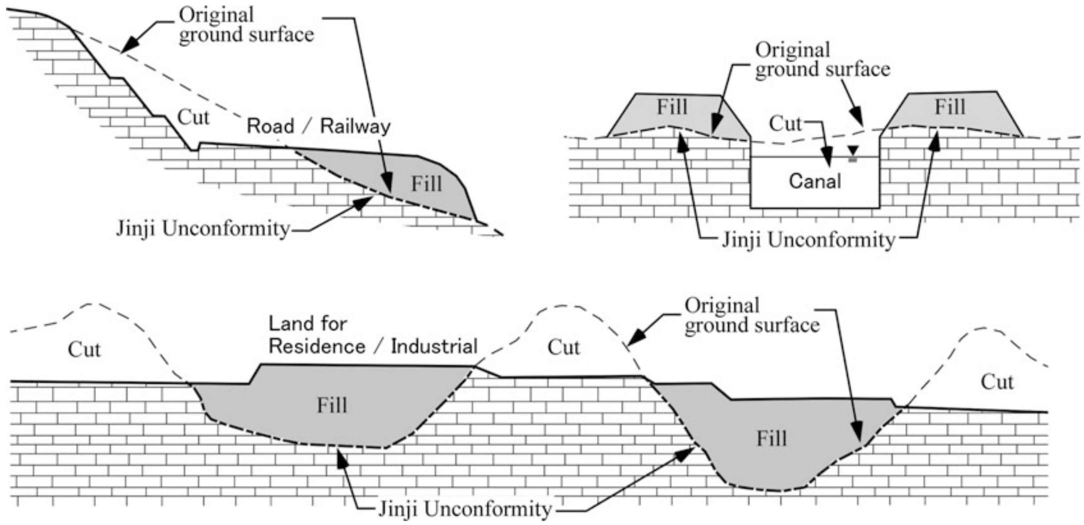
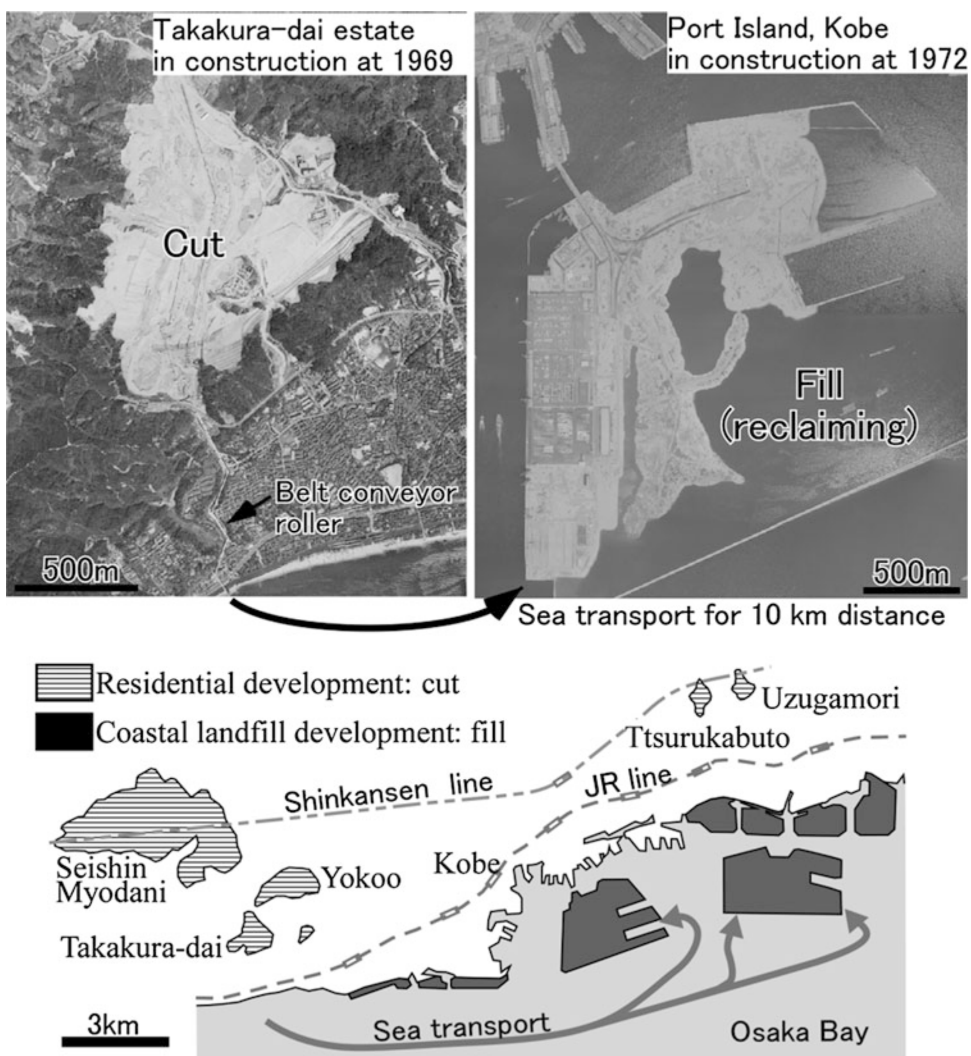


Cut and Fill, Fig. 1 Schematic section on cut and fill

Cut and Fill, Fig. 2 Cut and fill at the regional project on land fill of Kobe Port, Japan (1960s–1980s) (Aerial photos: the Geospatial Information Authority of Japan (GSI))



Depending on the physical, hydrogeological, and chemical properties of the fills, other problems may include:

- Increased susceptibility to liquefaction, fluidization, and ground waves during earthquakes
- Leachates causing contamination of soils or pollution of surface or groundwater if deleterious chemicals are present in the fills (Nirei et al. 2012)

Cross-References

► Artificial Ground
► Contamination
► Cut and Cover
► Excavation
► Fluidization
► Liquefaction

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D

Dams

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Synonyms

Barrier; Catchment; Embankment; Wall

Definition

An engineered barrier to the gravitational flow of water or other fluid that results in a reservoir for use in irrigation, power generation, water supply, or flood control. Dams are constructed using soil, rockfill, concrete, metal, or blocks.

Introduction

Classification

Dams may be classified into a number of different categories. Dams commonly are classified according to their use, their hydraulic design, or the materials of which they are constructed (e.g., USBR 1987).

Dams classified by use include:

- *Storage dams* are intended to impound water for specific uses, such as water supply, recreation, wildlife, or hydroelectric power generation.
- *Diversion dams* are constructed to provide head for water conveyance systems (canals, ditches, tunnels).
- *Detention dams* retard flood runoff to reduce the effect of sudden floods.

Many dams are constructed to serve more than one purpose. For example, a dam may combine storage, flood control, and recreational uses. Some dams have overflow structures, such as Slab Creek Dam, shown in Fig. 1.

The most common classification is based on materials used to build the structure and typically includes design types:

- *Earthfill or earth embankment* – Foundation and topographical requirements for earthfill dams are less stringent that those for other dam types. Earthfill dams built prior to the mid-twentieth century were commonly hydraulic fill or semi-hydraulic fill, both of which are less stable than compacted fill embankments. Use of locally available natural materials requires less processing, and large quantities of excavation and locally available borrow materials are positive economic factors for earthfill dams.

Figure 2 shows an example of an earth embankment dam (Leroy Anderson Dam, Santa Clara County, California).

- *Rockfill* – Rockfill dams use rock clasts to provide stability and a separate impervious membrane to provide water tightness. The membrane may be an upstream facing of impervious soil, a concrete slab, asphaltic concrete paving or other impervious elements, or an interior core of impervious soil. Rockfill dams are suitable for remote locations where the supply of good rock is available or where there is a lack of suitable soil material for earthfill construction. Rockfill dams require foundations that are not susceptible to large settlements (USBR 1987).

Both earthfill and rockfill dams are highly susceptible to damage from the erosive effects of overflowing water, and so they must have means of conveying water around the dam to prevent overtopping (spillway and outlet works).

- *Concrete Gravity* – Concrete gravity dams are suitable for sites where there is typically a competent rock foundation (alluvial foundation is acceptable for low structures with adequate cutoff). They may have overflow spillway crests, and gravity structures commonly are used for spillways for

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Dams, Fig. 1 Overflow spillway

earthfill and rockfill dams, or as overflow sections of diversion dams. Gravity structures may be either straight or curved in plan view, which allows some flexibility in selecting more competent abutment rock foundations, thus requiring less excavation. Roller-compacted concrete (RCC) dams are a specialized type of gravity structure.

- *Concrete Arch* – Concrete arch dams are suitable for sites where the foundation at the abutments is competent rock capable of resisting arch thrust, and the width to height ratio is relatively small. Uplift usually does not impact arch dam stability because of the relative thinness of the structure and the concrete-rock contact. Figure 3 shows a typical concrete arch dam in California (Junction Dam in El Dorado County, California).

Geologic Considerations for Design and Type

Selection of dam type involves evaluations of a number of physical factors, including topography, geology, seismicity, hydrology and stream conditions, geotechnical conditions, and construction material characteristics and availability. Ultimately, the selection of dam type at a particular location is determined by cost and socio-environmental impacts.

Dams, Fig. 2 Earth Embankment Dam

Dams, Fig. 3 Photograph of concrete arch dam

- *Topography* – Topography is a major factor in the selection of dam site and design type. Topographic characteristics include the configuration of the dam site, construction accessibility, and placement of appurtenant structures (e.g., spillways). Concrete dams are common in deep, steep-sided canyons, whereas earthfill embankments are more suited for broad, topographically low hills or plains.
- *Geology* – Geology controls the suitability of foundation and abutment conditions, foundation seepage, reservoir rim stability, landslide and erosion hazards, and potential construction materials (Arnold and Kresse 2010). Geologic conditions include types and thickness of various rock and soil units, stratigraphy, structure (shearing, fracturing, and inclination of geologic units), permeability, and strength (Fraser 2001). Geologic investigations are performed to establish detailed information on rock structure, seismicity and seismic-related effects, and geophysical properties of embankments and foundations.

Competent rock can provide suitable foundations for all types of dams (Volpe et al. 1991). If the rock has been adversely affected by excessive shearing, fracturing, or deep weathering, then deep removal (excavation) combined with consolidation grouting may be needed to provide a suitable foundation. Weak rock will generally not be suitable for tall or heavy dams, but may still be suitable for lower dams.

Gravel foundations are suitable for earthfill or rockfill dams, when compacted to appropriate density and strength (USBR 1987). Methods to provide adequate seepage control, including cutoffs or seals, are required for gravel and coarse-grained materials. Silt or fine sand can provide suitable foundations for low concrete dams and earthfill embankments. Design considerations include non-uniform settlement; piping, seepage, and uplift forces; erosion; and potential for liquefaction. Clay can provide suitable foundations for low earthfill dams with relatively low gradient embankment slopes due to lower foundation shear strengths.

In recent years, there has been a growing awareness of the potential and significance of liquefaction of alluvial foundation materials, even when those materials may have been removed from beneath the core of earthfill embankments. Leaving alluvial materials beneath the embankment shells was considered an appropriate design in past decades. However, seismic stability evaluation of many embankments indicates that alluvial materials will experience significant deformation, causing settlement and disturbance to the embankment crests, when subjected to severe earthquake shaking (Board on Earth Sciences and Resources 2016).

- *Construction Materials* – The availability of large quantities of construction materials is critical to a cost-effective project. Construction materials include sand and gravel for concrete, competent rock for rockfill, and both fine-grained to coarse-grained materials for earthfill embankments. Lower hauling and transportation expenses, due to close proximity to the construction site, can substantially reduce the total construction cost and commonly is the significant factor in selection of dam type for a particular location.

- *Seismicity* – Seismic conditions need to be characterized and incorporated into design of dam structures. Consequently, seismotectonic evaluations are performed to estimate the earthquake loading to which the structures may be subjected. Understanding the seismicity of a site requires evaluating the seismotectonic environment, including geologic, geomorphic and geo-structural analyses, review of earthquake history, and remote-sensing interpretation. Traditionally, either of two general approaches may be used to estimate ground motions at a site: (1) a deterministic approach that uses seismic source (fault) characteristics and historic seismicity combined with potential epicentral distances for each seismic source to determine the potential earthquake loading or (2) a probabilistic method that uses recurrence rates based on historical seismicity to predict epicentral distances for the maximum earthquakes in each source area and predicts events of lesser magnitude and distance for a given probability of occurrence. Probabilistic methods may be used alone or together with deterministic methods (Fraser 1996). The probabilistic events are then used to estimate potential earthquake loadings. Other seismic-related considerations include the potential for fault offsets in the dam foundation and abutments, relative movement (relocation) of the reservoir basin, and earthquake seiche in the reservoir.
- *Hydrology* – Hydrologic conditions typically influence the type and purpose of dam. Precipitation, watershed characteristics, and streamflow help determine the appropriate levels of reservoir storage, amount of freeboard, and outlet capabilities. During construction, bypasses, which may include surface diversions or tunnels, are greatly influenced by hydrologic conditions.

Engineering Geologic Investigations and Exploration

Engineering Geologists and other geotechnical design professionals have a variety of methods and tools available that are used to characterize site conditions for the purpose of addressing design considerations mentioned above.

Smaller dams made from Earth materials can benefit from investigation techniques described in a design manual by Stephens (2010). Small water supply reservoirs and stock ponds usually have little regulatory oversight, yet need to utilize standard practice in investigations and siting and address safety concerns.

Larger more complex dams have been built in the United States using engineering manuals and geologic guidelines developed by various governmental agencies, including the US Army Corps of Engineers (USACE) and US Bureau of Reclamation (USBR). Guidelines for geotechnical investigations and geophysical studies are provided by the USACE (2013, 2004, 1995). The USBR has prepared a two-volume engineering field manual for use by practicing geologists to obtain field data (USBR 1998).

In general, the level of complexity of a field investigation depends on the amount of available preexisting geologic data and how the site characteristics meet the design requirements of a particular dam type. The investigation will follow an iterative approach, beginning with remote sensing, field mapping, and surface geophysics, followed by borings, *in situ* and laboratory testing. Site characterization using long-term monitoring of piezometric or ground deformation instrumentation either before or during construction of the dam verify the site model and assumptions made during design. Manuals and guidelines prepared by the USACE provide a good basis for the proper testing or monitoring program.

Construction Issues and Considerations

The basic requirements of a safe and stable dam include the following (USACE 2004):

- Technical requirements:
  - Dam, foundation, and abutments must be stable under all load conditions.
  - Seepage through foundation, abutments, and embankment must be controlled and collected to prevent excessive uplift, piping, sloughing, and erosion.
  - Freeboard must be sufficient to prevent overtopping by floods and waves and include allowance for settlement of foundation and embankment over time.
  - Spillway and outlet capacity must be sufficient to prevent overtopping.
- Administrative requirements:
  - Ongoing operation and maintenance procedures
  - Monitoring and surveillance plan
  - Instrumentation
  - Documentation of design, construction, and operations
  - Emergency Action Plan
  - Dam safety program

Joints and Shears

Because of the high intact strength of most rock formations, failure generally is considered unlikely, unless it can occur along preexisting joints or fractures (FERC 1999). For failure to occur, movement of the rock wedge must be kinematically possible, that is, the orientation of the trend of the intersection of the rock fractures must normally daylight in a direction which would allow movement to take place under the applied loads with little to no shearing of the intact rock (FERC 2016). For a concrete arch dam, features of primary concern are large wedges of rock in an abutment foundation created by a planar rock fracture or the intersection of two or more rock fractures whose intersection trend daylights in a downstream direction. Joint connectivity also must be considered. Joint connectivity controls whether kinematically possible wedges are small, and of little consequence, or large and capable of compromising the stability of the dam.

If faults, shear zones, or wide joints occur in the embankment foundation, they should be dug out, cleaned, and backfilled with lean concrete to depths equal to several times their widths to provide a structural bridge over the weak zone and to prevent the embankment fill from being placed into the joint or fault.

Foundation Preparation/Treatment

Foundation preparation usually consists of clearing and grubbing to remove vegetation and large roots, and stripping to remove sod, topsoil, boulders, organic materials, rubbish fills, and other undesirable materials. Highly compressible soils occurring in a thin surface layer or in isolated pockets should be removed. After stripping, the foundation surface will be in a loose condition and should be compacted. Fine-grained (silt or clay) foundation soils with high water content and high degree of saturation will be disturbed by compaction efforts with heavy equipment; consequently, lightweight compaction equipment should be used. Traffic over the foundation surface with heavy equipment available can reveal compressible material that may have been overlooked in the stripping, such as pockets of soft material buried beneath a shallow cover. Voids left by stump and tree removals should be filled and compacted by power-driven hand tampers (USACE 2004).

Differential settlement of an embankment may lead to tension zones along the upper portion of the dam and possible cracking along the longitudinal axis in the vicinity of steep abutment slopes, or near the excavation margins separating areas where unsuitable foundation soils were removed and adjacent in-place foundation soils. Differential settlements along the dam axis may result in transverse cracks in the embankment which can lead to undesirable seepage conditions. To minimize this possibility, steep abutment slopes and foundation excavation slopes should be flattened, if feasible, particularly beneath the impervious zone of the embankment. The portion of the abutment surface beneath the impervious zone should not slope steeply upstream or downstream, as such a surface might provide a plane of weakness. The treatment of an Earth foundation under a rock-fill dam should be substantially the same as that for an Earth dam. The surface layer of the foundation beneath the downstream rockfill section must meet filter gradation criteria, or a filter layer must be provided, so that seepage from the foundation does not carry foundation material into the rock fill (Druyts 2007).

Rock foundations should be cleaned of all loose fragments, including semidetached surface blocks of rock spanning relatively open crevices. Projecting knobs of rock should be removed to facilitate operation of compaction equipment and to avoid differential settlement. Cracks, joints, and openings beneath the core and possibly elsewhere should be filled with mortar or lean concrete according to the width of opening.

Figure 4 shows placement of slush grout in exposed foundation rock (phyllite) fractures beneath main dam embankment at Mule Creek dam, Ione, California (1988).

The excavation of shallow exploration or core trenches by blasting commonly creates open fractures. The fractured rock then needs to be removed or treated with grout to seal potential seepage paths in the damaged rock. Where core trenches disclose cavities, large cracks, and joints, the trench should be backfilled with concrete to prevent possible erosion of core materials by water seeping through joints or other openings in the rock.

Limestone and other soluble materials may contain solution cavities and require detailed understanding of the geologic environment, including specialized investigations. The absence of surface sinkholes in karst ground is not sufficient evidence that the foundation does not contain solution features. The need for removing soil or decomposed rock overlying jointed rock, beneath both upstream and downstream shells, to expose the joints for treatment, may also require detailed study. If joints are not exposed for treatment and are wide, material filling them may be washed from the joints when the reservoir pool rises, or the joint-filling material may consolidate. In either case, embankment fill may be carried into the joint, which may result in excessive reservoir seepage or possible piping. An alternative is to provide filter layers between the foundation and the shells of the dam. Such treatment will generally not be necessary beneath shells of rock-fill dams.

Shale foundations should not be allowed to dry out before placing embankment fill, nor should they be permitted to swell prior to fill placement. Consequently, it is desirable to defer removal of the last few feet of shale until just before embankment fill placement begins.

Abutment Preparation/Treatment

Surface irregularities, and cracks or fissures in the cleaned abutment surfaces, can cause problems during placement and compaction of earth fill. Preliminary and final cleaning are commonly required of areas in contact with the core and filters. The purpose of the preliminary cleanup is to facilitate inspection to identify areas that require additional preparation and treatment. Irregularities and overhangs should be removed or reduced to form a uniform abutment slope. Concrete backfill can be used to fill voids beneath overhangs.

Dams, Fig. 4 Slush grouting dam foundation

Vertical rock surfaces beneath the embankment should be avoided or, if permitted, should not be higher than several feet. Benches between vertical surfaces should form a stepped slope comparable to the uniform slope on adjacent areas. Relatively gentle abutments are desirable to avoid possible tension zones and resultant cracking in the embankment.

Foundation Strengthening

Geologic and geotechnical investigations of foundations are required to determine appropriate design and construction parameters. Weak rock foundations generally require gentler embankment slopes than stronger rock foundations. Shallow groundwater and artesian conditions typically require dewatering systems, such as relief wells. Alluvial materials may be susceptible to liquefaction and normally require removal or treatment. Examples of *in situ* treatment include dynamic compaction, grouting (chemical and other), drainage systems, and Cement Deep Soil Mixing (CDSM).

Figure 5 shows CDSM rigs working on ground improvement at toe of zoned soil embankment dam (at Perris dam, Perris, California).

Seepage Control

The purpose of seepage control is to prevent or reduce adverse conditions that may develop, for instance, excessive uplift pressures, slope instability, erosion of the foundation and abutments, and piping through the embankment. Methods for seepage control involve earthwork to construct foundation cutoffs, wide core contact areas or gentle embankment slopes, embankment zonation, and drainage systems. Typically, embankments are constructed with zones, with the permeability increasing progressively from the impervious core outward toward the pervious shells. Transition zones are constructed to ensure filter compatibility between primary zones. The presence and availability of appropriate borrow areas normally determine the types and amounts of zonation.

Drainage systems may include vertical, inclined, or horizontal drains, depending on embankment materials properties and reservoir levels. Horizontal drains are used to control seepage through the embankment and to prevent excessive uplift pressures in the foundation (Druyts 2007). Cutoff trenches are normally employed when the foundation materials are not conducive to grout curtains. Some of the more common seepage control methods are described below:

Foundation Cutoff Trench: All dams on Earth (soil) foundations are subject to underseepage. One of the most successful methods for controlling underseepage is a foundation cutoff trench, in which a trench is excavated beneath the embankment core through pervious foundation strata and then backfilled with compacted impervious material. This method also provides a complete exposure that allows observation of natural conditions, so that the design can be adjusted according to actual ground conditions, permits treatment of exposed foundation material as necessary, provides access for installation of filters to control seepage and piping of soil interfaces, and allows high quality backfilling operations to

Dams, Fig. 5 CDSM treatment of liquefiable toe foundation

be carried out. The cutoff trench should penetrate the pervious foundation and extend into unweathered and relatively impermeable foundation soil or rock.

Slurry Trench: If the depth of a pervious foundation is too great for a backfilled cutoff, a slurry trench cutoff may be a viable alternative method. A slurry trench is excavated through the pervious foundation using sodium bentonite clay and water slurry to support the trench sideslopes. The slurry-filled trench is backfilled by displacing the slurry with a backfill material that contains enough fines to make the cutoff relatively impervious but sufficient coarse particles to minimize backfill settlement. Alternatively, cement may be introduced into the slurry-filled trench which is left to set or harden forming a cement-bentonite cutoff. Slurry trench cutoffs are not recommended when boulders or open jointed rock exist in the foundation due to difficulties in excavating through the rock and slurry loss through the open joints. Normally, the slurry trench should be located under or near the upstream toe of the dam. Piezometers located both upstream and downstream of the cutoff are needed to determine if the slurry trench is performing as planned.

Concrete wall: A concrete cutoff wall may be considered for seepage control; a pervious foundation is excessive and/or contains cobbles, boulders, or soluble material (e.g., limestone). The concrete cutoff is typically a cast-in-place continuous concrete wall constructed by tremie placement of concrete in a bentonite-slurry supported trench. Concrete cutoff walls are rigid and susceptible to cracking when subjected to strong earthquake shaking and therefore may not be used in severe seismic environments.

Upstream impervious blanket: An upstream impervious blanket tied into the impervious core of the dam may be used to reduce underseepage when the reservoir head is not great. The effectiveness of upstream impervious blankets depends upon the length, thickness, and vertical permeability and on the stratification and permeability of soils on which they are placed. Downstream seepage control measures (relief wells or toe trench drains) are generally constructed to complement the upstream blanket.

Relief wells: Relief wells installed along the downstream toe of the dam may be used to prevent excessive uplift pressures and piping through the foundation. Relief wells may be used in combination with other underseepage control measures. Relief wells are particularly useful where a pervious foundation has impervious overlying strata. The well section should penetrate the pervious foundation strata to obtain pressure relief. It is important that relief wells are accessible for cleaning, sounding for sand, and pumping to determine discharge capacity. Relief wells should discharge into open ditches or into collector systems located away from the dam, and independent of toe drains or surface drainage systems. Well discharge can gradually decrease with time due to clogging of the well screen and/or reservoir siltation.

Grouting: Grouting is a common method of controlling seepage in rock foundations, where seepage can occur through cracks and joints (Weaver and Bruce 2007). The principal objectives of grouting in a rock foundation are to establish an effective seepage barrier beneath the dam and to strength the foundation. The effectiveness of grouting depends on the structural characteristics of the rock (crack width, spacing, length, filling, etc.) as well as on grout mixtures, equipment, and procedures. Spacing, length, and orientation of grout holes and the procedure to be followed in grouting a foundation are dependent on the height of the structure and the geologic characteristics of the foundation. Grouting beneath a dam commonly takes two forms: (1) shallow, lower-pressure grouting of a large area of the foundation and (2) deeper, higher-pressure construction of a grout curtain using or more rows of drilled holes more or less along the axis of the dam. The design and construction of grouting programs requires consideration of site geology, recognition of specific intent of the program, development of grouting specifications, and execution and documentation of construction by experienced personnel.

A grout curtain is constructed by drilling grout holes and injecting a grout mix. It is common to drill and inject grout to multiple depths at different hole spacing. For example, shallower injection may take place in more closely spaced holes, whereas deeper injection may take place in more widely spaced holes. However, site geologic conditions, with knowledge of rock features such as shears and joints, provide the basis for design of the grout curtain. In addition, once grouting has been initiated, the grouting program can be adjusted as drilling yields additional geological information and observations of grout take and other data become available.

“Blanket” grouting refers to shallow grouting beneath embankment dams in order to reduce seepage through the foundation and prevent loss of core material into the foundation. “Consolidation” grouting is performed to strengthen the foundation beneath concrete dams, with the primary purpose of reducing settlement of the structure. Both methods typically are performed in a geometric pattern; however, investigation of foundation geology is performed prior to specific design of the grouting program.

The effectiveness of a grouting operation is evaluated by confirmatory drilling to observe grout filling of joints or other permeable zones and by performing pre and postgrouting water pressure testing (“packer tests”).

Figure 6 is a photograph of drilling for remedial foundation curtain grouting in an existing concrete dam (New Bullards Bar Dam, Yuba County, California).

Dams, Fig. 6 Curtain grouting an existing dam

Foundation drainage (concrete dams): Despite the construction of seepage control measures, water will still find paths through the foundation and structure. Foundation drainage is critical to intercepting and removing water to that it does not build up excessive hydrostatic pressures on the base of the structure. Foundation drainage typically involves drilling one or more rows of drain holes downstream from the constructed grout curtain. Like the grouting parameters, the depth, size, and spacing of drain holes are determined from foundation rock conditions. Drain holes are drilled from galleries within the dam or from the downstream face of the dam if galleries are not present. Drainage from the drain holes should be collected and conveyed to appropriate discharge locations downstream from the dam.

Dam Safety and Long-Term Performance

Concepts and procedures described below explain dam safety in terms of the United States regulatory and administrative situation. Regulations vary elsewhere but are broadly similar in most developed countries.

A variety of sources of information on dam safety is available. In the United States, the Federal Emergency Management Agency (FEMA) is responsible for coordinating government-wide relief efforts if dam failure occurs. The Federal Energy Regulatory Commission (FERC) licenses and inspects private, municipal, and state hydroelectric projects.

Concepts

The impoundment of water creates a potential hazard to public safety. Dam owners are solely responsible for keeping their dams safe and for performing and financing maintenance, repairs, and upgrades. Maintaining a safe dam is a key element in preventing failure and limiting liability. The purpose of a dam safety program is to recognize the potential hazards, monitor-specific elements contributing to hazards, keep operators aware of potential hazards, and acting to reduce or mitigate contributions to hazards if and when they develop.

Dam failure is usually defined as the uncontrolled release of water and does not necessarily require a catastrophic release. A hazard potential classification is a system that categorizes dams according to the degree of adverse incremental consequences from failure or misoperation that does not reflect on their current condition (FEMA 2016). Various governments and agencies may have different definitions; however, typical categories include:

- High hazard potential – loss of one or more human life is probable.
- Significant hazard potential – no probability of loss of human life, but possible economic loss, environmental damage, disruption of lifeline facilities, or other impacts.
- Low hazard potential – no probability of loss of human life and low economic and/or environmental losses.

The United States pursues dam safety through the National Dam Safety Program (NDSP). The NDSP is operated by FEMA and works with government and private sectors to educate and provide financial assistance to State dam safety programs. The United States Army Corps of Engineers (USACE) maintains the National Inventory of Dams (NID), which contains information on more than 87,000 dams in the United States.

Dam Safety Assessments

Periodic inspections and evaluations are essential to long-term public safety. The objective of periodic evaluations is early identification of conditions that could disrupt operations or threaten dam safety. The evaluations include visual inspections of the dam and reservoir, outlet works, spillways and appurtenant structures, and review of instrumentation and dam performance records.

A complete dam safety assessment includes two components: (1) inspection and data review and (2) analysis and recommendations. The inspection component involves an onsite examination of the dam, reservoir and pertinent auxiliary structures, and a review of design, construction, operation, and maintenance drawings and records. The analysis component includes development of appropriate action items to address, confirm, or correct identified deficiencies and supporting technical analyses.

Dam safety assessments are typically performed at 3 to 6-year time intervals, depending on regulatory jurisdiction. In the United States, most dams greater than a certain size fall within one or more of the following jurisdictions: FERC, USACE, USBR, and individual State dam safety agencies. Many small dams may not fall under Federal or State jurisdiction, but should still be inspected on a periodic basis. States regulate about 80% of the dams in the United States, with the Federal government regulating the remaining jurisdictional dams (FEMA 2016).

The engineering and geologic dam safety deficiencies identified from the on-site examinations are described in a written report and further assessed through evaluations and analyses, as appropriate.

The types of deficiencies and recommendations encompass a wide range of issues that normally apply to dams. These typically include the seismotectonic, geologic, geotechnical, hydrologic, hydraulic, mechanical, and structural issues. Supporting analyses use the state-of-the-art technology and methodology available within the various disciplines. The analyses are conducted using a phased approach. The first phase includes a technical assessment using available data and conservative assumptions to determine whether the identified deficiency is a significant dam safety issue. Results of the first phase technical assessment can conclude one of the following:

1. No further action is required because the threat to the safety of the dam is low or negligible.
2. A threat to the safety of the dam clearly exists, and a corrective action should be determined.
3. Additional field, instrumentation, or analytical studies are required to further assess the deficiency.

If the results of the first phase assessment are inconclusive or confirmed (items 2 or 3 above), a second phase of study may be required. The follow-up phase involves more detailed study, which may include field investigations, data acquisition, and laboratory tests to establish the necessary design parameters for more sophisticated analyses.

Common Dam Safety Deficiencies

Embankment Dams

- *Seepage* – Seepage is always a potential problem in Earth dams, and especially in homogeneous embankments that do not have impermeable cores or cutoffs, filter zones, and drains. Seepage may be caused or exacerbated by conditions allowing the formation of permeable ground or subsurface paths for water to migrate, such as poor compaction, animal burrows, tree roots, or leaks in conduits. Excessive seepage can lead to piping (internal erosion), instability, and eventual failure of all or part of the downstream face (Schmertmann 2002). Careful monitoring is useful in determining whether or not seeping water is indicative of internal erosion. Clear water is generally an indication that internal erosion is not occurring; however, care must be taken to ensure that the observations are representative of the entire seepage condition, and not simply missing sediment that may have settled out upstream of the observation point. An increase in flow quantity over time may indicate formation or increase in internal erosion (Brown and Bridle 2008). Vegetation can obscure adequate seepage observations. Collection boxes with v-notched weirs are commonly used to observe and measure seepage flow.

Figure 7 shows lush vegetation on downstream slope of small embankment dam is indication of seepage (agricultural dam, Santa Clara County, California).

Seepage is commonly prevented or controlled by countermeasures such as filters, drains, clay blankets, and flatter slopes. However, when such elements are not already part of the original construction, then considerable re-construction may be needed to mitigate excessive seepage and help improve the performance of the dam. The objective of seepage "filter" drains is to lower the phreatic surface within the embankment to prevent water from emerging from the downstream slope where erosive and absorptive flows could cause slumping of the material and endanger the whole structure. A few specific seepage conditions are highlighted below:

– *Seepage Flow Adjacent to Outlet Pipe* – A break or hole in the outlet conduit, or poor compaction around the conduit, can allow water to flow and create a pathway along the outside of the outlet pipe. Careful inspection of the outlet pipe and discharge point is needed to identify this type of seepage.
– *Seepage Water Exiting as a Boil Downstream of Dam* – Seepage emanating downstream from the dam is an indication that some part of the foundation is providing a path for reservoir seepage. The flow path may be provided by pervious material (e.g., sand or gravel) or geologic feature (e.g., shear zone) in the foundation.
– *Seepage Flow from Abutment Contact* – Water flowing through pathways in the abutment or along the embankment-abutment contact can result in internal erosion. Monitoring should be performed to detect changes in flow quantities over time.
– *Sinkholes* – Sinkholes or subsidence can result from internal erosion (piping) of underlying embankment materials. An eroded pipe in the embankment, cavity in the foundation, or leakage from an outlet pipe can result in subsidence and development of sinkholes.
– *Slope Instability (Slide, Slump or Slip)* – Embankment or foundation deformation can result from oversteepened slopes or, over-loading of weak foundation materials or shear zones, and can lead to instability of embankment slopes. Cracking, settlement, and bulging at the toe are

Dams, Fig. 7 Dam Seepage

typical indicators of slope instability. Reservoir rim instability can cause inlet obstructions, wave erosion of the dam, or (if large enough) seiches that can overtop the dam.

- *Dam Crest Cracking and Settlement – Transverse* cracking (perpendicular to crest alignment) can be caused by differential settlement between embankment materials, slope instability, or internal erosion. Seepage through cracks could initiate a breach in the embankment. *Longitudinal* cracking (parallel to crest alignment) can be caused by earthquake shaking, deformation of embankment materials, differential settlement, or slope instability. Excessive or differential settlement can lead to depressions in the dam crest. Periodic surveying is required to monitor the elevation of the dam crest. When abnormally low areas are detected, corrective actions may be required in accordance with dam safety procedures.
- *Surface Erosion* – Development of erosional rills and gullies may result from intense rain or snowmelt and can lead to deterioration of embankment slopes. If detected early, minor grading or planting of protective grasses could resolve surficial erosion. More extensive grading, drainage diversion, or placement of rock or riprap may also be required.
- *Toe Erosion from Outlet Releases* – Scour or erosion from outlet pipe discharge can result in damage or disturbance to the toe of dam embankments. Progressive erosion may result in larger instability of the embankment slope. Corrective actions may include extending the discharge location farther downstream, constructing an energy dissipating structure and/or protective rock or riprap, and constructing a stilling basin.

Other common deficiencies in embankment dams include deteriorated or missing riprap on embankment slopes, erosion from livestock and cattle traffic, animal burrows leading to shortened seepage paths and excessive vegetation.

D

Concrete or Masonry Dams

Concrete or Masonry Dams may over time outlive their usefulness or become a failure risk due to flooding or seismic events. If owners determine the benefit of removal outweighs that of remediation, then removal is an option. One example is San Clemente Dam in Carmel, California, USA. The dam impounded reservoir was over 90% full of sediment and did not provide water supply, flood control, or adequate fish passage. In addition, the dam was susceptible to failure due to a credible earthquake or a major flood event. As such the dam was removed. Figure 8 shows 106-ft-high concrete arch San Clemente dam being removed (2015).

Dams, Fig. 8 Concrete Arch dam removal

- *Cracking, Opening/Closing or Offsets at Joints* – Structural cracking, broken masonry, opening/closing or offsets at joints, and other apparent deformation can be indications of structure-foundation problems that need to be evaluated. Adverse conditions in foundations are a common cause of concrete and masonry dam failures.
- *Excessive Hydrostatic Uplift* – The build-up of hydrostatic pressures beneath concrete and masonry structures can be caused by poor foundation seepage conditions. Bedrock foundations can be pervious due to the presence of fractures, shears, and other geologic conditions. It is important that adequate foundation drains are constructed to reduce the potential build-up of excessive hydrostatic pressures. Monitoring of seepage, drains, and hydrostatic pressures are important elements of safety programs for concrete and masonry dams.
- *Deterioration of Structural Materials* – Deteriorated concrete and masonry materials may have lower strength and less ability to carry reservoir loads imposed on the dam. Periodic inspections and monitoring are typically conducted to evaluate structural materials.

Spillways

- *Excessive Vegetation or Debris in Spillway Channel or Inlet* – Obstructions in spillways can reduce the capacity to convey flow. Debris, vegetation, and other accumulated materials should be periodically removed to maintain spillway capacity. Log/debris booms can be placed in the reservoir to reduce floating debris from entering the spillway.
- *Erosion of Unlined Spillway Channel* – Erosion of unlined spillway channels can result in reduced capacity, unintended or uncontrolled releases, and adverse impacts on the dam. Spillway channels should be inspected along with dam inspections, and adverse conditions should be corrected.

Summary

Dams are designed and built to utilize the natural topographic setting or hydraulics of a river or stream for the benefit of humankind. Dams and the reservoirs they impound are classified by either the use or the shape and materials of its design. Table 1 provides a brief summary of dam classification.

Understanding the geology of a site is important with respect to economic benefit, safety concerns, and function of the dam. Key to the viability of a dam is the amount of site preparation needed, access to construction materials, effect of storm runoff or seismic impacts, and external economics (unique to hydropower schemes). Developing a geotechnical program that implements these key parameters is essential. Design guides are available and are used universally to ensure the performance and safety of dams. Factors such as environmental impacts, a reduction in geologic hazards, and dams reaching their design life have necessitated the need to retrofit and in some cases remove dams.

Dams, Table 1 Dam classification

Classification of dams			
Classification by use		**Classification by material or shape**	
Type	Uses	Type	Attributes
Storage	Water supply, recreation, wildlife, or hydroelectric power generation	Earthfill or earth embankment	Large footprint, abutment spillway, zoned with internal drainage, derived from site materials
Diversion	Hydraulic head for water conveyance systems (canals, ditches, tunnels).	Rockfill	Large footprint, abutment spillway, impervious barrier, noncompressible foundation
Detention	Retard debris or flood runoff to reduce the downstream impacts of sudden floods	Concrete arch or gravity	Overtop spillway structure, smaller footprint, sufficiently strong abutments, requires some imported materials
Other	Temporary cofferdam, Tailings for mine waste, navigation (lock system)	Masonry, metal, block or ice core	Smaller, lack sufficient onsite materials, uncommon

Cross-References

▶ Blasting
▶ Boreholes
▶ Cement
▶ Clay
▶ Cofferdam
▶ Consolidation
▶ Dewatering
▶ Earthquake
▶ Erosion
▶ Excavation
▶ Faults
▶ Field Testing
▶ Cut and Fill
▶ Foundations

- ▶ Geohazards
- ▶ Groundwater
- ▶ Grouting
- ▶ Hydrology
- ▶ Instrumentation
- ▶ Liquefaction
- ▶ Piezometer
- ▶ Reservoirs
- ▶ Rock Properties
- ▶ Site Investigation
- ▶ Tunnels
- ▶ Water

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D

Darcy's Law

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Definition

Darcy's law defines the rate of water flow through porous media, assuming a laminar flow. It states that the rate of flow per unit, cross-sectional area is equal to the product of the hydraulic conductivity of the material and the hydraulic gradient.

Overview

Henry Darcy (1803–1858), a French waterworks in Dijon, revealed a proportionality between the flow rate in clean sand and the applied hydraulic gradient. In its simplest form:

$$q = vA = kiA = k\frac{\Delta h}{L}A \qquad (1)$$

where q is the flow rate through the cross-sectional area (A); v is the flow velocity; i is the hydraulic gradient, calculated as the difference in pore pressure head per unit length ($\Delta h/L$); and k is the hydraulic conductivity.

Use

Darcy's law is widely used in the fields of Engineering Geology, Hydrogeology, Geotechnical Engineering, and

Environmental Sciences, in particular for estimating seepage rates, flow patterns, drainage, and contaminant transport through Earth materials. Darcy's law can be expanded for fluids other than water, for three-dimensional flows through anisotropic materials, and to consider inertial effects and drag forces (Nield and Bejan 2006).

Cross-References

- ▶ Engineering Geology
- ▶ Geotechnical Engineering
- ▶ Hydrogeology
- ▶ Pore Pressure

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Databases

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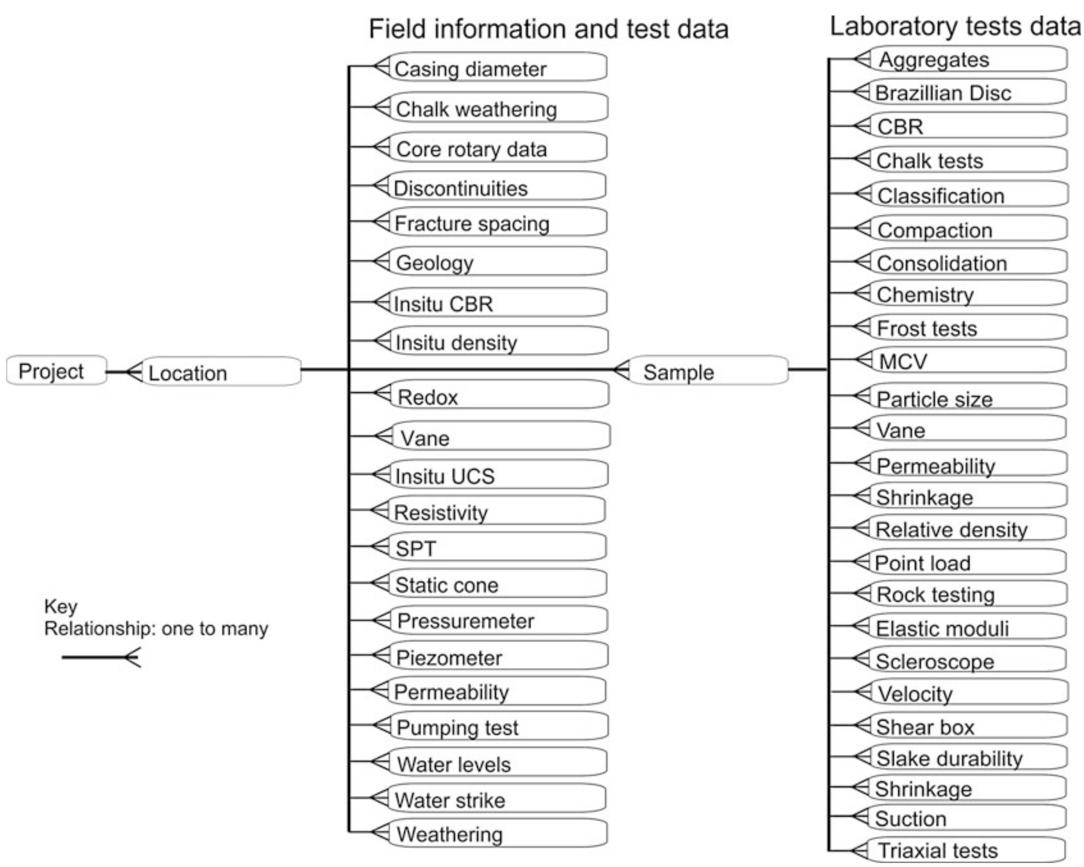
Definition

An organized collection of related data and information.

Databases have been used for many hundreds of years, but the advent of the computer provided a mechanism for cost effective and efficient databases. They exist as a shared collection of logically related data and a description of the data, designed to meet the information needs of an organization. They are structured for efficient management, addition, editing, storage, and retrieval of data and information.

Data modeling is the process of defining and analyzing data requirements needed to support the design process stages: conceptual, logical, and physical representation of the model (Rasmussen 1995; Nayembil and Baker 2015).



Databases, Fig. 1 Entity relationship diagram for the British Geological Survey (BGS) National Geotechnical Properties Database

Data models are logical, structured, representations of the things that need to be described in databases. The data structure is represented by **entities**, entity properties called **attributes** and the **relationships** between **entities**. ***An entity*** is a logically grouping of information about a "real world" thing or concept, such as a borehole, sample, or laboratory test type. **Entities** often become tables in database implementations of data models with ***attributes*** often forming the columns. The **relationships** between **entities** are often shown in an entity-relation model. Figure 1 is an example the entity-relation model for a simplified version of the British Geological Survey, National Geotechnical Properties Database (Self et al. 2012). This is based on the Association of Geotechnical and Geoenvironmental Specialists (AGS) digital data transfer format (Bland et al. 2014). The **entity-relation model** with examples of attributes:

Project includes **attributes** for the name of the project, the contractors name, and project date.

Location of a borehole, pit, or other that includes the **attributes** for easting, northing, and ground level to OD.

Twenty-four **entities** for field information and test data, which all have **attributes** of depth to top and base, this includes **samples** which are used in:

Twenty-six **entities** for laboratory test data, which include **attributes** for test data.

Other geological data models are available from the Earth data models website (www.earthdatamodels.org).

Database management system (DBMS) is the computer software application in which the users define, create, add, update, manage, administer, and access the data. Accessing the data is done through queries. The query language used is SQL (Structured Query Language). The computer-based databases can be used by several people at the same time and the software generally includes access control to the database, such as read, write, and editing privileges.

Types of Database

There are a number of database types including hierarchical and object-based database models, but the most commonly used is the relational model.

Relational Model (for Database Management)

This is a digital collection of **entities** organized on a relational model of the data (Codd 1970). The **entities** are organized as a set of formally described tables, each table containing closely related data as defined in the data model. The data can be accessed, via queries using SQL or reassembled in many different ways, without reorganizing the tables. **Dictionaries** can be defined within the database, to constrain and define values. There are a number of software packages to support the database (so-called relational database management system or RDBMS) and a large literature base.

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D

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Deformation

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Synonyms

Strain

Definition

Deformation. Change in size, shape, and/or volume of an object under the effect of internal or external forces.

Introduction

In continuum mechanics, as well as in engineering applications, deformation is often referred to as the ▶ strain induced when external forces are applied to a body (e.g., ▶ compression, tension, shearing, bending, and/or torsion). However, deformation can be also induced by intrinsic body forces (e.g., gravity), as well as by changes in the temperature or by chemical reactions (Jones 2009). A straightforward example of deformation is shown in Fig. 1, where a force is axially applied to a rod. The strain (ε) occurring along the rod axis can be calculated as the change in length ΔL with respect to the initial length:

$$\varepsilon = \frac{\Delta L}{L_1} = \frac{L_2 - L_1}{L_1} \tag{1}$$

where L_2 and L_1 are the final (deformed) and initial (undeformed) rod lengths, respectively. Deformation is dimensionless, being a ratio between length units. Conventionally, values of deformation are expressed in terms of "microstrains."

More in general, by considering the spatial variation (gradient) of the vector components associated to the deformed configuration (x) of a continuous* (*Note: differentiation requires continuity) body, with respect to each component of the vector associated to the original (undeformed) configuration (X), we can write:

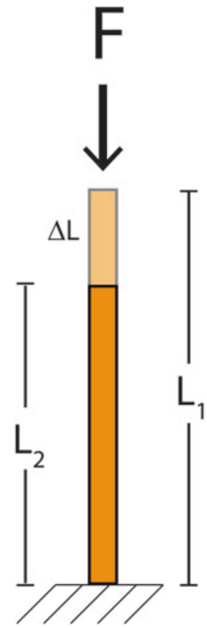
$$F_{ij} = \frac{\partial x_i}{\partial X_j} \tag{2}$$

F_{ij} is known as the "deformation gradient tensor" (Fig. 2), and fully describes the rotation, shearing, and stretching behavior of a continuous body (Hashiguchi 2013).

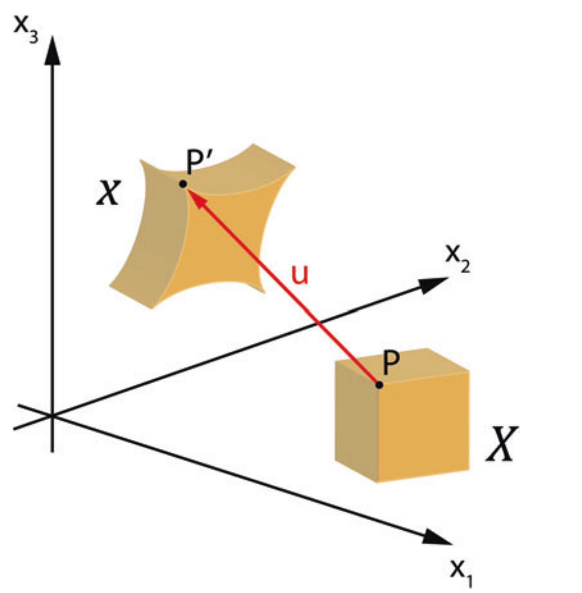
At infinitesimal scale, the concept of deformation is closely associated to this of displacement. The latter is defined as the change in the configuration of a body and is composed of two main elements: (i) rigid-body roto-translation and (ii) change of shape and/or size (i.e., the deformation). Displacement vectors can be obtained by evaluating relative variations between fiducial points, that is, measuring their change in separation (baseline). Displacement (u) at every point of a continuous body can be written as:

$$u = x - X \tag{3}$$

Combining the definitions (2) and (3), the deformation gradient tensor can be reformulated as:



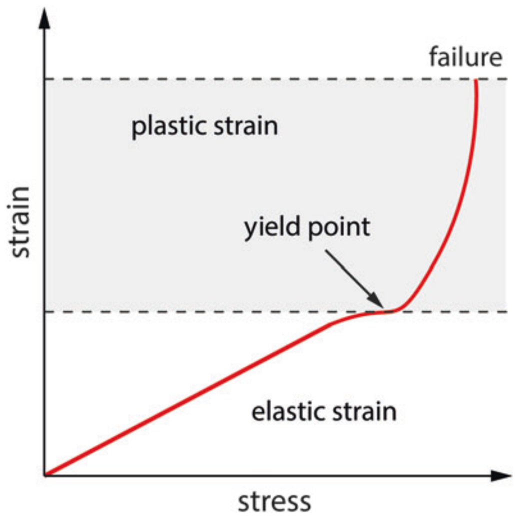
Deformation, Fig. 1 1-D deformation of a rod subjected to axial stress



Deformation, Fig. 2 Deformation in 3-D space. P and P' are the positions of a fiducial point before and after deformation, respectively, while u is the displacement vector

$$F_{ij} = \frac{\partial}{\partial X_j}(X_i + u_i) = I + \frac{\partial u_i}{\partial X_j} = I + D_{ij} \tag{4}$$

where I is the identity matrix and D_{ij} is the "displacement gradient tensor." From this formulation it is possible to highlight that deformation always induces displacement, but displacement does not always imply deformation.



Deformation, Fig. 3 Typical deformation behavior of elastoplastic materials when stress is applied progressively

Deformation in Engineering Geology

In engineering geology applications, the main interest is in the deformation behavior of two classes of materials, that is, rocks and soils, as well as the fluid and gases confined within (Price and De Freitas 2009). Laboratory and field tests provide a framework for the analysis of deformation of different scales. As an example, Fig. 3 shows the typical evolution of strain when a load is applied progressively to a rock sample. The linear portion of the plot refers to as the elastic deformation experienced by the specimen. Elastic deformation is commonly related to ▶ stress by ▶ Hooke's law. Ideally, elastic materials recover their initial configuration as soon as the forces are released. However, in most cases part of the deformation experienced is irreversible, and thus their behavior is described as plastic or elastoplastic deformation (Hashiguchi 2013). Excess deformation of a material can lead to damage, generate factures, and subsequently lead to ▶ fail ure. The deformation behavior of soils is typically described as ▶ compaction and/or ▶ consolidation.

Summary

Deformation (or strain) refers to the change in size/shape of an object under the effect of forces. In engineering geology applications, surface and subsurface deformation can be measured directly and/or indirectly by using several ▶ monitoring instruments and methods at different spatial and temporal scales, including ▶ extensometers, ▶ tiltmeters, ▶ inclinometers, geodetic tachymeters and levels, total stations, GPS, and differential ▶ InSAR. The analysis and interpretation of rock and soil deformation is often a key information necessary for understanding engineering geology problems.

Cross-References

▶ Compaction
▶ Compression
▶ Consolidation
▶ Extensometer
▶ Failure Criteria
▶ Hooke's Law
▶ Inclinometer
▶ InSAR
▶ Monitoring
▶ Strain
▶ Stress
▶ Tiltmeter

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Density

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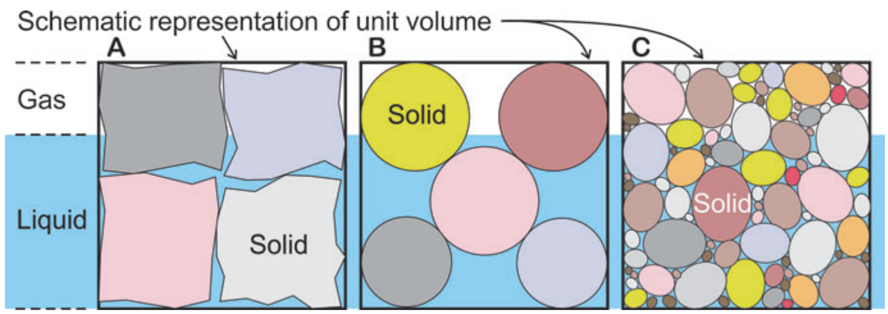
Definition

Density is a property dependent upon the compactness of matter, defined as the mass contained within a volume. With units of grams per cubic centimeter (g/cm^3, in the CGS system) or kilograms per cubic meter (kg/m^3, in the MKS system), the density of soil and rock depends on the volume of the mineral particles, and the volume of water and air contained within the voids between the particles and within the particles (Fig. 1). Wet density and dry density are calculated to indicate the *in situ* material condition relative to the solid components. Density of a material can also be expressed as the specific gravity, a dimensionless parameter, by relating the material density to that of water at 4 °C.

Density is an indicator of the engineering properties of the material; the presence of void space within soils and rocks increases porosity, decreases strength, increases deformability, permits higher water content, and may increase permeability. The measurement of density also provides an indication of the degree of compaction achieved when working with soils as an engineering material.



Density, Fig. 1 Schematic diagram representing three phases of material in a volume of earth materials: solid, liquid, and gas. (**a**) Angular particles with relatively small void volume (pore space). (**b**) Uniform rounded particles with a small range of sizes and relatively large void volume. (**c**) Uniform rounded particles with a large range of sizes and relatively small void volume. Small void volume correlates to higher density

When testing a soil or rock sample, the bulk density is measured, which includes the pore space between and within the mineral particles. As such, bulk density is inversely correlated to porosity – the greater the proportion of void space in higher porosity materials, the lower the bulk density. In the case of soil, the bulk density is dependent on the degree of compaction or consolidation and is, therefore, not an intrinsic material property, but it is an engineering property.

The bulk density of rock and soil can be determined in a number of different ways depending on the configuration and volume of the material sampled (ISO 11272:2017; Brown 1981). In the first method, which is used for testing the density of both rock and soil, the volume of the material within a cube (or other right regular prism) or cylindrical sample (core, pushed cylinder) of known mass can be calculated from dimensions measured with a caliper. When it is not possible to extract an undisturbed sample of the soil, in low-cohesion granular soils, for example, or fine soils containing larger fragments, then a sample of material is extracted from a hole or small test pit, and its mass is measured. The volume of the hole can be determined by measuring the volume of a material required to fill the hole: whether: (1) water in a rubber balloon (ASTM D2167) or contained by a plastic lining (ASTM D5030), (2) sand poured into the hole to replace the excavated volume (ASTM D4914 and ASTM D1556), or (3) filling the hole with plastic beads of known volume and packing density. An additional method may be used to measure or calculate the density of irregular-shaped pieces of rock or soil using Archimedes' principle. In this case, the material is submerged in water, and the volume is determined from the volume of water displaced. In this case, if the material can absorb water or dissolve, the pieces are coated with a waterproofing material such as wax (paraffin), which introduces some additional volume to the samples and therefore some error into the density calculation (ASTM D7263; Brown 1981). Tests may also be conducted on remolded soils (ASTM D7263).

Instruments exist to measure *in situ* density, for example, electromagnetic gauges (ASTM D7830), nuclear methods (ASTM D6938 and D5195), time domain reflectometry (ASTM D6780), and correlation with complex impedance (ASTM D7698).

Density of material can range from values of ~1.2 to 2.4 Mg/m^3 for soils, ~1.6 to 3.2 Mg/m^3 for sedimentary rocks, and ~2.35 to 3.5 Mg/m^3 for igneous and metamorphic rocks. Soils containing organic materials will be less dense.

Cross-References

- ▶ Compaction
- ▶ Consolidation
- ▶ Engineering Properties
- ▶ Field Testing
- ▶ Igneous Rocks
- ▶ Rock Field Tests
- ▶ Rock Properties
- ▶ Sedimentary Rocks
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Properties
- ▶ Voids
- ▶ Water

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ASTM D2167. Standard test method for density and unit weight of soil in place by the rubber balloon method
ASTM D4914. Standard test methods for density of soil and rock in place by the sand replacement method in a test pit
ASTM D5030. Standard test methods for density of soil and rock in place by the water replacement method in a test pit
ASTM D5195. Density of soil and rock in-place at depths below surface by nuclear methods
ASTM D6780. Water content and density of soil in situ by Time Domain Reflectometry (TDR)
ASTM D6938. In-place density and water content of soil and soil-aggregate by nuclear methods (shallow depth)
ASTM D7263. Laboratory determination of density (unit weight) of soil specimens
ASTM D7698. In-place density (unit weight) and water content of soil using an electromagnetic soil density gauge
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Desert Environments

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Definition

Environments with large contiguous areas and low vegetation cover due to dry conditions.

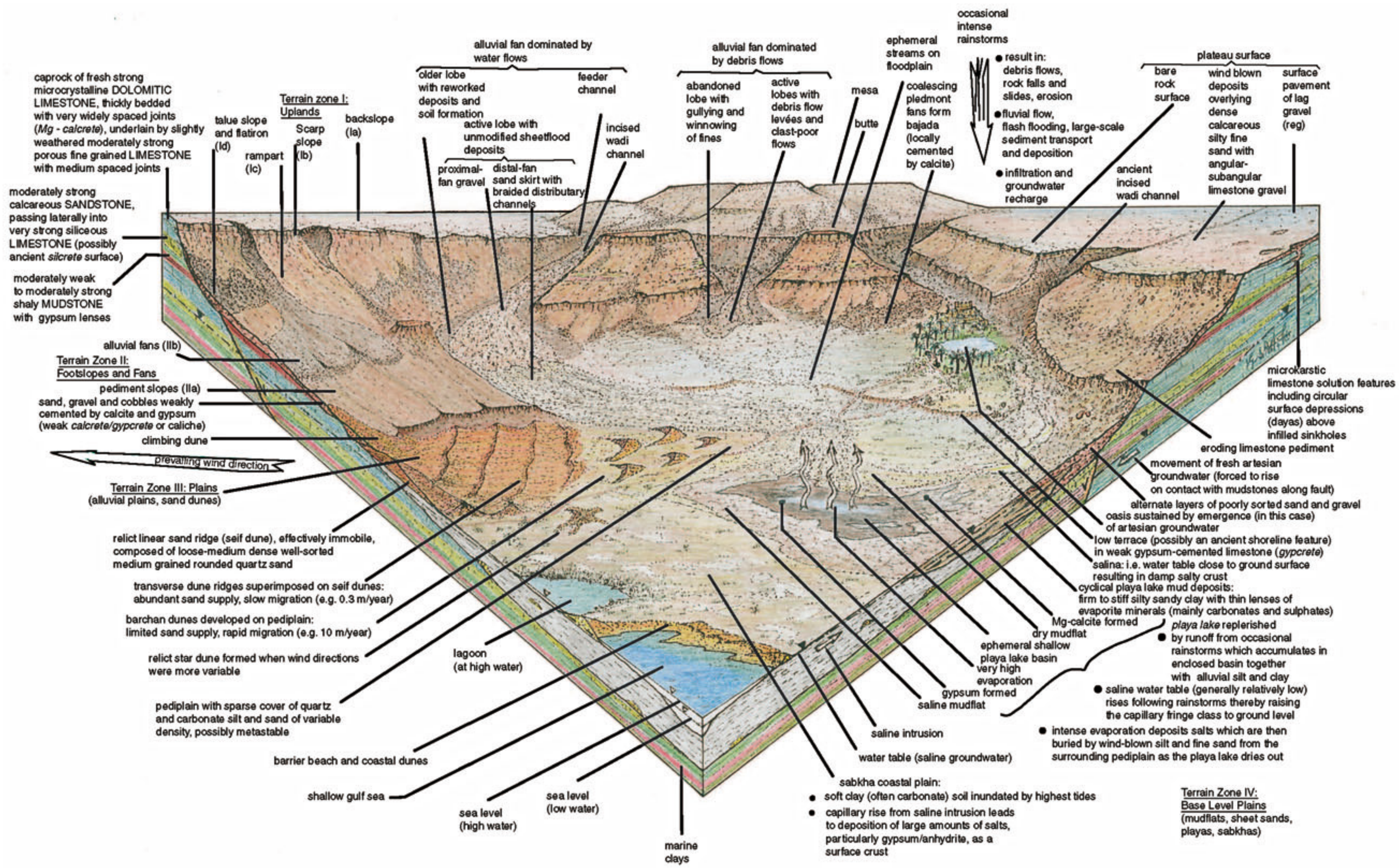
A desert can be defined by physical, biological, and climatological characteristics (UNEP 2006) with distinct hazards and engineering issues and solutions (Griffiths and Stokes 2012). Physically, they cover large surface areas with low vegetation cover developed into thin soils. Biologically, plants and animals are adapted for dry conditions. Climatologically, they are defined by moisture availability or temperature (Nash 2012). Moisture reflects (1) water supply via precipitation and (2) water loss from evaporation and plant transpiration, with classification into hyperarid, arid, semi-arid, and dry subhumid settings. Temperature regimes possess wide temporal variability, with classifications differentiating between hot/cold all year round and those with mild, cool or cold winters.

Desert Distribution and Controls

Deserts cover large areas of the Earth's land surface (47%), as polar (cold deserts) or low latitude concentrations (hot deserts; e.g., Sahara) (Nash 2012). Distributions have expanded and contracted in accordance with Pleistocene climate changes, with atmospheric-oceanic circulation patterns causing aridity influenced by the connectivity and proximity to moisture laden air masses. Geological controls on relief, landmass positioning, extent, and rock type can result in orographic, ocean current routing, continentality, and albedo effects that operate in combination with climate and groundwater controls (Nash 2012).

Desert Processes

The aridity means deserts possess distinct geomorphological processes that condition and shape the land surface (Fookes et al. 2007; Griffiths et al. 2012; Fig. 1). Weathering is pervasive, low rate, and restricted to surface settings. Temperature and moisture variations mean landscapes are subjected to physical (e.g., insolation, frost shattering), chemical (e.g., salt), and biological weathering processes, conditioning the land surface for wind and water erosion. Wind (aeolian) processes can dominate deserts creating grain abrasion and dune encroachment/burial hazards (Griffiths et al. 2012). Wind is generated by atmospheric pressure differences with air mass movement from high to low pressure areas. Reduced soil moisture and vegetation absences generate dust storms, with land surface stripping (deflation) and erosional shaping (e.g., yardangs). Aeolian sand deposition (ripples, dunes, sheet bedforms) typically occurs close to wind erosion areas, while dust (including Pleistocene cold-climate loess) can be transported further, sometimes over continental scale distances. Despite aridity, surface water flow and groundwater are important within deserts (Griffiths et al. 2012). Surface water flow is infrequent but of high magnitude and spatially localized, occurring as unconfined overland or channelized flows. Flood flows have high sediment loads leading to scour and pronounced sediment deposition with significant hazard. Water infiltration into the subsurface can lead to desiccation, salt formation (e.g., gypsum) and dissolution, forming duricrusts and expanding/collapsing soils, with cap rocks, voids, and piping being significant hazards. Elevated rainfall combined with high groundwater levels can form temporary lakes or saline coasts (sabkhas). Here, saline and highly cohesive soils are common salt pan features, with deflation and dust transport if subjected to wind erosion.



Desert Environments, Fig. 1 Terrain model of a typical desert environment in a hot nonpolar climatic setting (Fookes et al. 2007) (Reproduced with permission from Whittles Publishing)

Cross-References

- ▶ Aeolian Processes
- ▶ Fluvial Environments
- ▶ Biological Weathering
- ▶ Cap Rock
- ▶ Chemical Weathering
- ▶ Climate Change
- ▶ Coastal Environments
- ▶ Collapsible Soils
- ▶ Desiccation
- ▶ Dissolution
- ▶ Erosion
- ▶ Expansive Soils
- ▶ Floods
- ▶ Fluvial Environments
- ▶ Hazard
- ▶ Lacustrine Deposits
- ▶ Physical Weathering
- ▶ Sabkha
- ▶ Saline Soils
- ▶ Sand
- ▶ Voids

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Desiccation

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Synonyms

Soil drying

Definition

The process in which wet soils dry and soil moisture content decreases as the moisture evaporates into the surrounding environment, leading ultimately to cracking of the ground surface.

During desiccation, the bulk water pressure within the soil pores will become negative with respect to the atmospheric pressure. This depression of pressure (i.e., the difference between atmospheric pressure and bulk water pressure) is known as the soil matric suction and is associated with the formation of curved water menisci within soil pores. On the basis of the capillary tube principle, the soil suction that can be sustained within a soil pore meniscus may be represented as $2T\cos\theta/R$, where T is the water surface tension, θ is the wetting angle, and R is the radius of wetted part of soil pore. Therefore, the smaller the pore size, the higher the suction that can be sustained before soil becomes dry. Hence, clay soils tend to retain more moisture during drying than coarse-grained soils like sand, under the same ambient conditions. The rate of moisture evaporation is proportional to the difference between the vapor pressure of the soil pores and that of air directly above the soil and is also dependent on temperature (Wilson et al. 1994). Typically until suction is close to 3000 kPa, the moisture evaporation rate from soil is similar to that from a water surface. Above this suction, the evaporation rate will drop causing the soil to desiccate at a diminishing rate.

Desiccation Cracking

Soil can shrink during desiccation, the degree to which depends mostly on soil minerology and particle size. For instance, clay soils shrink more than sand due to desiccation. Soil shrinkage occurs in response to soil suction pulling soil particles closer. Desiccation cracking occurs when the desiccating soil is restrained against free shrinkage (Kodikara and Costa 2012). The restraints could come from the friction at the boundaries such as at the base of a container or internally when some part of the soil dries faster than the other in nonuniform drying. When the soil is restrained against free shrinkage, tensile stresses can develop within soil. Initiation of shrinkage cracking happens when the tensile stress developed within restrained soil exceeds the soil tensile strength.

Generally soil features comprise two broad categories of cracking referred to as orthogonal or non-orthogonal cracking. Orthogonal cracking occurs when soil cracks develop sequentially with subsequent cracks meeting already formed cracks orthogonally due to stress relief. In contrast, non-orthogonal cracks such as hexagonal formations occur when soils tend to crack simultaneously maximizing strain energy dissipation. Nonetheless, orthogonal formation is the most common, but in some cases, combinations of these crack forms could be observed.

Desiccation, Fig. 1 Small-scale desiccation cracking

Desiccation, Fig. 2 Large-scale desiccation cracking

Desiccation has several implications for engineering geology in formations with specific landform features such as surface crusting and mass wasting due to slope instability and development of wavy ground surfaces known as gilgai, influenced by desiccation cracks (Kodikara et al. 2002). In the field, desiccation cracks vary significantly in depth and spacing from tens of millimeters (Fig. 1) to tens of meters (Fig. 2). Desiccation can also lead to soil structure development including soil particle micro and macroaggregation and soil structure stabilization following repeated wet-dry cycles influencing drainage, volume changes in clay, consolidation, pore water pressure, slaking, residual shear, and tensile strength and so on.

Cross-References

▶ Atterberg Limits
▶ Characterization of Soils
▶ Clay
▶ Climate Change
▶ Collapsible Soils
▶ Compaction
▶ Consolidation
▶ Desert Environments
▶ Dewatering
▶ Fluid Withdrawal
▶ Hydrology

▶ Liquid Limit
▶ Noncohesive Soils
▶ Piezometer
▶ Plastic Limit
▶ Pore Pressure
▶ Saturation
▶ Shear Stress
▶ Soil Properties
▶ Water

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Designing Site Investigations

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Synonyms

Geotechnical investigation; Site assessment; Site characterization; Subsurface investigation

Definition

A site investigation is a planned field and office exercise used to obtain new information or verify existing data to support the design of a built structure, excavation, or site improvement. It may include collecting surface and/or subsurface information and be located on land, underwater, or a combination of both.

Introduction

The design of a site investigation generally follows an iterative process whereby basic or broad-based data are successively modified or supplemented by newer or more focused studies. The complexity of the site investigation is directly related to both the variability of the site conditions and the natural compatibility of the site to the planned improvement. Some complex sites occur in remote and often harsh environments and require specialized equipment. An example of this is an elevated jack-up drilling rig used for shallow water exploration in the Persian Gulf, as shown in Fig. 1.

A site investigation may have a variety of purposes such as verifying or supplementing an earlier investigation, complying with required investigations stipulated by a regulatory institution, or re-characterizing a site if new information becomes available. Established guidelines are available depending on the location and complexity of the site investigation (USACE 2001). In the United Kingdom, guidance on legal, environmental, and technical matters relating to site investigation is provided in BS 5930:2015 (BSI 2015).

Designing Site Investigations, Fig. 1 Nearshore jack-up rig, United Arab Emirates

Investigations of underground facilities (e.g., tunnels, caverns, repositories) worldwide can be planned using guidance from documents such as NRC (1984).

Site Planning

Initial planning for site investigations may be to evaluate site feasibility. For example, two coastal sites may be candidates to support the development of a marina or boatyard. One site might have level ground near a deep water embayment outside of the tidal zone but with no infrastructure; the other may have roads and utilities but may require more frequent dredging or site maintenance. The scope of the site feasibility may not involve subsurface investigations but instead may be accomplished using office research and a field reconnaissance only. For environmental site assessments, practitioners in the USA follow the American Society for Testing and Materials (ASTM) standard for phase I studies (ASTM E1527 – 13 2013).

Screening level site investigations may include a minimal amount of subsurface drilling work requiring mobilization of equipment and crews and obtaining the necessary permits. One purpose of a screening level investigation would be to determine the size or location of a facility to be built and to confirm the subsurface conditions, that is, depth to bedrock or soil profile for input to a calculation of seismic hazard. For environmental site assessments, this would include phase II studies following the standard in ASTM E1903-11 (2011).

Preliminary or final site investigations, other than the most simple, usually involve specialists or teams of specialists with varied technical backgrounds. They would include, in addition, engineering geologists, geotechnical engineers, seismologists, hydrogeologists, geophysicists, wildlife biologists, and civil engineers. These professionals are usually complimented by drillers, surveyors, and other licensing or planning personnel to plan, budget, and perform the work of a site investigation.

All site investigations require an evaluation of the potential safety risks to personnel and the public. It is best to determine what the risks are before mobilizing to the field and to develop a Health and Safety Plan (HASP) that properly identifies the hazards and how they can be mitigated.

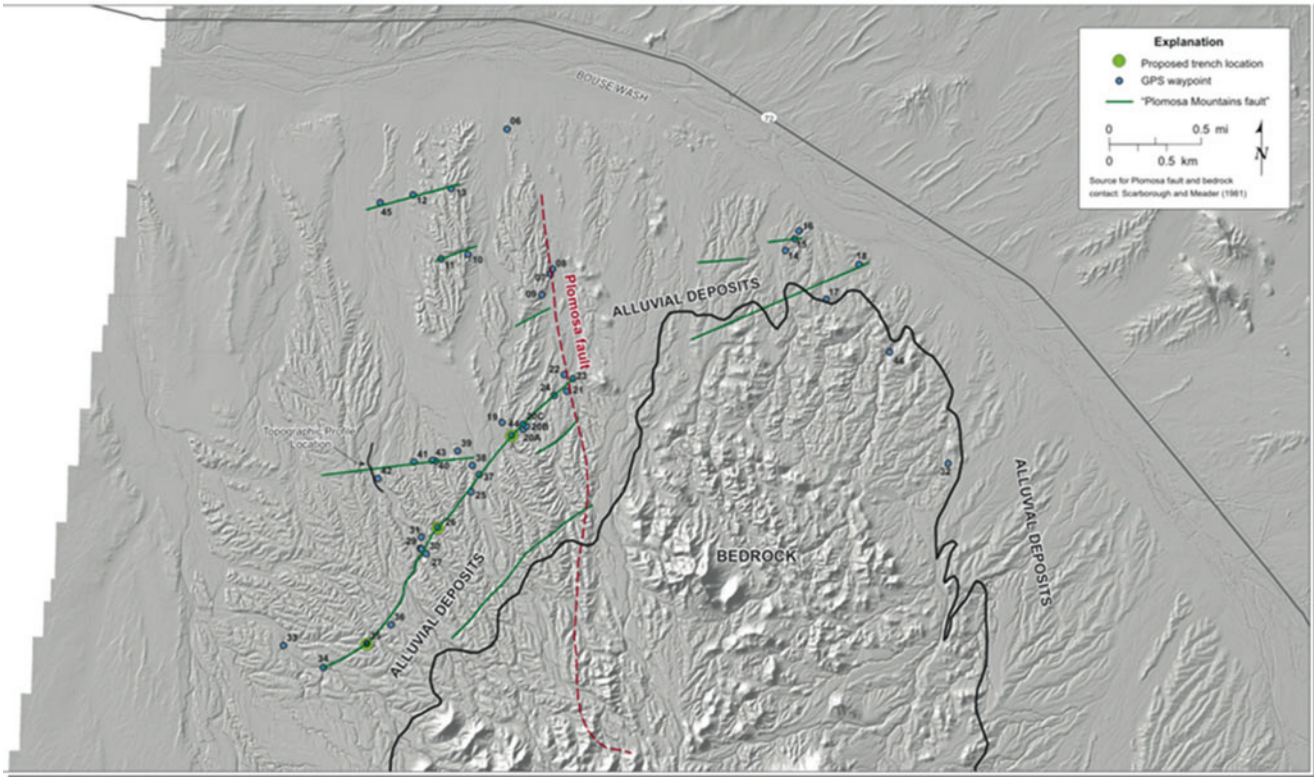
Office Research

Before mobilizing to the field, a site investigation will benefit from integration of pre-existing reports, data, and maps in order to develop a conceptual model of the site and its potential impact from an intended development, such as a built structure, environmental remediation, or mineral or water extraction. The following steps generally are followed before field investigation.

Reference Review – Governmental agencies publish technical papers, studies, and maps of study areas which are available in printed or digital form of a particular area. Consultant reports including boring logs, cross sections, and geologic mapping for a specific project are available with permission. University theses or dissertations provide technical sources of useful data for site investigations. Compiling a reference list or bibliography of these sources is essential for future report preparation. Scanning maps or imagery from these sources for inclusion into a geographical information system (GIS) is useful, provided the source is correctly referenced and/or permission is provided. Obtaining source imagery such as shape files for GIS is optimal for creating new figures and conducting queries and analysis.

Remote Sensing – Both government and private companies employ different airborne and satellite platforms to collect data from the surface of the earth. Multispectral data, digital spot imagery, Light Detection and Ranging (LiDAR), and interferometric synthetic aperture radar (INSAR) are examples of remotely sensed data sources. One advantage of collecting remotely sensed LiDAR data is to provide a base map for plotting field observations in areas beneath vegetative cover. INSAR and derivations of that method are useful in change detection such as geologic subsidence features. Remotely sensed data create representations of the Earth's surface that can be manipulated in GIS. The aerial coverage of the study area depends on the specific area of study, for instance, elongated corridors for highways or pipelines and broad polygonal shapes for power plants, wetland restoration, etc. Fig. 2 provides an example of fault hazard information plotted on a shaded relief surface derived from LiDAR.

Site Model Development – A site model, even in its simplest form, may benefit from compiling data into a GIS, a type of relational database that links spatial attribute data (water bodies, roads, topography, census data, climate, etc.) to established coordinate systems and topology. This is particularly important in areas of sinkholes or karst. The GIS can be used to create an initial model of a site by building data layers of topography, soil, bedrock, faults, hydrology, land use, roads, etc. Attribute links to borehole data, water well levels, ownership records, earthquake ground motions, and the like can be built into the model. The model can be queried, for example, to find out distances between features such as buildings and faults, buffers from sensitive areas to the intended development, and temporal data such as rainfall and runoff over certain time periods.



Designing Site Investigations, Fig. 2 Fault mapping, Plomosa Mountains, Arizona, USA

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Surface Exploration Methodology

Surface exploration includes methods that can gather information about the Earth's surface with little surface disturbance. These include airborne reconnaissance with helicopters or fixed wing aircraft, geologic field mapping, and selected geophysical methods. Surface methods are probably the most practical means of identifying existing slope instability, including the limits of landslides.

A HASP should be prepared that addresses exposure of personnel to equipment, biological or environmental hazards, how they can be mitigated, and where and how treatment can be obtained to treat injuries.

The following are typical surface methods:

Site Reconnaissance and Geologic Mapping – Designing a geologic mapping and site reconnaissance program is critical, especially when the site is remote, access is limited, or weather conditions are not ideal. A reliable reference for water resource investigations which is also useful for many other applications is the *Engineering Geology Field Manual*, published by the US Bureau of Reclamation (USBR 1998). In addition, Turner and Schuster (1996) provide an excellent approach to landslide investigations for highways in the USA. Key issues to resolve before heading to the field to conduct mapping include preparation of base maps, establishment of the proper mapping scale, identifying a team with a minimum of two people for safety reasons, geologic nomenclature, and checking for spatial clarity and geo-reference of geologic features. New technological advances now allow mapping using pen or tablet computers which allow multi-scale coverages, downloading of digital base maps from the GIS, and uploading of maps from the field for quicker use and safe keeping.

Geophysics – The use of surface geophysical surveys is ideal as a screening level tool to obtain nonintrusive imagery of subsurface conditions for little relative costs when compared to drilling or excavations. It also allows interpolation between future subsurface exploration points, such as boreholes. The most common surface geophysical methods include seismic refraction and reflection (including interferometric multichannel analysis of surface waves, IMASW), resistivity, magnetic, and gravity. These methods are described in detail in a publication from the Society of Exploration Geophysicists (SEG 2005). Geophysical seismic reflection has advanced substantially in both data collection and data processing to provide 3D, high-resolution imaging capability. Vibratory energy sources allow for geophysical data collection in sensitive environments such as coastal bluffs near operating nuclear power plants, as shown in Fig. 3.

Designing Site Investigations, Fig. 3 Minvibe seismic survey, Avila Beach, California, USA

In areas of karst, the use of multiple geophysical methods is a key objective as sinkhole development may not manifest itself at the ground surface.

Site Model Refinement – Continuing with the use of GIS, a subsurface exploration plan and work plan can be developed that takes into account the new geologic mapping and geophysical surveys and the location, depth, and details of subsurface exploration. In karst, the mechanisms of limestone solution and the defects produced by those processes require diligence, as described in Sowers (1996).

Preliminary geologic profiles can be created that allow the engineering geologist to recommend the preferred depths and quantity of boreholes or test pits and trenches to characterize the site. For example, maximum spacing of boreholes along a linear alignment might be 1,000 ft (300m.) on center for feasibility level studies but much closer for final design if conditions such as high groundwater or deep saprolite warrant it. The model might suggest inclined or higher density of borings in karst terrain to intercept irregularly shaped cavities.

Subsurface Exploration Methodology

From a health and safety point of view, the highest hazard exposure involves using heavy equipment or blasting to penetrate or expose geological features in the earth. Amending the HASP to address these hazards using job hazard analyses (JHA) is necessary to avoid injury or death. For example, extraction of water or solids at hazardous waste sites increases exposure of personnel to chemicals from drilling. Excavations into soil and rock increase slipping, tripping, and caving exposure to field geologists, as summarized below.

Borehole and Trenching Exploration – Drilling boreholes into soil or rock allows the engineering geologist to log the stratigraphy of the geologic materials retrieved for classification and for later laboratory index or specialized testing. Choosing the correct drilling method requires experience with drilling tools and familiarity with the ground conditions described in the earlier site model studies. Typical drilling methods include rotary wash, air rotary, hollow-stem auger, sonic, and cable tool.

Investigation of landslides may require different subsurface methods to identify failure surfaces based on depth, such as large-diameter boreholes (deep) and test pits (shallow). If the project appears stable but will include future deep, high cuts, obtaining samples for direct shear or other strength tests will provide a basis for the design of restraint systems or recommended slope inclinations.

Environmental site investigations also require careful sample collection, packaging, and in particular preservation. Having properly trained personnel in the collection of these

Designing Site Investigations, Fig. 4 Drill rig, Baker Beach, San Francisco, California, USA

Designing Site Investigations, Fig. 5 Fault trench, Manila, Philippines

samples is a key step in having proper laboratory testing, as shown in Fig. 4. Investigations for hazardous waste require preparing work plans, HASP, sample, and collection plans.

If the office research, field mapping/reconnaissance, and surface geophysical studies suggest that characterizing fault rupture risk requires excavating fault trenches at a site, then a fault rupture study should be initiated. Not all fault investigations include trenching as the soil horizon of interest may be either too deep or in a location (i.e., urban area) that precludes open excavation methods. In these situations, a combination of continuous coring and cone penetrometer testing (CPT) along a profile can provide stratigraphic interpretations. Ideally, trenches are key to determining recurrence intervals and slip rate and obtaining samples for absolute age dating. Figure 5 shows a fault trench for an investigation in Greater Manila, Philippines.

Sample Collection and Age Dating – Sample collection planning is challenging in that it involves mobilizing specialized equipment and personnel to the site to extract soil, rock, and water from the earth under sometime challenging environments and preserving the samples for future laboratory testing. The most challenging part of performing this collection is at a site with no previous investigation.

Soil and rock samples generally fall under two basic types: disturbed and undisturbed. Disturbed samples include those extracted from cuttings, drive samples, and block samples. Undisturbed samples can be obtained using rotary wash drilling coupled with sampling tubes (e.g., Pitcher barrel, fixed piston corer, Shelby and Denison barrel).

Groundwater samples may be extracted from either open-pipe piezometers or from discrete intervals using bailers and vacuum technology. Special modifications to the CPT tool allow *in situ* water sampling.

Environmental samples of soil and water may contain chemicals of concern (e.g., petroleum hydrocarbons, volatile organic compounds, heavy metals, BTEX, etc.) that require special handling and preservation. Duplicate, blank, and other additional samples are needed to provide quality control of samples where concentrations are measured to the parts per billion or smaller.

Seismic hazard analysis, an important part of site investigations in regions of elevated seismicity, demands an understanding of the frequency and age of earthquake events. Knowing the relative and absolute age and sense of movement of offset geologic units helps engineering geologists calculate the recurrence intervals and slip rates of damaging earthquakes. Noller et al. (2000) provide a comprehensive summary of age-dating techniques using laboratory analysis and observational methods.

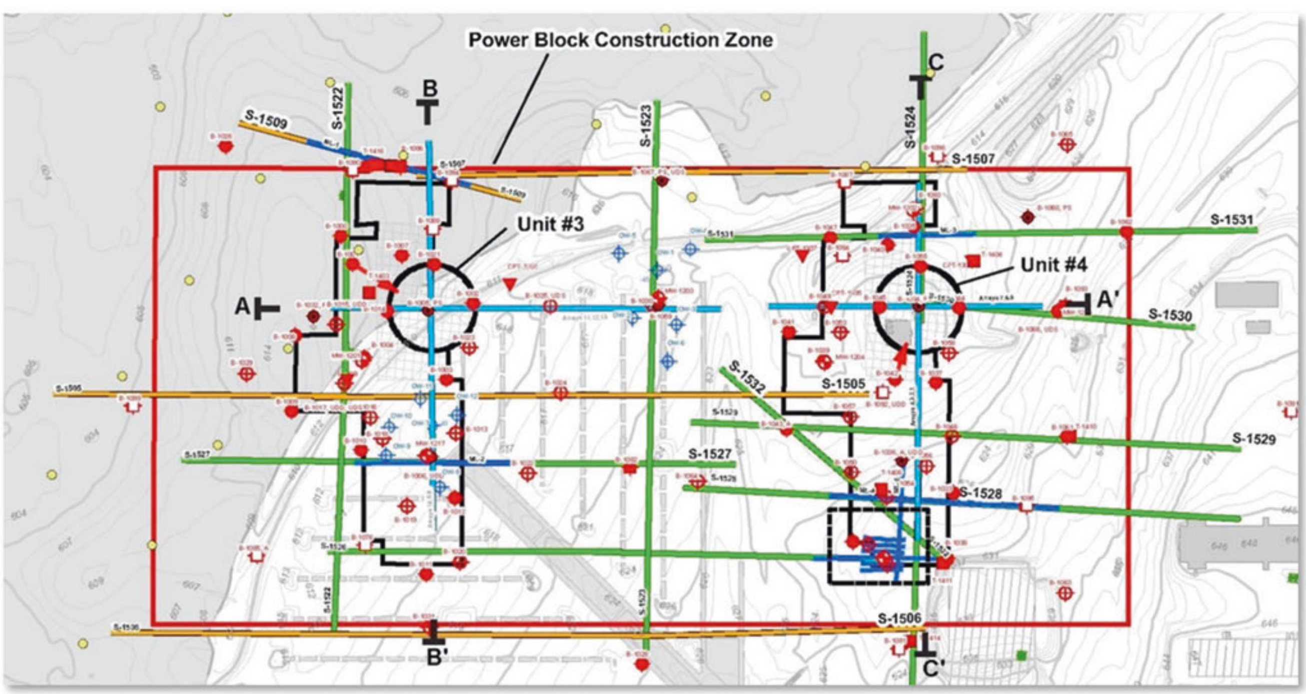
Borehole *In situ* Testing and Geophysical Surveys – There is an advantage to acquiring *in situ* properties of sensitive materials such as soft or swelling clay, collapsible silt and sand, and organic soils versus sample testing in the laboratory. Sample deterioration, volumetric change after retrieval, desiccation, and general disturbance are the primary reasons for using *in situ* borehole testing. Methods are available for determining elastic modulus and Poisson's ratio including pressuremeter (soil and soft rock) and Goodman Jack (hard rock) from boreholes. Elastic modulus can be determined from other non-borehole methods including flat jack tests, radial jacking, and pressure chamber, all of which utilize underground openings in rock. Groundwater packer testing is an *in situ* method for calculating hydraulic conductivity (K) typically in uncased rock formations, whereas falling or constant head permeability tests are used to measure K in cased or uncased boreholes in soil. CPT push technology is considered an *in situ* method and can obtain data such as tip resistance and skin friction that can be correlated to construct relatively accurate lithologic logs, in addition to shear wave measurements with tool modification.

When planning borehole geophysical surveys, care should be taken into account for borehole wall instability, possibly impacted by *in situ* testing. Borehole geophysical testing can include primary (P) and secondary (S) wave velocity determinations, either via the downhole or crosshole method (utilizing cased boreholes) or the P-S suspension logging method. Methods used to obtain continuous stratigraphic logs for lithologic interpretation include natural (N) gamma, induction logs, temperature, and flow logs. Density logging requires use of downhole radioactive source (gamma-gamma) element.

Borehole investigation and *in situ* testing in karst terrain need to account for lateral variability and material filling. For example, bedrock solutioning in the Appalachian mountain area of the USA might have softer clayey soil filling of voids, whereas the Florida panhandle can have more variable shell hash and coralline void fill. Although the risk of sinkhole development is similar, they may require different approaches for mitigation of built structures.

Borehole Monitoring and Instrumentation – When planning site investigations, sometimes temporal monitoring data is needed after the initial borehole data is collected or if future site disturbance from construction is a concern. Planning for changes in site behavior, the parameters to be monitored, and the anticipation of the magnitude of change are important. Key aspects for instrumentation monitoring include sensitivity of the instruments, location, procedures for measurement (manual or remote), and repair and maintenance.

Boreholes initially drilled for sample collection and *in situ* testing can be completed with groundwater wells to allow measuring changes in water levels or samples for geochemical analysis. In urban areas or where underground construction will occur, baseline elevation measurements may need to be acquired to compare with settlement measurements from extensometers or embedded load cells. Measurements of temperature, especially in Arctic sites, can utilize borehole



Designing Site Investigations, Fig. 6 Exploration plan, nuclear power plant, Alabama, USA thermistors or transducers. Measuring stress changes in soil and rock can utilize earth pressure cells and inclusion cells, respectively.

Laboratory Assignments – Laboratory testing is required in site investigations to determine the concentrations of chemicals of concern in environmental characterization and the range of material properties in geotechnical practice.

Methods (primarily ASTM) for testing soil and rock in nuclear power plant site investigations are detailed in appendices contained in USNRC (2014). These methods are reliant on the sampling procedures, preservation, and handling methods to assure high-quality results.

Site Model Refinement and Parameter Development – On more complex, critical facilities sites, such as a hospital, refinery, or power plant, a site model will need refinement or more detailed investigation following screening-type studies. In the USA, nuclear power plants require multiple investigative methods in increasingly dense configurations to ensure the risk of settlement, collapse, or deformation from geologic phenomena is thoroughly understood. Figure 6 provides an exploration plan that shows seismic refraction, downhole and resistivity lines, vertical and inclined borings, and multitude of *in situ* testing for siting a twin-unit power plant in northern Alabama.

Summary

The level of effort to design a site investigation depends on the complexity of the site, the interaction between the site, and the built structure and the regulatory environment. The general approach to designing the site investigation includes a process of office research, site reconnaissance, and model development using a GIS. This is followed by intrusive subsurface investigation and laboratory test methods that provide data to modify the site model. Sufficient guidance is available that provides procedures to obtain geologic, geophysical, and geotechnical data.

Cross-References

► Aerial Photography
► Borehole Investigations
► Brownfield Sites
► Characterization of Soils
► Engineering Geomorphological Mapping
► Excavation
► Geophysical Methods
► GIS
► Karst
► Land Use
► Marine Environments
► Remote Sensing
► Risk Assessment
► Subsurface Exploration
► Waste Management

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Deviatoric Stress

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Definition

Deviatoric stress is the difference between the stress tensor $\boldsymbol{\sigma}$ and hydrostatic pressure tensor $\boldsymbol{p}$ acting on the rock or soil mass.

Context

Stress that causes a change in volume of a rock or soil reference cube without also causing a change in shape is called hydrostatic pressure, because it acts equally in all directions; thus, hydrostatic pressure is a normal stress. Stress produced by tectonic forces, external loads, and excavations that may remove earth materials which provide support for adjacent earth material differs from the hydrostatic stress and can cause deformations and changes in shape. The reference cube under purely hydrostatic stress conditions need not be rotated to an orientation in which the shear stresses reduce in magnitude to zero and the normal stresses become principal stresses because the hydrostatic pressure tensor consists of only normal stresses. Thus, the hydrostatic pressure $\boldsymbol{p}$ can be subtracted from the normal stresses in the stress tensor, resulting in the deviatoric stress tensor $\boldsymbol{s}$.

$$\begin{aligned} \boldsymbol{s} &= \boldsymbol{\sigma} - \boldsymbol{p} = \begin{bmatrix} \sigma_{xx} & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_{yy} & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_{zz} \end{bmatrix} - \begin{bmatrix} p & 0 & 0 \\ 0 & p & 0 \\ 0 & 0 & p \end{bmatrix} \\ &= \begin{bmatrix} \sigma_{xx} - p & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_{yy} - p & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_{zz} - p \end{bmatrix} \end{aligned} \tag{1}$$

The simplest example of deviatoric stress is provided by the laboratory uniaxial or unconfined compression test on a rock core sample. A properly prepared sample is placed in the testing machine and the axial load is applied; the applied load is recorded during the test and the maximum load at the time the core sample breaks is divided by the cross-sectional area of the core sample to produce the diameter of the Mohr circle of stress, which is twice the deviatoric stress. Because the applied hydrostatic pressure confining the sample is zero, subtraction is trivial. The next simplest example of deviatoric stress is provided by the laboratory triaxial compression test of a rock core sample. In this test, the properly prepared sample is placed in the testing machine, the test chamber filled with deaired water or oil is pressurized to the desired confining pressure, and the axial load is applied. The maximum load at the time the core sample breaks is recorded. The confining pressure is taken to be the intermediate and minor principal stresses (σ_2 and σ_3, respectively; $\sigma_2 = \sigma_3$), whereas the axial load divided by the sample cross-sectional area is the maximum principal stress (σ_1). Further discussion of this topic is available online (Eberardt 2009; Rock Mechanics for Engineers 2016). Deviatoric stress is $(\sigma_1 - \sigma_3)/2$, which is the radius of the Mohr circle of stress and the magnitude of the maximum shear stress on the Mohr circle that corresponds to mean normal stress $(\sigma_1 + \sigma_3)/2$. Triaxial test stresses may be evaluated algebraically rather than as tensor quantities because triaxial compression tests are set up effectively with the Cartesian coordinate system axes oriented with the major principal stress direction axial to the core sample and the intermediate and minor principal stress directions perpendicular to the core sample axis.

Cross-References

▶ Bulk Modulus
▶ Effective Stress
▶ Hooke's Law
▶ Modulus of Deformation
▶ Modulus of Elasticity
▶ Mohr Circle
▶ Mohr-Coulomb Failure Envelope
▶ Normal Stress
▶ Poisson's Ratio
▶ Pressure
▶ Rock Mechanics
▶ Shear Modulus
▶ Shear Strength
▶ Shear Stress
▶ Soil Mechanics
▶ Stress
▶ Young's Modulus

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Dewatering

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Synonyms

Groundwater control; Groundwater lowering

Definition

The process of lowering groundwater by pumping or installing cut-off walls to prevent ingress of water into excavations or tunnels.

Construction and mining projects often require excavations below groundwater level in soils and rocks. Where groundwater is encountered during excavation, problems can occur either by flooding of the excavation or in the form of instability induced by its presence.

Depending on the nature of the ground being excavated, the groundwater conditions encountered can vary greatly from site to site. A thorough hydrogeological investigation may be needed to allow groundwater conditions to be defined (Younger 2007).

Excavations below groundwater level often encounter problems, including flooding and instability, caused by groundwater inflows and pressures. Dewatering is used to allow excavations for construction, mining, and engineering purposes to be formed in workably dry and stable conditions. There are two principal approaches to dewatering: dewatering by pumping, where an array of wells or sumps is pumped to lower groundwater levels, and dewatering by groundwater exclusion, which relies on low permeability cut-off walls or ground treatment barriers to prevent or reduce groundwater inflows.

Objectives of Dewatering

There are two principal objectives for dewatering. The first is to prevent excavations below groundwater level from being inundated by groundwater. The second (and often more important) objective is to avoid groundwater-induced instability of the excavation by controlling pore water pressures and hence effective stresses around the excavation.

The importance of controlling pore water pressures for excavations in soils can be illustrated by Terzaghi's equation of effective stress (Powrie 2014). Soil behavior is controlled by the effective stress σ', which is related to total stress σ (due to external loads) and the pore water pressure u by:

$$\sigma' = \sigma - u \tag{1}$$

The shear strength τ_f of a soil depends on the normal effective stress, according to the Mohr–Coulomb failure criterion:

$$\tau_f = \sigma' \tan \phi' \tag{2}$$

where ϕ' is the effective angle of soil friction.

Dewatering reduces pore water pressure u at constant total stress σ, this increases the normal effective stress σ' and thereby enhances the ability of the soil or rock to resist shear, thus improving stability of slopes and excavation formation level. Conversely, the positive pore water pressures associated with seepage into the excavation have a destabilizing effect and can result in slumping of side slopes and hydraulic failure of the base. Such instability can be avoided by the use of a suitable dewatering system.

Dewatering Methods

Dewatering methods to control groundwater are categorized into two groups:

(i) Pumping methods where groundwater is pumped from an array of wells or sumps located in or around the excavation to temporarily lower groundwater levels (Fig. 1a).
(ii) Exclusion methods that use low permeability cut-off walls to exclude groundwater from the excavation (Fig. 1b).

Pumping and exclusion methods may be used in combination. Detailed guidance on dewatering methods can be found in Cashman and Preene (2012) and Powers et al. (2007).

Control of Surface Water

Even where a dewatering system is deployed to deal with groundwater, there will usually be a requirement to control surface water in an excavation. Surface water can come from a variety of sources, including rainfall, direct seepage from nearby rivers or lakes, leaking sewers and water mains or the construction operations themselves. Any excavation, including those above the water table, should have a system for surface water control, typically consisting of sumps, drainage channels, and French drains to collect the water, and sump pumps to remove the water.

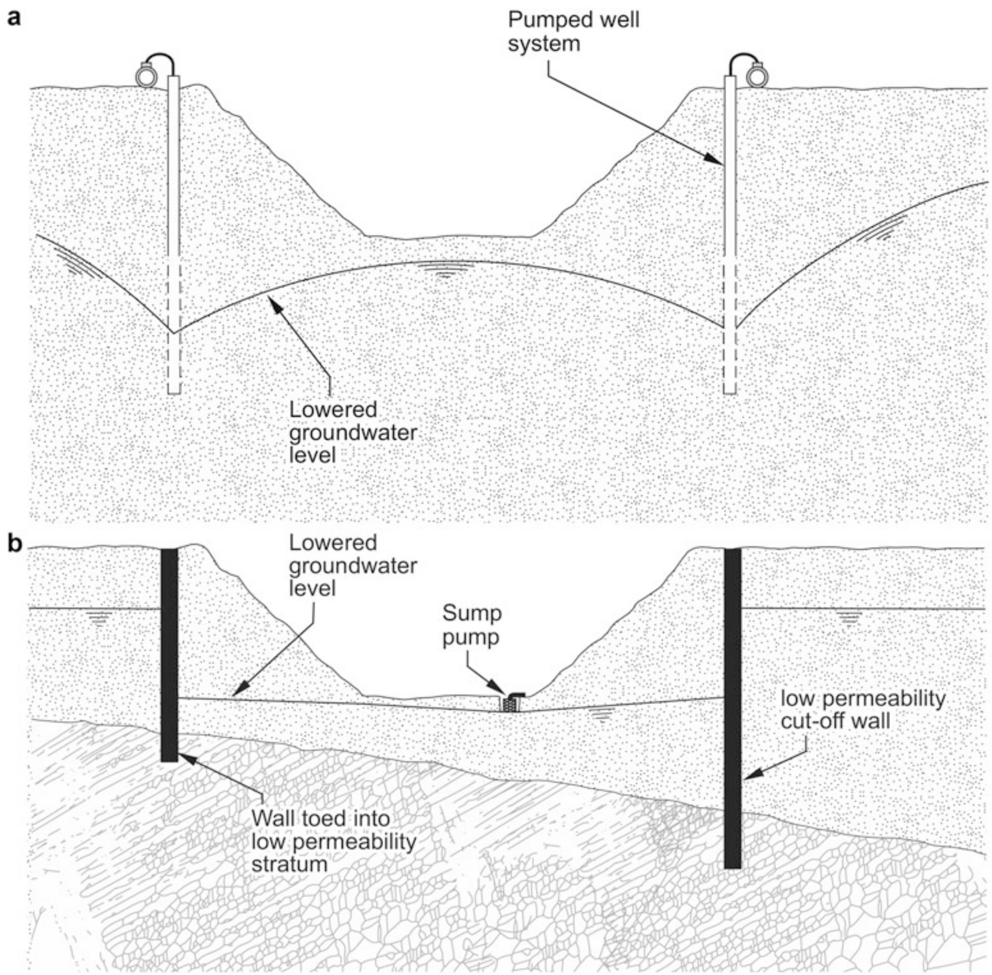
Dewatering by Pumping

Groundwater control by pumping (Fig. 1a) involves pumping groundwater from an array of wells or sumps to lower groundwater levels in and around the excavation. The amount of lowering of the groundwater level is known as drawdown.

Table 1 lists the various pumped well groundwater control methods available. However, the vast majority of projects are carried out using just four main conventional dewatering techniques: sump pumping, wellpoints, deep wells, and ejector wells.

Each of the main methods has a specific range of application where the method is most likely to be effective. Fig. 2 defines the range of application relative to two key parameters: drawdown required and soil permeability (hydraulic conductivity).

An important distinction within the groundwater pumping methods is between open pumping and predrainage methods.



Dewatering, Fig. 1 Categories of dewatering methods for excavations. (**a**) Groundwater control by pumping (Groundwater is pumped from lines or arrays of wells or sumps located in or around the excavation to lower groundwater level below the base of the excavation.) (**b**) Groundwater control by exclusion (Low permeability cut-off walls are used to form a barrier around the excavation. In combination with naturally occurring low permeability strata (e.g., clays or unfissured mudstone) the walls form a barrier to groundwater flow, and effectively exclude groundwater from the excavation. The water trapped in the soil pores or rock fissures within the area enclosed by the cut-off wall is typically pumped away by sump pumping during excavation

- Open pumping, most commonly carried out by sump pumping, involves allowing groundwater to seep into the excavation, from where it is removed by pumping. While simple in practice, open pumping has the disadvantage that groundwater levels cannot be lowered in advance of excavation. Open pumping typically requires water to enter the excavation before it can be pumped away, and localized instability of the excavation may result as the result of seepage forces where water enters the excavation.
- Pre-drainage methods (which include wellpoints, deep wells, and ejector wells) work on the principle that groundwater levels can be lowered in advance of excavation works. This group of methods has the advantage that groundwater can be managed so that water does not enter the excavation, reducing the risk of groundwater-induced instability.

Dewatering by Groundwater Exclusion

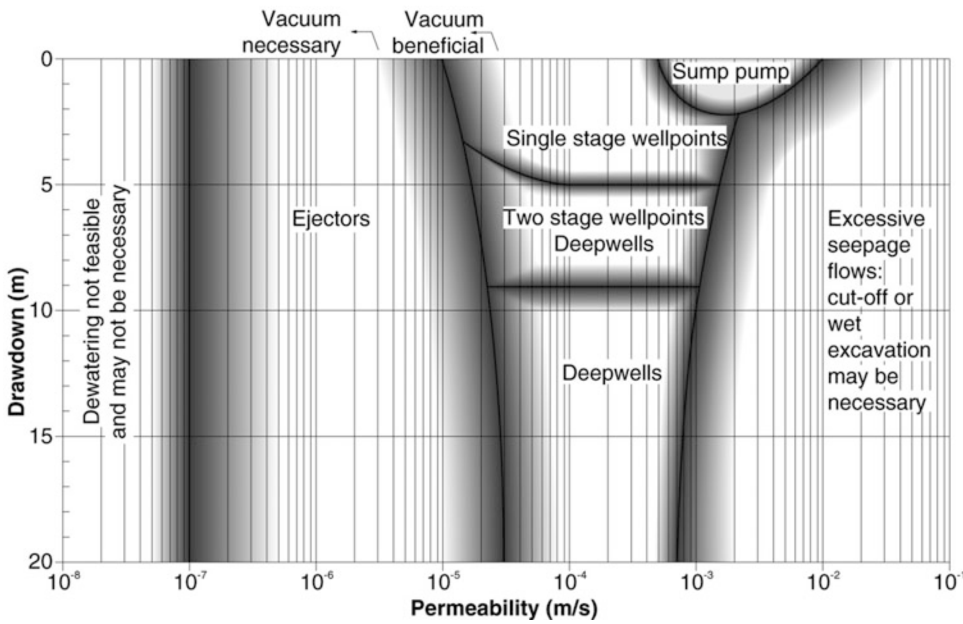
This group of methods involves installing a very low permeability physical cut-off wall or cofferdam around the excavation to exclude groundwater.

If an impermeable stratum exists at shallow depth beneath the excavation, then the cut-off wall may be able to toe into that stratum to create a full cut-off (Fig. 1b). The only groundwater pumping requirement will be required to deal with:

- Groundwater trapped within the area enclosed by the cut-off wall;
- Rainfall and precipitation;
- Seepage through the wall and through the ground.

Dewatering, Table 1 Dewatering methods by groundwater pumping (A range of different groundwater pumping techniques can be used for dewatering; each has its own characteristics and suitability for application. The most commonly used techniques are: sump pumping, wellpoints, deep wells, and ejector wells)

Method	Typical applications	Description
Drainage pipes or ditches (e.g., French drains)	Control of surface water run-off and shallow groundwater (including perched water and residual seepages into excavation)	Pipes, ditches, and trenches to divert or remove surface water from the working area. May obstruct construction traffic, and will not control groundwater at depth. Unlikely to be effective in reducing pore water pressures in fine-grained soils
Sump pumping	Shallow excavations in clean coarse-grained soils or stable rock for control of groundwater and surface water	Water is collected in pits or low points (sumps) within the excavation, from where it is pumped away. May not give sufficient drawdown to prevent seepage from emerging on the cut face of a slope, possibly leading to loss of fines and instability. May generate silt or sediment laden discharge water, causing environmental problems
Wellpoints	Generally shallow, open excavations in sandy gravel down to fine sand and possibly silty sand. Deeper excavations (requiring >5–6 m drawdown) will require multiple stages of wellpoints to be installed	Lines or rings of closely spaced small diameter wells installed around an excavation and pumped by a suction system. Quick and easy to install in sand. Suitable for progressive trench excavations. Difficult to install in ground containing cobbles or boulders. Maximum drawdown is ~ 5–6 m for a single stage in sandy gravel and fine sand, but may only be ~ 4 m in silty sand
Horizontal wellpoints (machine laid)	Generally shallow trench or pipeline excavations or large open excavations in sand and possibly silty sand	Horizontal drainage pipe, laid by specialist trenching machines, pumped by suction pumps. Suitable for long runs of trench excavations outside urban areas, where very rapid installation is possible
Deep wells with electric submersible pumps	Deep excavations in sandy gravels to fine sand and water-bearing fissured rocks	Slimline borehole submersible pumps installed in bored wells. No limit on drawdown in appropriate hydrogeological conditions. Installation costs of wells are significant, but fewer wells may be required compared with most other methods. Close control can be exercised over well screen and filter
Deep wells with electric submersible pumps and vacuum	Deep excavations in silty fine sand, where drainage from the soil into the well may be slow	Slimline borehole submersible pumps installed in bored wells with a separate vacuum system used to apply vacuum to the wells. Number of wells may be dictated by the requirement to achieve an adequate drawdown between wells, rather than the flow rate, and an ejector system may be more economical
Shallow bored wells with suction pumps	Shallow excavations in sandy gravel to silty fine sand and water-bearing fissured rocks	Bored wells pumped by surface suction pumps. Particularly suitable for coarse, high permeability materials where flow rates are likely to be high. Useful where correct filtering is important as closer control can be exercised over the well filter than with wellpoints. Drawdowns limited to ~4–7 m depending on soil conditions
Ejector wells	Excavations in silty fine sand, silt or laminated or fissured clay in which pore water pressure control is required	Low capacity wells pumped by a nozzle and venturi system. Drawdowns generally limited to 20–50 m depending on equipment. Low energy efficiency, but this is not a problem if flow rates are low. In sealed wells, a vacuum is applied to the soil, promoting drainage
Passive relief wells and sand drains	Relief of pore water pressure in confined aquifers or sand lenses below the floor of the excavation to ensure basal stability	Vertical boreholes used to create a vertical flowpath for water into the excavation; water must then be directed to a sump and pumped away
Collector wells	Deep excavations in relatively permeable soils such as sand and gravel, where surface access does not allow the installation of large numbers of wells	Sub-horizontal wells drilled radially outwards from a central shaft. Each collector well is expensive to install, but relatively few wells may produce large flow rates and be able to dewater large areas
Siphon drains	Long-term slope drainage and landslide stabilization in low permeability soils	A self priming siphon system installed in large diameter wells. Can allow passive drainage of slopes, without the need for pumping
Artificial recharge	Soils of high to moderate permeability and fissured rocks, where lowering of groundwater is to be controlled so that environmental impacts can be mitigated	Reinjection of pumped water back into the ground. Typically complex to operate and maintain. Recharge wells often suffer from clogging due to water chemistry effects and may require periodic backflushing and cleaning
Electro-osmosis	Very low permeability soils, e.g., clay, silt, and some peats	A system of anodes and cathodes used to promote groundwater flow in very low permeability materials. Only generally used for pore water pressure control or ground improvement when considered as an alternative to ground freezing. Installation and running costs are comparatively high



Dewatering, Fig. 2 Range of application of pumped well groundwater control techniques (Adapted from Roberts and Preene (1994), and modified after Cashman (1994)) (from Preene et al. 2016: reproduced by kind permission of CIRIA) (The range of application of pumped groundwater control methods are shown relative to two key parameters: drawdown (i.e., the vertical lowering of groundwater level) required and ground permeability. The shaded areas at the boundary between techniques represent zones where there is overlap between the capabilities of different methods)

If an impermeable stratum does not exist at a convenient depth, only a partial cut-off can be formed, where the cut-off walls exclude lateral groundwater flow from the sides, but groundwater can enter the excavation by flowing beneath the toe of the wall. A partial cut-off increases the seepage path length and reduces the flow rate compared to the case when there is no cut-off at all. The cut-off should be designed to be of adequate penetration to prevent piping failure of granular soils.

A wide range of methods can be used to exclude groundwater from excavations. Key attributes of more commonly used cut-off methods are described in Table 2.

Some cut-off methods are temporary. For example, the groundwater will thaw when artificial ground freezing is discontinued, or steel sheet piles can be extracted at the end of the job. These temporary methods should not have a significant effect on groundwater conditions at the site following the end of construction. However, methods which permanently affect soil permeability (e.g., grouting) can permanently alter groundwater flow regimes – it is essential that the potential impact of this is assessed at design stage.

Environmental Impacts of Dewatering

Groundwater control has the potential to have measurable effects (such as lowering of groundwater levels) at considerable distances (sometimes several hundred meters) from the dewatered excavation. The nature and extent of the environmental impacts are dependent on the hydrogeological setting and may need to be assessed at design stage so that any necessary mitigation measures can be identified.

Environmental impacts can result from groundwater control, even if pumping is not involved – for example, cut-off walls installed as part of groundwater exclusion schemes may act as underground dams and may cause local changes in groundwater levels.

Table 3 summarizes the range of potential impacts from groundwater control works.

Design of Dewatering Schemes

The design of dewatering systems must address hydrogeological factors in relation to the calculation of pumped flow rates, environmental impacts, etc., but must also address the performance and selection of suitable technologies for groundwater pumping and groundwater exclusion.

In all but the simplest of groundwater control problems, the design process should include the following steps:

1. Definition of problem and constraints;
2. Development of hydrogeological conceptual model;
3. Selection of method of groundwater control;
4. Design calculations;
5. Assessment of environmental impacts;
6. Review of design.

At the stage that the design is implemented in the field, it is important that the performance of the dewatering system is monitored (e.g., by recording pumped flow rates and lowered groundwater levels) so that the system performance can be validated against the design. If the system performance deviates significantly from the design values,

Dewatering, Table 2 Dewatering methods by groundwater exclusion (Several different methods are available to exclude groundwater from an excavation; each has its own characteristics and suitability for application. Displacement barriers involve driving the wall elements into the ground, displacing the soil; excavated barriers involve excavating the profile of the wall and backfilling to replace the soil or rock with lower permeability material; injection barriers involve injecting low permeability fluid (grout) to fill soil pores or rock fissures, to produce a zone of lower permeability treated ground)

Method	Typical applications	Description
Displacement barriers		
Steel sheet-piling	Open excavations in most soils, but obstructions such as boulders or timber baulks may impede installation	Sectional, interlocking steel sheets are driven, vibrated, or pushed into the ground. May be installed to form permanent cut-off, or used as temporary cut-off with piles removed at the end of construction. Can support the sides of the excavation with suitable propping. Seal may not be perfect, especially if obstructions present. Vibration and noise of driving may be unacceptable on some sites, but “silent” methods are available where piles are pressed into the ground by hydraulic jacks. Relatively cheap
Vibrated beam wall	Open excavations in silt and sand. Will not support the soil	A vibrating I beam is driven into the ground and then removed. As it is removed, grout is injected through nozzles at the toe of the pile to form a thin, low permeability membrane. Rapid installation. Relatively cheap, but costs increase greatly with depth
Excavated barriers		
Slurry trench wall using cement-bentonite or soil-bentonite	Open excavations in silt, sand, and gravel up to a permeability of about 5×10^{-3} m/s	A trench is excavated under bentonite slurry and is backfilled with a cement/bentonite or soil/bentonite mixture. The resulting trench forms a low permeability curtain wall around the excavation. Quickly installed and relatively cheap, but cost increases rapidly with depth
Concrete diaphragm walls	Side walls of excavations and shafts in most soils and weak rocks, but presence of boulders may cause problems	A trench is excavated under bentonite slurry and is backfilled with concrete (which displaces the bentonite). Can support the sides of the excavation and often forms the sidewalls of the finished construction. Can be keyed into rock. Minimum noise and vibration. High cost may make method uneconomical unless walls can be incorporated into permanent structure
Bored pile walls (secant and contiguous)	As concrete diaphragm walls, but penetration through boulders may be costly and difficult	Bored piles (formed from concrete) are installed in lines at close centers to form a continuous (for secant piles) or contiguous wall. Method has similar characteristics as concrete diaphragm walls, but more likely to be economic for temporary works use. Sealing between contiguous piles can be difficult, and additional grouting or sealing of joints may be necessary
Injection barriers		
Permeation and rock grouting using cement-based grouts	Tunnels and shafts in gravel and coarse sand, and fissured rocks	Fluid grout is injected from closely spaced boreholes. The grout fills the pore spaces in soil and fissures in rock, reducing the flow of water through the ground. Equipment is simple and can be used in confined spaces. A comparatively thick zone needs to be treated to ensure a continuous barrier is formed. Multiple stages of treatment may be needed
Permeation and rock grouting using chemical and solution grouts	Tunnels and shafts in medium sand (chemical grouts), fine sand and silt (resin grouts), and fissured rocks	Fluid grout is injected from closely spaced boreholes. The grout fills the pore spaces in soil and fissures in rock, reducing the flow of water through the ground. Method has similar characteristics as cement-based grouting, but materials (chemicals and resin) can be expensive. Silty soils are difficult and treatment may be incomplete, particularly if more permeable laminations or lenses are present
Jet grouting	Open excavations in most soils and very weak rocks	Down the hole jetting equipment is used in close spaced boreholes to form a series of overlapping columns of soil/grout mixture. Inclined holes possible. Can be messy and create large volumes of slurry. Risk of ground heave if not carried out with care. Relatively expensive
Mix-in-place walls	Open excavations in most soils and very weak rocks	Overlapping columns or panels of low permeability material are formed by *in situ* mixing of soil and injected grout. Columns formed using auger-based equipment, panels formed using cutter soil mixing (CSM) equipment. Produces little spoil. Less flexible than jet grouting. Relatively expensive
Other methods		
Artificial ground freezing using brine or liquid nitrogen	Tunnels and shafts. May not work if groundwater flow velocities are excessive (>2 m/day for brine or >20 m/day for liquid nitrogen)	Very low temperature refrigerant (brine or liquid nitrogen) is circulated through a line of closely spaced boreholes to lower ground temperatures. A “wall” of frozen ground (a freezewall) is formed, which can support the side of the excavation as well as excluding groundwater. Liquid nitrogen is expensive but quick; brine is cheaper but slower. Liquid nitrogen is to be preferred if groundwater velocities are relatively high. Plant costs are relatively high
Compressed air	Confined chambers such as tunnels, sealed shafts, and caissons	Increased air pressure (up to 3.5 bar) is applied to confined excavations (such as tunnels or shafts) to raise pore water pressure in the soil or rock around the chamber, reducing the hydraulic gradient and limiting groundwater inflow. Potential health hazards to workers. Air losses may be significant in high permeability soils. High running and setup costs

Dewatering, Table 3 Categories of environmental impacts from dewatering (based on Preene and Fisher 2015) (Classification of potential dewatering impacts in these categories can be useful to at design stage to allow design studies to focus on potential impacts to help identify sites and projects that may be significantly impacted)

Impact category	Type of impact	Possible scenarios where significant impacts may occur
Geotechnical	Ground settlement – effective stress	Increases in effective stress are caused by lowering of groundwater levels, resulting in compression and consolidation of the ground. Such settlements are an unavoidable consequence of lowering of groundwater levels, but in relatively stiff soils settlements are often too small to cause damage or distress to structures
Geotechnical	Ground settlement – loss of ground	Removal of fine particles from the ground (loss of fines) which can occur when poorly controlled sump pumping draws out fine-grained soil particles (clay, silt, and sand sized) with the pumped water. With good design and implementation, loss of fines (and the associated settlement risk) can be avoided
Contamination	Mobilization by pumping	Hydraulic gradients created by dewatering pumping will typically be much larger than natural gradients, and any nearby groundwater contamination may be mobilized and will tend to be drawn toward the pumping system
Contamination	Creation of vertical flow pathways	If dewatering wells or investigation boreholes do not have suitable grout or bentonite seals, they can act as vertical pathways and allow migration of contamination between strata
Water feature	Reduction in flow	Groundwater pumping near natural water-dependent features such as wetlands or groundwater springs can result in reduction in flow to those features Even if groundwater pumping is not planned to be significant, low permeability cut-off walls used as part of groundwater exclusion methods can also have impacts. Groundwater levels may rise on the upgradient side and fall on the downgradient side, which can affect flows to natural features
Water feature	Change in water quality	Water chemistry in natural features (e.g., wetlands or ponds) may change if dewatering systems affect the nearby flow regime
Water feature	Change in water level	Water levels in natural features (e.g., wetlands or ponds) may change if dewatering systems affect the nearby flow regime
Water resource	Change in water availability	Large-scale dewatering pumping may reduce available water resources, due to lowering of groundwater levels or reduction in yield of existing water supply wells and springs
Water resource	Change in water quality	Changes in groundwater flow regimes due to dewatering pumping or low permeability cut-off walls may affect water chemistry; for example, by drawing in saline water from coastal waters or drawing in poorer quality water from abandoned mine workings
Water discharge	Change in water quality	When dewatering water is discharged (to surface water or to groundwater, via recharge wells) the pumped water quality may be different to the receiving water environment. Differences in water temperature, water chemistry, and suspended solid load may affect the ecosystem or amenity value of the receiving waters
Water discharge	Downstream scour and flooding	Discharge of large flow rates of dewatering water into a surface watercourse may cause scour at the discharge location or may cause flooding downstream

consideration should be given to modifications to the dewatering system.

Further details of design methods are given in Powers et al. (2007), Preene et al. (2016), and Cashman and Preene (2012).

Selected Case Studies

A key element of successfully implementing a dewatering scheme is characterizing the hydrogeological regime to an acceptable level of detail, and then developing a scheme based on a dewatering technology that has capabilities suitable for conditions at the site. At the Teesdale Barrage in the UK, Leiper and Capps (1993) describe how the barrage substructure was formed in a large construction basin which, when pumped out by sump pumping, provided dry working conditions while the river flow was diverted to one side during the works. However, despite the basin providing dry working conditions, hydrogeological investigations identified a permeable sandstone stratum below the floor of the basin. The high piezometric level in this stratum meant that, when the basin was pumped dry, if the piezometric pressure was not lowered significantly, there would be a risk of heave of the base of the excavation. The solution adopted was to install a system of deep wells around the perimeter of the basin to lower groundwater levels and ensure that factors of safety against heave were acceptably high. Initially two wells were installed and test pumped (by constant rate pumping and recovery tests). Data from the pumping tests was used to finalize the dewatering system design of an array of 16 pumped deep wells around the basin.

Dewatering methods are divided into two main groups – pumping methods and exclusion methods (see Fig. 1). Often the choice of appropriate method is controlled by the hydrogeological conditions on site, rather than the depth and type of the excavations. Bickley and Judge (2015) describe three case studies of excavations of very similar size and depth that were dewatered by different approaches. One was dewatered by the pumping method, using deep wells. One was dewatered using the exclusion method, in the form of a concrete secant pile wall that sealed into a low permeability stratum below the base of the excavation. The third used a cut-off wall to exclude shallow groundwater, but also used dewatering pumping (by deep wells) to deal with groundwater in deeper strata. This set of case studies highlight that each dewatering system must be developed on a site-by-site basis, including the development of suitable geological and hydrogeological models to provide the best construction solution.

It is sometimes necessary to consider the potential environmental impacts from dewatering and, if necessary, to mitigate them. One example is for the construction of tunnel portals adjacent to the River Thames in the UK, as described by Roberts and Holmes (2011). The portals were located on the river flood plain. Excavations up to 18 m deep were required through up to 10 m of soft alluvial soils (clay and silt) overlying Terrace Gravels and Upper Chalk, with groundwater levels close to surface. Dewatering involved both a groundwater exclusion wall and pumping from deep wells. Adjacent to the north portal was a petroleum tank farm, which was assessed as having a high sensitivity to settlement. Numerical modeling showed that in the absence of mitigation measures drawdown in the Terrace Gravels below the tank farm was likely to be up to 4 m. This would have led to the underdrainage and consolidation of the soft alluvial soils above, hence generating unacceptable surface settlements. A mitigation scheme was developed to artificially recharge groundwater into the Terrace Gravels, via an array of recharge wells around the tank farm. The objective of the artificial recharge scheme was to limit drawdown of groundwater levels in the Terrace Gravels to no more than 0.5 m below the ambient tidal cyclic groundwater level, thereby significantly reducing the consolidation settlement of the overlying soft alluvial soils. Protocols for monitoring and control of groundwater were used to ensure that groundwater levels in key locations stayed within design limits. The scheme ensured that no significant surface settlement was recorded at the tank farm – this contrasts with settlements of up to 100 mm recorded in other areas that were not protected by the artificial recharge system.

Summary

Dewatering is used to control groundwater to allow below-ground construction, mining, and engineering projects to be carried out in dry and stable conditions. There are two principal approaches to dewatering: dewatering by pumping, where an array of wells or sumps is pumped to lower groundwater levels, and dewatering by groundwater exclusion, which relies on low permeability cut-off walls or ground treatment barriers to prevent or reduce groundwater inflows.

Cross-References

► Groundwater
► Hydrogeology
► Tunnels

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Diagenesis

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Definition

Diagenesis is the sum of all chemical, physical, and biological changes that occur to sedimentary materials after deposition but before lithification (conversion to sedimentary rocks). Diagenesis occurs at pressures and temperatures lower than those required for the formation of metamorphic rocks and can be broken down into early and late diagenesis, although some workers restrict the term solely to early diagenesis.

Characteristics

Early diagenesis occurs in the upper portions of sediments (upper ~1 m to several 100 m) where temperatures are less than ~50 °C. Sediment pore spaces are water-saturated, although in some cases gas (methane) bubbles may occur. During diagenesis, compaction with burial decreases sediment porosity from ~50% to 90–95% in surface sediments to 20% (or less) within 1–2 km of burial (Burdige 2006).

The lithogenic and biogenic materials initially deposited in sediments are involved in a number of early diagenetic reactions. In the latter case, this often occurs because they (i.e., biogenic silica and carbonates along with organic matter [OM]) are biogeochemically reactive once removed from their site of production (Aller 2014).

The oxidation, or remineralization, of OM in sediments is the direct or indirect causative agent for many early diagenetic reactions (Burdige 2006). Bacteria mediate much of this OM remineralization since sediments often become anoxic (i.e., devoid of oxygen) close to the sediment-water interface (generally <1 cm in coastal sediments to several cm's or greater in deep-sea sediments). Surface sediments (typically ~10 cm or less, but sometimes as deep as several meters) are also often colonized by benthic macrofauna such as burrowing clams and tube-dwelling polychaetes. The presence and activities of benthic macrofauna can have a profound effect on sediment diagenesis (Aller 2014).

Early diagenesis is, in general, also strongly intertwined with processes associated with OM preservation (Burdige 2007), which ultimately result (at pressures and temperatures much higher than those of early diagenesis) in the formation of kerogen and, in some cases, coal, oil, and natural gas (thermogenic methane) (Killops and Killops 2005). Note that natural gas also has a biogenic origin at low temperatures and pressures through methanogenesis, an OM remineralization process that occurs in organic-rich sediments.

In situ formation of authigenic phases (e.g., pyrite during bacterial sulfate reduction) is an important part of sediment diagenesis. These processes also include "reverse weathering" reactions that produce authigenic clays from reactants such as biogenic silica, lithogenic clays, iron oxides and/or basalt, and metastable volcanic debris (Aller 2014). These transformations occur under early as well as late diagenetic conditions. Biogenic silica may undergo other transformations during late diagenesis, producing more stable forms of silica such as chert. During late diagenesis, processes such as compaction and cementation are also involved in conversion of biogenic calcite and aragonite into limestone and dolomite (see Holland and Turekian 2014 and chapters therein).

Diagenetic processes occurring in marine sediments have a profound effect on the local and global cycling of many elements. For example, the balance between OM preservation and remineralization represents the key link between carbon cycling in active, surface reservoirs in the oceans, atmosphere, and on land, and carbon that cycles on longer, geological timescales in sedimentary rocks and in fossil fuel deposits (Burdige 2007).

Cross-References

- ▶ Acidity (pH)
- ▶ Alkali-Silica Reaction
- ▶ Classification of Rocks
- ▶ Clay
- ▶ Coal
- ▶ Coastal Environments
- ▶ Compaction
- ▶ Consolidation
- ▶ Deformation
- ▶ Engineering Properties
- ▶ Evaporites
- ▶ Facies
- ▶ Fluid Withdrawal
- ▶ Geochemistry
- ▶ Geothermal Energy
- ▶ Hydrocompaction
- ▶ Hydrothermal Alteration
- ▶ Mineralization
- ▶ Pressure
- ▶ Rock Mechanics
- ▶ Sand
- ▶ Sedimentary Rocks
- ▶ Sediments
- ▶ Shear Stress
- ▶ Silt
- ▶ Strength
- ▶ Volcanic Environments
- ▶ Water

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Dilatancy

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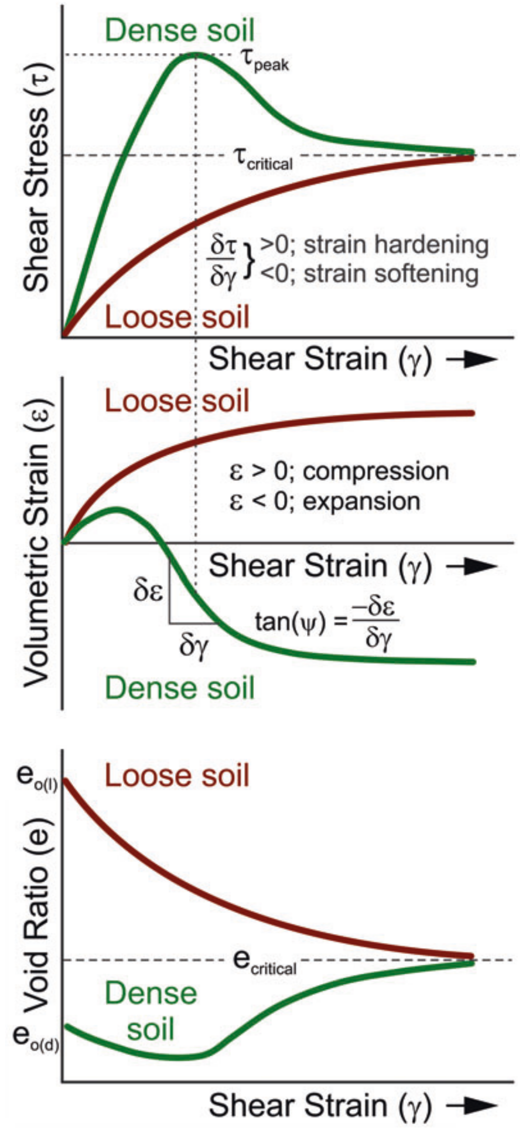
Definition

Dilatancy is the property of soil material that refers to a change in its volume in response to shearing under a certain normal or confining stress.

Context

Soil material in an initially high relative density condition (low initial void ratio, e_o) will increase in volume (increase in void ratio) to a condition of constant volume with continued shearing under the same normal stress. Conversely, the same soil material in an initially low relative density condition (high e_o) will decrease in volume (decrease in void ratio), ultimately converging to the same constant volume with continued shearing under the same normal stress (Houlsby 1991). The high-density soil response is dilative, whereas the low-density soil response is contractive (Fig. 1); the term dilatancy refers collectively to soil volume change response to shearing. The constant void ratio with continued shearing under a certain normal stress is a steady state condition known as the critical void ratio (Fig. 1). The state parameter, ψ, is defined as the current void ratio, e, of the soil minus the critical void ratio, e_c, at the same state of stress (Jefferies and Been 2016): $\psi = e - e_c$.

The same ψ symbol is used to denote the angle of dilatancy, which is the ratio of a volumetric strain rate, $\dot{\varepsilon}$, and a shear strain rate, $\dot{\gamma}$.



Dilatancy, Fig. 1 Shear stress, volumetric strain, and void ratio as a function of shear strain for loose and dense soils

$$\tan(\psi) = \frac{-\delta\dot{\varepsilon}}{\delta\dot{\gamma}} \quad (1)$$

For the case of plane strain, $\varepsilon_2 = 0$ and principal strain rates are used:

$$\sin(\psi) = \frac{-(\dot{\varepsilon}_1 + \dot{\varepsilon}_3)}{(\dot{\varepsilon}_1 - \dot{\varepsilon}_3)} \quad (2)$$

Cross-References

▶ Classification of Soils
▶ Compression
▶ Liquefaction

► Shear Strength
► Shear Stress
► Soil Laboratory Tests
► Strain
► Stress
► Voids

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Dispersivity

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Definition

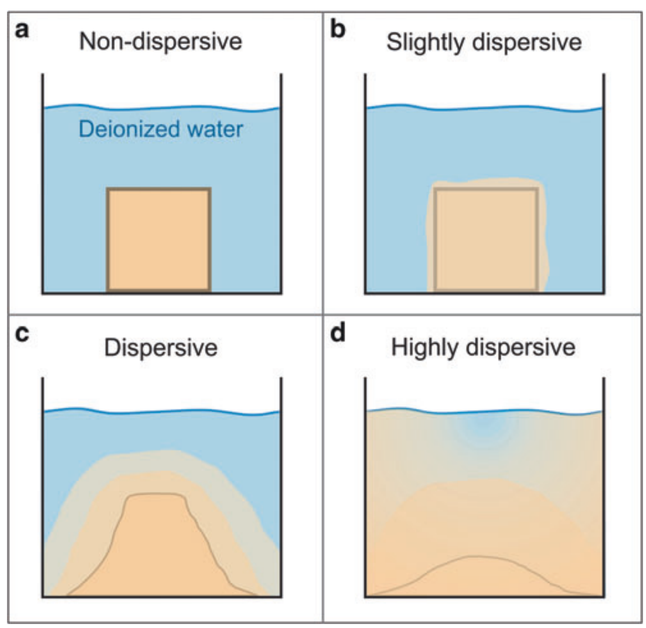
Dispersivity is the tendency of some clayey or cohesive soils exposed to saturation by surface or groundwater to separate into individual particles instead of forming small clumps or aggregates of particles known as flocs.

Context

Dispersivity results in poor behavior of compacted soil embankments, particularly those that impound water, and can lead to failure caused by erosion associated with soil piping, internal erosion into cracks, fissures, and joints, or other macro-scale openings, or migration of soil fines into pore space between larger soil grains, such as gravel or cobbles. Dispersive soils have monovalent exchangeable cations, predominantly sodium, Na^+, in the pore water, whereas nondispersive soils have divalent cations (calcium, Ca^{2+}; magnesium, Mg^{2+}; sodium, Na^{2+}; potassium, K^{2+}). Dispersive soils can be associated with weathering products derived from residual soils formed on marine shale, alluvial deposits transported from drainage basins containing marine shale lakebed sediments, and loess. Conventional geotechnical tests, such as hydrometer analyses and Atterberg limits, when used alone are insufficient for distinguishing dispersive soils from "normal" soils.

Supplemental tests developed for this purpose consist of double hydrometer tests, pinhole tests, and crumb tests. The double hydrometer test (ASTM 2011) is reported as the ratio of the percentage of soil particles finer than 5 µm from a hydrometer test run with a dispersive agent, typically sodium hexameta-phosphate, to the percentage of soil particles finer than 5 µm from a hydrometer test run with distilled water. A pinhole test (ASTM 2013a) is based on the flow of distilled water under a constant head though a 1-mm-diameter hole extending horizontally through a 25.4-mm- (1-inch-) long sample of undisturbed soil; discharge of turbid water with enlargement of the hole indicates loss of soil particles caused by dispersive reaction. A crumb test (ASTM 2013b) is a simple reaction of a lump of soil to being submerged in deionized water (Fig. 1). A test for shale and other weak rocks that is similar to the crumb test is called the jar slake test (Santi 1998). A proposed empirical equation for estimating dispersivity of cohesive soils (Dcs, dispersivity value) based on liquid limit (LL, percent), clay content (CC, percent), sodium content in pore water (PWS, percent), and pH was developed by Fan and Kong (2013):

$$\mathrm{Dcs} = 4 - 0.01\ (2\ \mathrm{LL} + \mathrm{CC} - \mathrm{PWS}) + 0.1\ \mathrm{pH} \qquad (1)$$



Dispersivity, Fig. 1 Schematic diagrams of a crumb test. (**a**) Non-dispersive soil lump shows no reaction when submerged in dionized water. (**b**) Slightly dispersive soil lump releases fine soil particles into suspension around the lump with minor coarser particles accumulating at the base of the lump. (**c**) Dispersive soil lump releases fine soil particles into suspension around the lump; substantial discoloration and cloudiness of the deionized water around the lump. (**d**) Highly dispersive soil lump disintegrates completely in deionized water, which becomes discolored and cloudy throughout the test container

Dispersivity also is a term used to refer to the ability of the concentration of contaminants in moving groundwater to diminish by the process of dispersion.

Cross-References

- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Cohesive Soils
- ▶ Engineering Properties
- ▶ Erosion
- ▶ Liquid Limit
- ▶ Reservoirs
- ▶ Saline Soils
- ▶ Sinkholes
- ▶ Soil Laboratory Tests
- ▶ Soil Properties

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Dissolution

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Definition

The process by which soluble rocks such as limestone (predominantly calcium carbonate), chalk (also calcium carbonate), dolomite (magnesium calcium carbonate), gypsum (hydrated calcium sulfate), and halite/“rock-salt” (sodium chloride) are dissolved by the passage of water or weakly acidic water either over the rock surfaces or through fractures and pores in the rock.

Introduction

Dissolution of soluble rocks proceeds at varying rates depending on the mineralogy of the rock and composition of the water. The rate is slower in less soluble rocks (limestone, chalk, and dolomite) but quicker in more soluble gypsum. Over time, the fractures become enlarged and increasingly interlinked, eventually forming complex subsurface drainage systems and in the stronger rocks, cavernous ground. The dissolution features form a landscape known as karst, which is typified by caves, sinkholes/dolines (surface subsidence features caused by collapse into caves/voids), sinking streams (surface streams that drain into sinkholes), and springs emerging at lower levels.

Dissolution of Less Soluble Rocks

Dissolution of the carbonate rocks is strongly influenced by the amount of dissolved carbon dioxide in the water. Rain water is the most common initial source picking up carbon dioxide as it passes through the atmosphere.

The simplified reactions are:

$$H_2O\text{ (water)} + CO_2\text{ (carbon dioxide)} = H_2CO_3\text{ (carbonic acid)}$$
$$CaCO_3\text{ (limestone)} + H_2CO_3\text{ (carbonic acid)} = Ca_2^{+} + 2HCO_3^{-}$$

An additional source of carbon dioxide comes as the water drains through the topsoil. Water that has passed through organic soils has some of the highest chemical aggressiveness to carbonate rocks. In addition to the carbonic acid this water contains humic acid, from the decay of humus in soil producing acidic groundwater with a pH of 4.5–5.0.

Waltham et al. (2005) pointed out the aggressiveness of water to carbonate rocks is related to the amount of dissolved carbon dioxide. As there is more available carbon dioxide in organic soils, the aggressiveness is greatest where the organic soils are thicker and more biologically active. Consequently, the availability increases at higher temperatures in low latitude and low altitude locations. As a result, carbonate karst is more developed in humid tropical and temperate areas where there is more available soil carbon dioxide and higher amounts of rainfall. Table 1 summarizes this effect and shows how the amount of denudation of limestone increases from cold Arctic areas to the hot and very wet tropics. Figure 1

Dissolution, Table 1 The influence of climate on maturity of karst terrains, expressed by the mean rates of surface lowering (excluding dissolution erosion that takes place underground) (After Waltham et al. 2005)

Environment	CO_2 content (%)	Mean air temperature (°C)	$CaCO_3$ in solution (ppm)	Rainfall (mm/a)	Denudation (mm/ka)
Mean atmosphere (temperate)	0.03	10	70 in rainfall	–	–
Mean atmosphere (tropical)	0.03	30	50 in rainfall	–	–
Arctic karst	Air in soil: 0.1	0	50 in streams	100	10
Temperate karst	Air in soil: 1.0	10	200 in streams	1,500	30
Hot wet tropical karst	Air in soil: 10	30	300 in streams	2,500	60
Hot, very wet tropical karst	Air in soil: 10	30	300 in streams	5,000	120

Dissolution, Fig. 1 Limestone pavement at Malham Cove, near Malham, North Yorkshire, England, showing the dissolution of limestone along joints forming grykes (or grikes) separating blocks of limestone called clints (© A H Cooper)

shows dissolution of a Carboniferous Limestone surface in North Yorkshire, England, since the last glaciation some 10,000 years ago.

Dissolution of More Soluble Rocks

Below about 1,000 m depth of burial, gypsum (hydrated calcium sulfate) gets dehydrated to anhydrite (calcium sulfate) with a considerable loss in volume. When it is exhumed it is metastable and at depths less than 1,000 m, and only if water is available, it hydrates back to gypsum with an approximately 63% increase in volume. Continued addition of water then dissolves the gypsum forming cavities and caves (Klimchouk et al. 1996). This process tends to occur at depths from surface to about 120 m. Gypsum can form cave systems; its high solubility (Table 2) and dissolution rate mean that in some locations phreatic cave walls might retreat at a rate of as much as 0.2–1.0 m/year depending on water flow and the amount of dissolved sulfate in the water. Unlike with limestone, dissolved carbon dioxide has little effect, but saline waters can produce much enhanced dissolution rates.

Dissolution, Table 2 Solubility of dolomite, limestone/chalk, gypsum, and halite. Solubility of common evaporites at 20° C (partly after Klimchouk 1996; Warren 2016)

Rock type	Solubility in fresh water, g/L at c. 20° C	Solubility with respect to limestone	Dissolution rate
Halite (NaCl)	359	43,900	Extremely high
Gypsum ($CaSO_4{\cdot}2H_2O$)	2.531	100	Very high
Limestone/chalk ($CaCO_3$)	0.014[a]	1[a]	Low[a]
Dolomite $(CaMg)(CO_3)_2$	0.050[a]	3.5[a]	Low[a]

[a]Note the solubility of limestone increases 10–30 times in water with dissolved carbon dioxide; solubility of dolomite is also affected

There are similarities between limestone and gypsum karst: both can contain sinkholes, but gypsum karst tends to evolve more quickly due to the high dissolution rate. Surface gypsum karst is unusual except in very arid climates and much of the gypsum karst occurs as buried karst beneath covering rocks and superficial deposits.

Halite (rock-salt), because of its extremely high degree of solubility (Table 2), only occurs at the ground surface in extremely arid areas; even then, bare salt karst is very rare. In wetter climates, salt only survives beneath very low permeability cover rocks such as mudstone or clay soils. Where salt comes near outcrop it is generally dissolved, but may be protected by a layer of dense saturated brine. Natural brine movement to salt springs can cause dissolution and sinkholes may occur in such areas. In areas where brines have been commercially abstracted, freshwater can be drawn in, reactivating subsurface dissolution channels and producing large-scale subsidence.

Engineering Problems Caused by Dissolution

Engineering problems associated with soluble rocks include subsidence, sinkhole formation, uneven rock-head, and reduced rock-mass strength. Sinkhole formation and subsidence has the potential to cause damage to buildings and infrastructure. Subsidence can be triggered by human disturbance of the ground, a change in drainage patterns, heavy rain or by water abstraction. Karstic rocks are often important aquifers, so their vulnerability to pollution is of particular concern. It is important to note that dissolution rates in limestone are so low that there is no threat to buildings or infrastructure from the creation of *new* cavities. However, cavernous gypsum can be enlarged much more quickly, particularly beneath dams and reservoirs. Gypsum is also weaker than limestone and so collapses more readily. Salt near-surface can be particularly hazardous to engineering. The dissolution of gypsum and salt produce groundwater that is aggressive to concrete whereas salt is also very aggressive to steelwork.

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Drilling

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Synonyms

Augering; Boring; Punch

Definition

Drilling is an excavation process of rocks and soils in cylindrical form that requires special tools to provide direct access to geological, geotechnical, geochemical characteristics of corresponding geomaterials for different geological activities. These geological activities can involve the exploration on type and distribution of mineral deposits, extraction of rocks/soils to provide detail on their physical, mineralogical, and mechanical properties for geotechnical projects, efforts to reach oil, shale gas, and geothermal resources and other aims.

Introduction

In parallel with population growth, societal development, and urban spread, the demand for basic requirements such as food, water, and energy has increased rapidly at accelerating speeds. To meet these requirements, particularly to provide water, salt, other mineral deposits as well as hydrocarbon resources, many technological advances have been developed in the methodologies of subsurface investigation. Kuhn (2004) emphasized that there is significant evidence indicating that humans started digging wells to access water for agricultural purposes in China at least 7000 years ago. The first known drilling carried out in China in circa 2000 BC was to produce salt brine by using a drilling tool consisting of bamboo (Mir-Babayev 2012). The depth of drilled wells reached up to 140 m (Kuhn 2004) at the beginning of the third century in conjunction with the increase in need and usage of water. Between the late 1800s and early 1900s, many significant advances (e.g., drilling the first oil borehole, utilizing water circulation in drilling, inventing rotary and mechanical percussion drilling, utilizing drilling rigs, inventing diamond core drilling, using compressed air in drilling, using flexible shafts to rotate drilling bits, inventing water-cooled drilling and using a rotary table and kelly in drilling, etc.) were observed in drilling technology due to the need for oil and its products. Modern horizontal drilling technology started in 1929 for extracting oil near Texon, Texas (Helms 2008). The deepest boreholes to date were drilled in 1984 on Kola Peninsula (Russia) to a depth of 12,261 m, in 2008 in Al-Shaheen (Qatar) to a depth of 12,288 m, and recently in 2011 in Sakhalin (Russia) to a depth of 12,345 m (Mir-Babayev 2012). From these previous studies it is clear that drilling was performed in the direction of very basic needs for mankind that eventually evolved to an unstoppable quest for energy in the last century.

Currently, there are many different reasons to perform drilling in many engineering projects. These are: (1) opening a hole to describe *in-situ* characteristics of soil and rock masses, (2) extracting samples to measure physical, mineralogical, and mechanical properties of subsurface soils and rocks, (3) performing *in-situ* testing inside the drilling borehole to obtain strength and deformation properties of soil and rock mass, and (4) reaching water, mineral deposits, and hydrocarbon resources to measure their geometries and spatial distributions or extraction. Drilling should be the final step after performing all preliminary assessment techniques (such as site investigation, geological mapping, geophysical surveying, opening trial pits and trenches, and collecting disturbed and undisturbed samples from outcrops and carrying out required laboratory tests on these selected samples) and thus obtaining an adequate database of information. The location, depth, and number of drill holes are important controlling parameters of the project budget. These parameters change depending on the importance and type of project, geological conditions, and information provided from preliminary studies. Below, the comprehensive information related to drilling techniques, sampling, and borehole logging, the *in situ* tests performed in boreholes, borehole instability problems of drilling, and finally developments recorded in deep drilling technology are discussed in detail.

Drilling Techniques

Different kind of rigs (e.g., mechanical, electric, hydraulic, pneumatic, and steam) exist from small/portable to exceptionally large, and their use varies given the aim and depth of drilling that is to be used in drilling projects (Gandhi and Sarkar 2016). Briefly, drilling is performed according to three main different methods. These are percussion, auger, and rotary type of drilling. The details related to these drilling methodologies follow.

Percussion Drilling

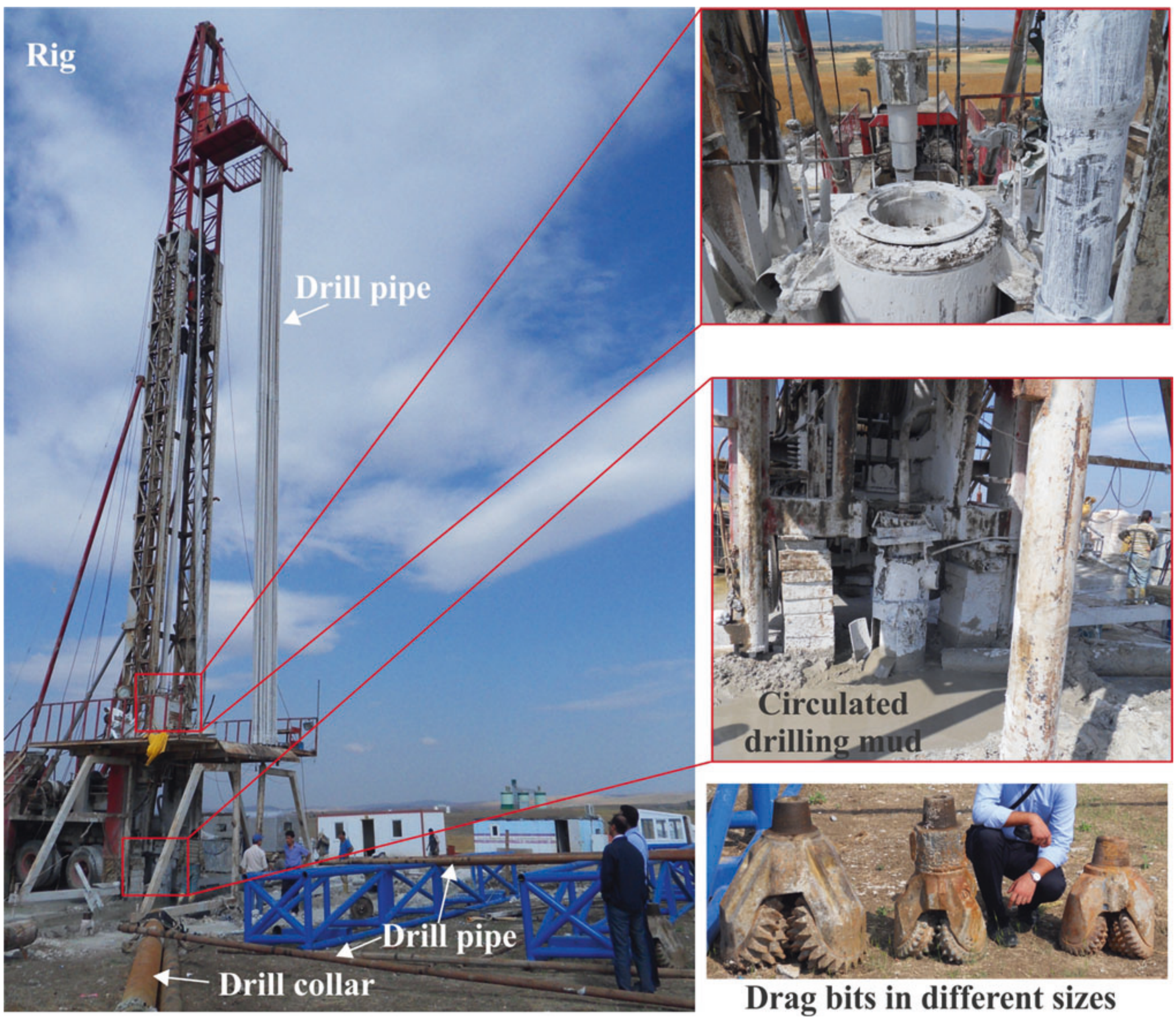
Percussion drilling techniques are known as the earliest methodology, performed by repetitively raising and dropping heavy chisel faced bits inside a drilling hole to break and pulverize geological units. After this process, the slurry of fragments and water accumulates at the bottom of borehole that eventually reaches a certain level that limits the penetration rate. To extend this threshold, the bottom of the borehole is periodically cleaned with slurry and an appropriate bailer. This drilling technique consists of four components: “drilling cable,” “swivel socket,” “drill stem,” and “drill bit.” There are many reasons to perform percussion drilling. These include relatively less water dependence in comparison with other drilling methods, drilling with a minimum of individuals, low investment costs, as well as cheap maintenance and operating costs. In contrast to these advantages, the reduction of penetration rates in hard rocks, very heavy and relatively expensive characteristics of percussion equipment and the necessity of vertical drilling due to the use of gravity reflect the main limitations of this method. In conclusion, the percussion drilling method is generally used for shallow depths and is rarely used in most engineering projects.

Auger Drilling

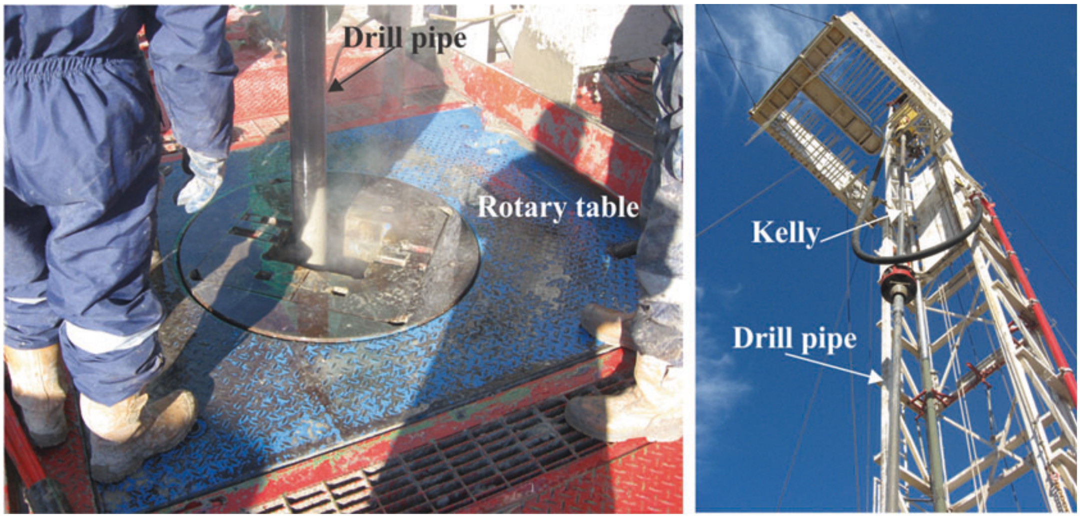
Auger drilling is a low-cost and fast method to collect disturbed samples from unconsolidated soil or weak and soft rocks for preliminary evaluation and investigation. This drilling technique is used in the reconnaissance stage of mineral exploration in addition to extracting suitable specimens for geotechnical studies, soil surveys, environmental studies, and construction purposes (Gandhi and Sarkar 2016). The extraction of undisturbed samples is not possible with this method. The mechanical properties of geomaterials and the size of the rig and stem control the penetration rate of auger drilling (Gandhi and Sarkar 2016). In this method, cuttings are directly removed from borehole by the auger without using drilling fluids such as water, air, bentonite, or any kind of mixture. According to Gandhi and Sarkar (2016), samples from 3 m to 50 m depths can be collected by hand augers and tractor/tire-mounted augers, respectively.

Rotary Drilling

Rotary drilling is a process of opening a cylindrical-shaped excavation down through the Earth using a sharp drill bit that turns around on its own axis. Although this drilling methodology is not new and dates back to 3000 BC (Gandhi and Sarkar 2016) based on archeological studies of ancient Egyptian civilizations, we conclude that rotary drilling was not commonly preferred until the first decade of the 1900s. Along with the increase in consumption of oil, gas, and other commercial mineral and energy deposits, a significant percentage of boreholes have been drilled by rotary techniques. With the help of modern technological advances and new innovations, rotary drilling can be completed in any dip and dip direction, which is particularly useful for projects related to shale gas/oil exploration and development. The basic equipment and accessories for rotary drilling can be grouped as follows: prime movers, derrick and hoist, rotating and circulating systems. Some of this equipment is shown in Fig. 1. The prime mover is the source of power to the entire rig. The derrick and hoist are used for lowering and raising equipment inside the borehole. This system is designed to have adequate capacity to pull, holdback, and lower the combined loads of the drill bit, drill collars, and drill pipe. As the depth of borehole increases, the height of a rig's derrick also increases. The rotating equipment, which involves a number of different components such as swivel, kelly, rotary table assembly (Fig. 2), and drill pipe whose assembly includes drill string, drill collars, and drilling bit, provides two main functions: changing the direction of rotation and then transferring power generated by the prime mover to the drill bit. In the case of deep drilling such as for geothermal exploration, a drill collar is generally used to supply extra weight to the rotating system. Different types of drill bits (tungsten carbide, steel tooth bits (e.g., tricone), polycrystalline diamond compact bits, and diamond bits) are designed to take into account the strength, and mineralogical composition of formations. The bit is pushed into the bottom of borehole and the generated force rotates it to cut and break up the geological formation as it continues penetration. The cuttings (fragments and other fine particles formed as result of drilling) are removed at the bottom of the borehole by circulating systems. For this process, a drilling fluid is pumped



Drilling, Fig. 1 A rotary drilling rig for geothermal exploration in Kutahya (Turkey)



Drilling, Fig. 2 A view from the rotary table during the addition of a new drill pipe to the drill string.

inside the drill string and then the mixture of drilling fluid and cuttings exits from the annulus space between the drill pipe and the wall of borehole. In addition to removing cuttings and cleaning the bottom of borehole, drilling fluids also have many other significant functions including supporting the sidewall from instability problems such as caving, balancing the subsurface pressures of water and gas, cooling and lubricating the bit, decreasing friction, suspending cuttings, and blocking leakage problems by building a filter cake on the wall.

Depending on the drilling purpose, the depth of borehole, and the type of geomaterials, the more commonly used types of drilling fluids can be water, the mixture of water and natural clay or commercial bentonite, mud-laden, air, or oil based mixtures (Shuter and Teasdale 1989). The mixture of clay and water called mud is utilized in subsurface investigation of soft geomaterials such as sand, silt, and clay layers (Gandhi and Sarkar 2016). According to Gandhi and Sarkar (2016), mud-rotary drilling methods are used in many projects such as ship/offshore platforms for oil and gas exploration that reach greater depths. Gandhi and Sarkar (2016) specified that the air-rotary drilling technique is generally applicable to the soft rocks up to 25 m depth. In this method, many of above given functions of drilling fluid (such as removing cuttings, cleaning the bottom of the hole, and cooling the bit) are provided by forcing down compressed air. However, it is recognized that some structural and geomechanical characteristics of discontinuities may not be preserved in the samples extracted by the air-rotary drilling method.

In addition to the abovementioned drilling methods, more details regarding other drilling methods such as “percussion rotary air-blast drilling,” “rotasonic (sonic) drilling,” and “hydraulic drills” can be found in a research performed by Gandhi and Sarkar (2016).

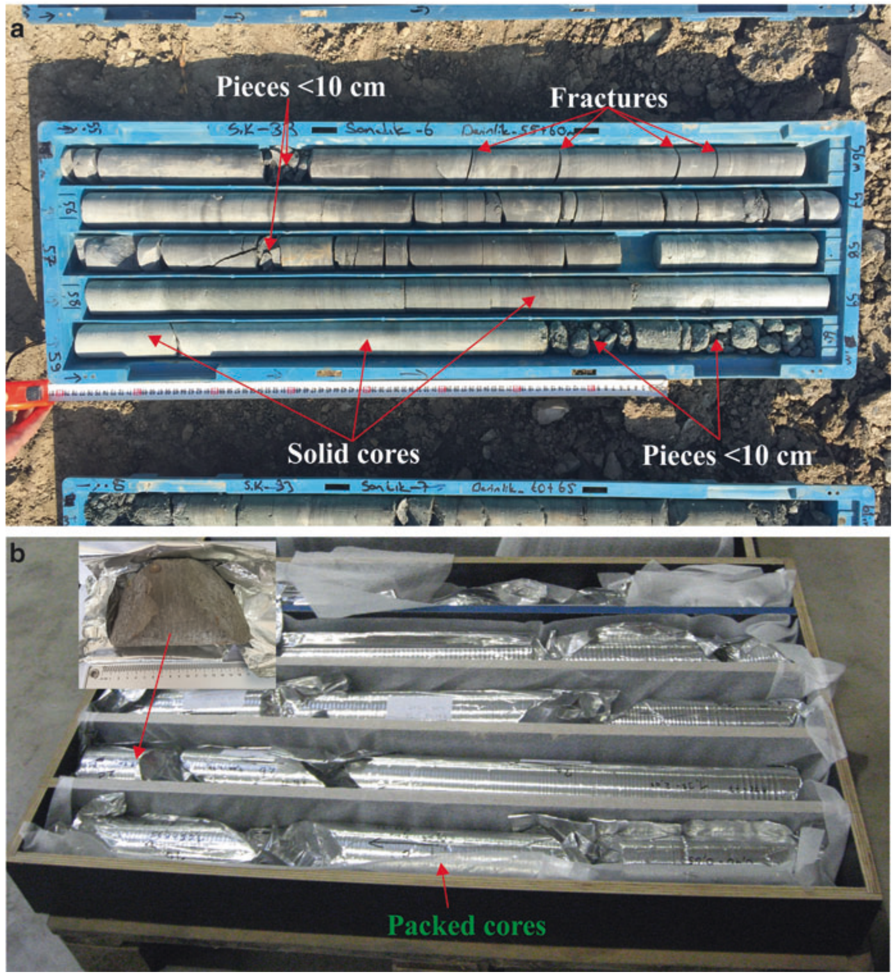
Sampling and Borehole Logging

The efforts for obtaining subsurface geomaterials for geological, geochemical, and geotechnical studies generally constitute the main purpose of drilling programs in many engineering projects. Thus, to directly describe and measure the physical, mineralogical, and mechanical characteristics of subsurface soils and rocks at relevant depth, disturbed and undisturbed samples are obtained from boreholes during the drilling processes. The disturbed samples, which consist of cuttings or sludge taken from air-flushed diamond, mud rotary, auger, or percussion drilling or soil samples obtained from split-barrel sampler of standard penetration test (SPT), are used to determine physical (e.g., Atterberg limits, specific gravity, color, particle shape and grain size distributions of soils) and mineralogical characteristics of geomaterials. Whereas, the undisturbed samples, in addition to their usability for these physical and mineralogical properties, are also used to determine the mechanical properties of materials as well as the geomechanical characteristics of discontinuities. Many tools are manufactured for extracting soil and rock samples during drilling. These are: (1) open-drive and piston sampler, (2) thin-wall open-drive sampler, (3) piston-drive sampler, (4) split-barrel sampler, (5) double-tube soil core barrel, (6) single-tube rock core barrel, and (7) double-tube rock core barrel (USDA 2012a). Furthermore, considering the time-consuming nature of these conventional sampling tools for deeper drilling, a wireline core barrel system is also used to reduce the time for collecting cores at the bottom of borehole. Besides sampling and visual classification of subsurface geomaterials, drilling also provides suitable spaces for installing *in situ* testing tools to measure groundwater level, determine strength, deformation and permeability characteristics of the mass of soils and rocks, as well as to identify *in-situ* stresses.

Extracting samples from the subsurface and performing *in situ* tests inside the drill hole are expensive stages of engineering projects. Thus all information obtained from drilling should be technically recorded very carefully using a proper borehole log as a database for a comprehensive assessment of the current project or for any other research requiring further investigation and observation. A draft borehole log should be provided during drilling and then this draft version should be updated to produce a finalized borehole log that integrates the results obtained from *in-situ* and laboratory studies. The finalized borehole logs generally involves at least the project details (e.g., name of project, contractor, date of drilling, the coordinates of the location, etc.), the depth of groundwater, description and boundaries of encountered geomaterials, drilling methods, run lengths, drill bit types, tools used for sampling, sampling intervals and types, the core-loss and core recovery, as well as the results of *in-situ* and laboratory tests and measurements. In the case of drilling soils, visual or field index tests-based soil description and classification are presented in borehole logs. For naming, description, and classification of soils, the cuttings, disturbed samples obtained from SPT split spoon sampler and undisturbed samples of thin-wall (Shelby) tube or piston drive sampler are generally used. Clean drilling cores should be used for naming and describing intact rocks under wet and daylight condition soon after sampling (Fig. 3). In addition, rock type, degree of weathering, geomechanical properties of discontinuities (type, dip, fracture frequency, roughness, aperture, and infilling) and core recovery are determined on core samples and presented on borehole logs. The extracted cores are generally preserved in a plastic, aluminum, or wooden tray (core boxes) labeled with drilling details for future reference. A typical plastic core box is shown in Fig. 4a. Considering fast disintegration behavior of some clay bearing rocks (e.g., Boom Clay and Opalinus Clay) to the natural physical and chemical weathering processes of core storage space, the extracted core samples are sometimes packed with an aluminum-polyethylene film under vacuum conditions before storage in a core box (Fig. 4b). The conventional core sizes are AQ (27 mm.), AX (30.1 mm.), BQ (36.5 mm.), BX (42.0 mm.), NX (54.7 mm.), NQ (47.6 mm), and HQ (63.5 mm), and the determination of some parameters of geomaterials requires extracting a representative core sample. For instance, cores having a diameter smaller than BX size should not be used to measure Rock Quality Designation (RQD) in accordance with ASTM D6032 (2002).

In geotechnical studies, representing core recovery with a suitable index parameter is very important for understanding the fracture state and thus the failure behavior of rock masses. For this purpose, Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), and Fracture Frequency (FF) are used for quantitative description of fractures on core box. TCR (%) is the percentage of the length of recovered core divided by the total drilled length of the core run. The recovered cores can be solid or intact without the full diameter for TCR. However, SCR (%) is defined as the percentage of the length of solid core recovered relative to the total drilled length of the core run. For measuring RQD, the length of recovered core pieces should be at least 10 cm. In addition, RQD is recommended only for rocks, and so this parameter should not be used as fracture index of soil cores. FF is the number of identifiable natural fractures per meter run. Core breaks caused by drilling processes, blasting and other artificial triggers should be ignored and thus counted as a single piece of solid core in defining these quantitative fracture parameters. Core recovery in medium to hard formation changes between 80% and 100% and so it is very hard to achieve a core recovery of 100% from drilling soft rocks or heavily jointed rock mass formations (Gandhi and Sarkar 2016). According to Gandhi and Sarkar (2016), the core recovery generally ranges between 10% and 20% for soft/friable and loose soil/sand. The main factors affecting core recovery are the type of lithology, fracture density, the drilling and sampling equipment, excessive core

Drilling, Fig. 3 Core samples of various rock types extracted by diamond rock core barrels having different dimensions (the core of Opalinus Clay was obtained by using a lathe)



Drilling, Fig. 4 (**a**) A typical plastic core box, (**b**) core box with samples packed with an aluminum-polyethylene film (Boom Clay at the underground rock laboratory (SCK•CEN), Mol, Belgium).

washing depending on the type of selected core barrel, inappropriate selection of bit speed and feed, excessive vibration of string, and selection of the wrong drilling fluid (Gandhi and Sarkar 2016).

In Situ Tests Performed in a Borehole

It is frequently difficult to extract undisturbed samples from soft and sensitive soils. In addition, when there is disturbance during sampling, transporting, and finally preparation for testing, scale effect and changes in *in-situ* stresses must be considered, such that the data obtained in the laboratory may not be representative. Thus, a number of *in situ* techniques exist to measure more accurately design parameters of soils and rocks inside boreholes. The main advantages of field testing are: (1) insignificant or complete absence of the effect of disturbance in *in-situ* tests, (2) achieving more representative engineering parameters (Bell et al. 1990), (3) immediately acquiring test results, and (4) using a larger size to overcome or reduce the scale effect on test results. Drilling boreholes provides both samples and a suitable space for performing *in-situ* tests. Some *in situ* tests carried out in boreholes are discussed below.

Standard Penetration Test (SPT)

This *in situ* test is performed at the bottom of a borehole to measure the resistance of soils, particularly the relative density of cohesionless soils and the relative field consistency of cohesive soils by freely and repeatedly dropping a drive hammer with a mass of 63.5 kg anvil until reaching a penetration depth of 450 mm. The number of SPT blowcounts is recorded for each 15 cm increment of the penetration, and the sum of the last 30 cm of penetration called the “N-value” is used for resistance assessment of soils. Standard penetration tests involve drilling equipment, hammer, anvil, drilling rods, and split-tube sampler. The disturbed samples accumulated inside a SPT split-tube sampler are generally used for determining consistency limits and grain size distributions of soils. Based on the available project budget, the sampling interval of SPT should not be more than 1.5 m for each drill hole. For more detailed information on SPT, check a study performed by Schnaid (2009).

Cone Penetration Test (CPT)

Similar to the SPT test, this *in-situ* test is also widely performed in determining the resistance of soils (e.g., soft clay, silt, sand, etc.) that are difficult to sample. In geotechnical site investigations, CPT is known as a quick and cheaper *in situ* test to provide a continuous reading of tip resistance (q_c), porewater pressure (u), and sleeve resistance (f_s), then used for assessments on evaluating relative density, strength, bearing capacity, and liquefaction potential of sensitive soils. The mechanical cone penetration test method mainly consists of a cone, load cell, friction sleeve, and rod. There are currently different types of CPT such as "mechanical cone penetration test," "electrical friction cone," "piezocone penetration test," "piezocone with dissipation," "seismic piezocone test with embedded geophones," and "resistivity piezocone test" in site investigation of soils (USDA 2012b).

Vane Shear Test

Vane shear test provides an *in situ* methodology to determine peak and remoulded undrained shear strength of rather sensitive soil in which undisturbed sampling is very difficult. A typical vane shear test usually contains vane, casing, torque device, extension rods, and spanner for an extending rod. The procedure for this testing is rather simple. The vane test is initiated by inserting a vane vertically into soils and then the vane is rotated by a torque device at a speed of 6–12° per min. The rotation force is applied to the vane until reaching a maximum torque value. According to Schnaid (2009), this test can be performed on all soils, which have an undrained shear strength value up to 200 kPa, except for sand, gravel and other highly permeable soils. In addition to field vane test, a pocket vane shear test device is also available to quickly obtain undrained shear strength of soils found in excavated pits or trenches (USDA 2012a).

Pressuremeter Test

The pressuremeter test developed by Louis Ménard in 1957 is based on the principle of measuring stress-strain behavior of the drilled materials by vertically placing a cylindrical probe consisting of guard and measuring cells inside a borehole and then applying radial pressure to the borehole wall by this flexible membrane. The probe and control unit consisting of a pressure supply, displacement, and pressure sensors are the main components of a typical pressuremeter (Schnaid 2009). Pressuremeter tests are used to provide deformation characteristics for overconsolidated clay, weak clay-bearing rocks (e.g., mudrock) and sand (Bell et al., 1990). In addition, Schnaid (2009) emphasized that the capacity of the instrument changing between 2.5 MPa and 20 MPa provides applicability for the pressuremeter test to identify stress-strain relationships of hard soils and soft rocks. Three different types of pressuremeter tests, "*pre-bored,*" "*self-boring,*" and "*push-in*" pressuremeters, are based on installation method, as utilized in geotechnical site investigation (Schnaid 2009).

Deformability Test

There are several testing techniques to measure deformation characteristics of geomaterials inside a drilled borehole. The typical type of tests widely used in geotechnical site investigation are "Downhole Plate Test," "Downhole Flexible Dilatometer," and "Downhole Stiff Dilatometer." The required equipment and procedures for these tests are detailed in the contribution "Rock Field Tests" in this volume.

Permeability Tests

Permeability tests performed in a borehole are classified into three groups: constant-head, falling-head, and pressure tests (USDA 2012a). The permeability of soils can be identified by measuring the flow rate at the steady state after changing the groundwater level in a borehole (Bell et al. 1990). The constant-head permeability test is used for coarse-grained and unconsolidated soils revealing moderate to highly permeable inherent characteristics. However, the falling-head permeability test is preferred in fine-grained unconsolidated soils that generally have permeability values ranging from low to very low. The hydraulic conductivity of consolidated geomaterials (e.g., fractured rock masses) is measured by techniques involving changing pressures in an isolated part of a borehole and then recording flow rate. This pressure test method is also called a "Lugeon test" or "packer test". In order to accurately measure permeability of geomaterials in a drill borehole, drilling fluid should be prepared at minimum values of viscosity, density, and gel strength (USDA 2012a). In addition, USDA (2012a) specifies that the test performed in boreholes drilled by augers or rotary systems and an engineered drilling fluid, drilling with caution gives the most precise values for permeability characteristics of related geomaterials.

Hydraulic Fracturing Test

Hydraulic fracturing method is performed in drilled boreholes to find *in-situ* stresses by recording fluid pressure needed to initiate a new fracture or reopen an existing fracture. The main components of this method are packer, high-pressure hose, winch, power supply, pump, packer and interval pressure transducers, recorder, and computer. For details on hydraulic fracture test, check the contribution “Rock Field Tests” in this volume.

Borehole Instability Problems of Drilling

The nature of soils, rocks, or drilling methods may all contribute to borehole instabilities and other technical problems that result in a loss of money, time and drilling equipment, and significantly reduce the success of the drilling program. Experience shows that the risk of borehole instability problems increases with increasing drilling depth. In addition, 75% of the drill holes on shale and recorded that 90% of all borehole instability happens in shale formations (Steiger and Leung 1992). Thus, borehole instability is a very common problem especially during horizontal drilling for extracting shale gas (You et al. 2014). Pašić et al. (2007) stated that borehole instability can occur as a result of the combined effects of controllable and uncontrollable (natural) factors. The natural factors include heavily fractured or faulted formations, tectonically stressed formations, high *in-situ* stresses, rocks having nonelastic behavior, unconsolidated soils, naturally overpressured shale collapse, and induced over-pressured shale collapse (Pašić et al. 2007). According to these researchers, the factors contributing to borehole instability that can be controlled during drilling are fluid density, borehole inclination and azimuth, transient pore pressures, physical and chemical reaction between rock and fluid, the vibrations of the drill string, erosion, and temperature. Some of the common borehole instabilities and technical problems are summarized below based on previous studies (e.g., Pašić et al. 2007; CNPC 2011; Capuano 2016; Gandhi and Sarkar 2016) and the author’s experience with shallow geotechnical and deep geothermal drillings:

- Falling large or small rock pieces into boreholes due to fracture frequency, fault zones, disintegration behavior of clay-bearing rocks with water interaction or low shear strength parameters of some foliated metamorphic rocks such as serpentinite. In addition, this kind of instability may also depend on the drilling method. For instance, clogging/obstructing debris cuttings inside the annulus space in percussion rotary air-blast drilling (Gandhi and Sarkar 2016). This mechanism of instability results in jamming or sticking of the drilling string.
- The collapse of boreholes due to intersection of weak bedding planes and a borehole at an unfavorable angle (Pašić et al. 2007). A typical example for such instability occurred in a horizontal borehole drilled in the Opalinus Clay of the Mont Terri Rock Laboratory (Switzerland) is given in Fig. 5.
- The splintery cavings in tectonically stressed formations (Pašić et al. 2007).
- Unloading dependent squeezing of some geomaterials such as salt and swelling type clay mineral bearing rocks.
- The collapse of borehole due to unloading dependent physical weathering of shales.

Drilling, Fig. 5 Sliding along bedding planes in a horizontal borehole drilled in the Opalinus Clay at the Mont Terri Rock Laboratory (Switzerland)

- Falling of cohesionless soils into boreholes due to insufficient thickness of the filter cake.
- Sidewall instability due tensile spalling characteristics of rocks.
- Failure of the sidewall due to change in the strength of corresponding rock as a result of the physical and chemical reaction between rock and drilling fluid.
- The drill string vibration based on enlarging the diameter of the borehole.
- Too high of an annular circulating velocity based on erosion of the borehole wall (Pašić et al. 2007).
- Collapse in the pipe in the case of zero internal pressure of (no fluid inside) the pipe (Capuano 2016).
- Solution of some geomaterials such as clay, salts, and trona, etc.
- Swelling of clay-bearing rocks and sticking of the drilling string.
- Deviated borehole in particularly for deep drilling. According to Gandhi and Sarkar (2016), the deviation is insignificant for short boreholes having a depth less than 100 m; however, it is likely to be 25° or even more for a borehole depth of 800–1000 m.
- Thermal fatigue due to thermal expansion and contraction in geothermal drilling (Capuano 2016).
- The leakage of drilling fluid into the surrounding rock due to high pressure and high density of the drilling fluid in geothermal boreholes (Capuano 2016).

The increase in cuttings, caving at surface, borehole fill after drilling, requirement for excess cement, high torque and drag, hanging up of drilling equipment (drill string, casing, or coiled tubing), stuck pipe, extraordinary vibration of drill string, failure of drill string, problems in controlling deviation, etc., are good indicators for understanding the above mentioned borehole instabilities (Pašić et al. 2007).

Developments in Deep Drilling Technology

In addition to a drilling technique dependent classification, drilling is also classified based on the depth of boreholes. Accordingly, there are five different drilling categories: very shallow borehole (depth $<$ 100 m), shallow borehole (depth between 100 m and 1000 m), deep borehole (depth between 1000 m and 4000 m) and very deep borehole (depth $>$ 4000 m). In addition, CNPC (2011) suggests to classify boreholes as deep wells that have a depth between 4500 m and 6000 m, and ultra-deep wells whose depth is more than 6000 m. The increase in the depth of drilling and the complexity of geological units require more modern solutions in drilling technology. The steady increase in demand for energy with each day and the reduction of conventional fossil fuels has led engineers to embark on modern projects in developed countries to achieve alternative energy sources. As a result of these research efforts, significant technological developments are acquired in drilling technology for the extraction of shale gas and oil.

As discussed above, the efforts to reach these deeper sources has resulted in the development of directional drilling technology to deviate and orientate drilling tools into geological units having horizontal and inclined bedding. At the present time, the innovations in drilling equipment as well as steerable wireline core barrels provides horizontal, stepwise horizontal, and multilateral boreholes at a high drilling rate and low cost in many large worldwide projects (CNPC 2011). CNPC (2011) specifies the need for such drilling equipment as new drilling rigs, drilling pump/high pressure manifold systems, winch, disc brake, electronic driller system, wireless measurement and control system; underbalanced equipment such as rotary BOP, air compressors, boosters, and snubbing tools; and suitable well-advanced software. Combining these advances in drilling technology and hydraulic fracturing techniques now provides shale gas to be the alternative geo-energy source. However, it should also be noted that such advances in providing unconventional shale gas and oil brings potential problems. Durand (2012) stated that the hydraulic fracturing irreversibly changes the permeability of the rock masses and therefore, the continuous accumulation of shale gas over geologic time is likely to be an important environmental disaster in the future due to the impossibility of returning permeability characteristics of the rock matrix to its initial state.

Summary

Drilling as an expensive subsurface investigation technique acquires precise information required for final evaluations on the applicability of the geological, geochemical, and geotechnical based projects. The first drillings were performed because of very basic requirements such as providing water for drinking and later for irrigation and producing salts. Studies indicate that the Chinese initiated opening wells to obtaining sufficient amounts of water for use in agricultural activities about 7000 years ago. In order to obtain salt brine, the first drilling was also performed in China some 4000 years ago by utilizing drilling tools manufactured from bamboo. The main purpose of reaching water and salt in drilling evolved very quickly to new geological activities over time. New purposes such as explorations for mineral deposits, conventional and unconventional hydrocarbon resources have been included to these initial goals of drilling in particularly during the last two centuries. Accordingly, many significant innovations (e.g., directional drilling) are new to drilling technology. Drilling techniques can be classified into three main groups: percussion, auger, and rotary types of drilling. Such drilling techniques are used in describing *in-situ* characteristics of geomaterials, extracting disturbed and undisturbed samples for identification of physical, mineralogical, and mechanical properties of subsurface soils and rocks, realizing *in-situ* testing and reaching as well as measuring geometries and spatial distributions of water, mineral deposits, oil, and gas.

Cross-References

- ► Borehole Investigations
- ► Boreholes
- ► Drilling Hazards
- ► Exposure Logging
- ► Hydraulic Fracturing
- ► Rock Field Tests
- ► Soil Field Tests
- ► Wells

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Drilling Hazards

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Definition

Any unplanned event or activity that forces a drilling operation to deviate from its predefined plan or critical path (Pritchard 2010).

Also known as a "trouble"; these unplanned events lead to non-productive drilling time or minor impacts to the drilling (small amounts of fluid lost) to catastrophic wellbore failure and loss of control in the drilling operation (bottom hole assembly [BHA] is stuck in the borehole and the drill string twisted-off). These events are attributed to not only geological complexity but also mechanical failures or human error. Ultimately, these disruptions will affect the drilling timeline, budget, project completion, and reputation of the companies involved. Uncertainty drives risk everywhere. In the literature, drilling hazards are most often discussed with the completion of deep sea boreholes (e.g., Gala et al. 2010) but are not uncommon on land for engineering geology and associated geotechnical testing and monitoring boreholes, oil and gas wells, mineral exploration, geothermal wells, stratigraphic boreholes, and developing water supplies.

Managing drilling hazards is one of the most important aspects of conducting subsurface geology and engineering investigations. Drilling hazards stemming from uncertainties in geological conditions, mechanic failures, extremes in environmental conditions, or human error are experienced at offshore and terrestrial sites. If not planned for or mitigated during the drilling process, their adverse effects lead to events ranging from non-productive time to catastrophic wellbore failure or even loss of well control (Pritchard 2010).

Studies conducted over the past decade have shown that ~50% of the drilling hazards resulting in non-productive time can be avoided or mitigated using good drilling practices, pre-drilling geophysical surveys, and real-time data collection (Pritchard et al. 2010). It is estimated that 10–20% of drilling time is spent recovering from such unexpected incidents (Hoetz et al. 2013). It is anticipated that actual costs will exceed planned costs. Therefore, contingency funds, often

>10% of the total budget, are retained to cover these unexpected costs (Gala et al. 2010). However, when there is a need to maximize drilling efficiencies to meet budgetary restrictions, proactive evaluation processes and cutting-edge technologies are implemented to address drilling hazards upfront (Gongquan and Zhizhan 2011).

To assist the drilling industry to maintain safe working conditions and control operating costs governments and professional organizations have developed practical guidelines and codes of practice. They outline the range of hazards associated with drilling operations and the risks associated with operating drilling equipment. Furthermore, they discuss the risk management process and provide guidance on the methods and systems that can be used to eliminate or reduce some of the risks associated with drilling activities. The following publications are sources of the information regarding drilling safety:

- International Association of Drilling Contractors: IADC Drilling Manual (12th Edition). http://www.iadc.org/ebookstore/ebook-the-iadc-drilling-manual-12th-edition-complete/
- Canadian Diamond Drilling Association: Safe Work Methods Surface Handbook. https://www.cdda.ca/product/safe-work-methods-surface-handbook/
- British Drilling Association: BDA Health & Safety Manual for Land Drilling 2015. http://www.britishdrillingassociation.co.uk/Publications/BDA-Health-Safety-Manual-for-Land-Drilling-2015-A-Code-of-Safe-Drilling-Practice-Free-for-BDA-members-10
- National Groundwater Association: Environmental Remediation Drilling Safety Guideline. http://www.ngwa.org/Documents/erdsg.pdf
- Government of Western Australian: Guidance About Exploration Drilling Hazards. http://www.dmp.wa.gov.au/Safety/Guidance-about-exploration-6803.aspx
- US Federal Energy Regulatory Commission: Guidance for Drilling in and Near Embankment Dams and Their Foundations. https://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide/drilling/guidelines.pdf
- Prospectors and Developers Association of Canada: E3 Plus: Framework for Responsible Exploration. http://www.pdac.ca/docs/default-source/priorities/e3-plus—toolkits—health-and-safety/drilling.pdf?sfvrsn=7e281e6d_4
- International Continental Scientific Drilling Program: Scientific Drilling. https://www.scientific-drilling.net/index.html

The publications cover aspects of offshore and land-based drilling for engineering tests and foundations; oil, gas, and mineral exploration; mining and blasting; water supplies and aquifers; and scientific research. The authors discuss the potential control measures and assessment that could be adopted to reduce or eliminate the effects of hazards, and the role of training and education in providing information to the client, contractors, and professional memberships.

Hazard Types

The hazards associated with offshore and land-based drilling can be discussed in terms of unexpected events associated with geological heterogeneity and complexity in the subsurface, difficulties specific to the various drilling techniques and equipment, and risks associated with undertaking activities at the work site.

Complex Geology and Subsurface Conditions

No matter how much planning is done, it is likely problems will arise while drilling a borehole. The ability to maintain a stable wellbore is a challenge and becomes increasingly more difficult when completing directional sections within a small diameter hole and applying enough energy to clean out the borehole. The most prevalent drilling hazards include geological faults and structures, pipe sticking and drill pipe failures, lost circulation, borehole deviation, pipe failures, borehole instability, formation contamination, hydrogen sulfide or other gas, hydraulic fracturing, buried valleys, and man-made features (Mitchell 2007; Baird 1976).

Geological Faults and Structures

Faults can act as conduits for high pressure oil, gas, or water from depth. A sudden influx of fluids and gas could impact the wellbore. In addition, faults may separate formations with contrasting pressures and porosities, which if cross-connected may lead to loss of hydrostatic head. This could result in loss of primary well control. Drilling could cause further fracturing of rocks, creating voids, and lead to fluid loss. Mineralization in fault zones may cause deflections in the drill string and lead to BHA. The drilling process could induce seismicity.

Pipe Sticking and Drill Pipe Failures

During the drilling operation, the drill pipe may become stuck from mud-hydrostatic-pressures, caving, sloughing, or collapse in the borehole, in plastic shale or salt sections, and key seating. Drill pipe failures occur as twist offs caused by excessive torque, parting from excessive tension, burst or collapse caused by excessive internal pressure or external pressure, respectively, and fatigue as a result of mechanical cyclic loads with or without corrosion.

Lost Circulation

Lost circulation is defined as the uncontrolled flow of mud into a formation when mud continues to flow to the surface with some loss to the formation or mud flows into a formation with no return to surface. Loss of circulation may occur in formations that are inherently fractured, cavernous, or have high permeability.

Borehole Instability

Borehole instability is an undesirable condition in open holes where the wellbore narrows (creep), enlarges (washout), fractures, or collapses. The change in structural integrity is caused by mechanical failure by *in-situ* stresses, erosion caused by fluid circulation, and chemical caused by interaction of borehole fluid with the formation.

Contamination of Producing Formations and Aquifers

Drilling fluids may cause impairment to the producing formation (reservoir) or aquifer, if the fluid is allowed to invade the rock/sediment surrounding the wellbore. The fluids will reduce the *in situ* permeability or may lead to cross-contamination of reservoirs/aquifers.

Shallow Gas

If sufficient volumes of gas or water are encountered unexpectedly during drilling, a blowout may occur. Gas trapped in the near-surface unconsolidated (Quaternary) sediments and bedrock originates either from deeper reservoirs or from biogenic activity. In sedimentary bedrock, there may be the potential to encounter toxic or flammable gases such as hydrogen sulfides or methane. In these situations, site specific safety operating procedures (SOPs) are required to address the potential risk of encountering gas. Appropriate equipment such as continual gas monitoring and masks must be available at the drill site.

Hydraulic Fracturing

Excessive pressures from water, air, drilling fluid, or grout can fracture embankment and foundation materials or bedrock. Hydraulic fracturing leads to loss of fluid circulation, blowouts into nearby borings, seepage of drilling fluids on the face of the embankment, and other similar situations. Hydraulic fracturing can also lead to induced seismicity and ground shaking. Sherard (1986) contains references that provide a comprehensive evaluation of the issues along with numerous case histories.

Buried Valleys

In buried valleys, higher fluid pressures may be encountered requiring casing to be installed for the control of artesian conditions, if the pressures are anticipated to be significant and/or derived directly from reservoir head. In this situation, the contractor must be informed and instructed to use blowout protection on drill equipment. Consideration should be given to extend the exclusion zone around the rig to prevent potential exposure to other workers.

Anthropogenic Hazards

Drilling in areas impacted by human activity may encounter, for example, pipelines, buried debris, landfill, voids associated with past mining, and historical archaeological items. The disturbed ground when penetrated may subside or collapse resulting sudden loss of drilling fluids or workings and blowouts.

Mechanical Systems

At the drill site, hazards specific to drilling technique and the associated machinery, tools, and equipment require attention to prevent damage and injuries. While most tasks related to drilling are the responsibility of the lead driller, the other personnel onsite (e.g., engineering geologist, geotechnical engineers, samplers, and project geologists) should be familiar with the drillers' work so they can identify and report potential hazards. This will enable the levels of risk to be evaluated and reported, as part of taking responsibility for their own safety (PDAC 2009).

The working condition of drilling equipment and its maintenance are major factors in minimizing hazards. Functioning monitoring and recording systems are required to monitor trend changes in all drilling parameters that may identify potential hazards and malfunctions. The following is a brief summary of the drilling techniques and the associated hazards.

Diamond or Rotary Drilling

Diamond or rotary drilling uses slurries of bentonite and synthetic muds or water circulation to remove cuttings and keep the borehole wall stabilized. The rotating equipment and parts present a hazard to the personnel working near the drill rig. The use of hydraulic systems, including hoses and hose couplings, should be secured to restrain the hose in case of failures, which is a serious hazard. Drill rigs with automated rod handling equipment are safer than rigs requiring manual handling of the drill rods. The heavy equipment (drill rods, samplers, and augers) have the potential to cause injuries to the back and hands. Dust is a hazard when mixing the drilling fluids. Slippery or dangerous work areas occur near the mud pits or troughs. High noise levels require ear protection.

Reverse Circulation (RC) and Compressed Air Drilling

Compressed air is used as the circulation medium for reverse circulation (RC), rotary air blast (RAB), air core, and rotary percussion drilling. These methods recover rock chips or gravel and cobbles for sampling. Wellbore stability and consequential hazards such as stuck pipe, fluids loss, and equivalent circulating density (ECD) require attention. The use of compressed air and associated dust is a serious hazard. Blown hoses may result in severe injury when high pressure air lines burst due to blockages or become uncoupled. The cyclone should be set up downwind from the primary working areas. Rock fragments are ejected at such speed that a cyclone is required to separate the rock cuttings and dust from the return air. Samplers must be aware of the hazards and no worker should stand near the cyclone while the drill is operating. The high noise levels require ear protection.

Auger Drilling

Augers are used for drilling through soft and unconsolidated sediments to take samples and cuttings for engineering classification and characterization. Similar hazards exist from rotary drilling. Auger flights are very sharp and loose clothing can be pulled into the rotating machinery resulting in serious injury or death. Augering through buried geotextile and other fabricated caps can pull the operator off their feet and into the rotating flight unless the drill is operated from a platform.

Hydraulic Systems

Nearly all operating drilling rigs contain some type of hydraulic system as part of the mechanical operation. To reduce risks, operators should ensure: (1) hydraulic pressures do not exceed the manufacturer's recommendations, (2) hoses are inspected frequently and properly secured, (3) damaged hoses or couplings are replaced immediately, (4) replaced hoses are pressure-tested and hose fittings are compatible, and (5) applicable safety guards are properly installed and used.

Site Conditions

Specific hazards related to the siting and operations at the drill site should be included in the planning process and assessment (e.g., Moganti 2016). For example, when drilling near power lines or buried utilities, use caution and follow jurisdictional regulations. Typically, buried infrastructure or utilities should be located prior to the subsurface work. The drilling should maintain a minimum and predetermined standoff distance from these types of utilities. Contact with overhead power lines can result in electrical shock and electrical burns. Additional hazards may include extreme climate and/or terrain conditions (high latitude, desert, or mountainous regions), wildlife (e.g., venomous animals, biting insects, and predatory animals). Other constraints may include for example cultural, language, or security issues. Winter drilling programs require specific site assessments and safety plans. Drilling at old mine sites and hazardous waste dumps may expose personnel to toxic materials. In urban areas, drilling at night may need to consider noise restrictions. Guards or support staff may be needed to secure the drill site. A lack of experience of the site geologist and/or drill crew may increase the potential for injury when hazards are not foreseen and their levels of risk elevated and mitigated.

Summary

The hazards associated with drilling involve the subsurface, mechanical, and operational aspects that have economic, environmental, and safety consequences. The effects of these hazards have implications on the project completion, economic performance, and professional reputation. Managing the drilling hazards is an important aspect of engineering and geological projects. The management often requires advanced technologies and predrilling surveys. When evaluating the success of drilling projects, it is necessary to view them in terms of timely completions, in a safe manner, using the available technology while minimizing overall costs.

Cross-References

▶ Boreholes
▶ Drilling
▶ Hazard
▶ Risk Assessment
▶ Wells

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Canadian Diamond Drilling Association: Safe work methods surface handbook. https://www.cdda.ca/product/safe-work-methods-surface-handbook/

British Drilling Association: BDA Health & Safety Manual for Land Drilling 2015. http://www.britishdrillingassociation.co.uk/Publications/BDA-Health-Safety-Manual-for-Land-Drilling-2015-A-Code-of-Safe-Drilling-Practice-Free-for-BDA-members-10

National Groundwater Association: Environmental remediation drilling safety guideline. http://www.ngwa.org/Documents/erdsg.pdf
Government of Western Australian: Guidance about exploration drilling hazards. http://www.dmp.wa.gov.au/Safety/Guidance-about-exploration-6803.aspx
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Prospectors and Developers Association of Canada: E3 Plus: Framework for responsible exploration. http://www.pdac.ca/docs/default-source/priorities/e3-plus—toolkits—health-and-safety/drilling.pdf?sfvrsn=7e281e6d_4
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Durability

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Synonyms

Resistance to deterioration or wear

Definition

Durability can be defined as the resistance of geomaterials to deterioration caused by physical, chemical, and biological agents acting in a specific environment. Resistant materials maintain their original and distinctive characteristics and appearance over a period of time.

Characteristics

Geomaterials such as natural stones in buildings and historic monuments, concrete aggregate, and road aggregate can deteriorate and disintegrate at different rates when exposed to weathering agents. The decay rate depends on the mineralogical composition and the physical and mechanical properties of rock materials. Geotechnical characteristics are closely related to their geological origins and degree of weathering.

Durability is the capacity of a geomaterial to resist either to weathering processes or the decay caused by anthropogenic activities in a given period of time. Durability is a time-based concept in which a rock can preserve its original features, such as the mineralogical composition, structure, texture, shape, and grain size of mineral constituents, cementing materials, fracturing degree, and mechanical properties.

Rocks are exposed to the action of several weathering agents, which cause their decay. The most important agents are the atmosphere, rainwater, and capillarity phenomena of groundwater, mainly in the case of dissolved salts (Winkler 1997). Also important for rock deterioration are temperature and pressure variation, atmospheric pollution and biological activity of bacteria, as well as mechanical and chemical actions caused by plants and animals.

Since durability is not a fundamental property, it cannot be assessed in the laboratory by using a single and simple test method. An adequate assessment requires a deep understanding of the rock material properties and behaviour, as well as an understanding of the environment in which the rock is located (Přikryl 2013). Several tests have been proposed to evaluate durability, always with the purpose of creating a simple way to quantify and predict durability based on easily measurable parameters. For durability assessment, several approaches have been adopted, such as (a) accelerated laboratory standard durability tests (freeze–thaw cycling, wetting–drying durability, salt crystallization resistance, thermal cycling), (b) complex testing in an environmental test room, (c) *in-situ* ageing tests by exposure in real environmental conditions, and (d) testing methods to measure structural, physical and mechanical parameters of rock to establish correlations with the results of standard durability tests (strength, porosity, or effective surface area characteristics and petrographical or mineralogical characteristics).

Standard durability tests, despite attractive approaches due to their simplicity and rapid assessment, have many limitations affecting their representativeness. This approach was criticized and new testing methods at different scales have been proposed, such as field exposure testing and the combination of standard freeze–thaw, moisture variation and salt crystallization tests. Despite these attempts, the possible differences of deterioration processes and the great variability of rock materials can make durability assessment difficult. A dynamic perspective of durability, referred by Fookes et al. (1988), according to which the durability assessment is based on the resilience rather than resistance. The resilience corresponds to the ability of geomaterials to admit modifications without collapsing, whereas resistance is the capacity to endure the action of physical and chemical stresses. A dynamic durability assessment is a more useful approach and takes into account a broader range of decay mechanisms at different scales (Viles 2013).

Cross-References

- ▶ Aggregate
- ▶ Mechanical Properties
- ▶ Strength

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Dynamic Compaction/Compression

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Definition

A class of soil improvement methods that involves application of repeated impulsive loading onto the ground surface.

Dynamic compaction (DC) was originally developed for densifying loose granular fills and its effectiveness for such materials is well documented. The most common method of applying impulsive loading is by dropping a disk-shaped heavy mass with a weight of between 10 and 40 tonnes and a radius of between 2 and 4 m, from a height of between 5 and 30 m (Lee and Gu 2004).

The primary mechanism causing densification are compressional (P-) waves generated by the impact of the falling weight on the ground. The passage of these waves causes a large, transient increase in effective stress, resulting in densification and plastic volumetric change of the soil (Gu and Lee 2002). The passage of shear (S-) waves causing cyclic shearing may also have a secondary effect, but this is likely to be much less significant, since the number of cycles due to impulsive loading is often quite limited. Liquefaction has also been cited as an improvement mechanism, but this is probably a mistaken belief since DC works equally well in dry as well as saturated sand. The depth of improvement is often limited to about 10 m in granular soils owing to the tendency of the compressional waves to disperse laterally as they propagate downwards.

A typical DC program consists of two to three passes, each pass comprising a regular grid of DC "footprints" spaced about 3 m to about 8 m apart (Mayne et al. 1984, Lee and Gu 2004). Each footprint is generated by repeated dropping of the weights until the ground surface settlement stabilizes. The footprints are not contiguous. However, improvement is likely to be contiguous at greater depths owing to lateral dispersion of the stress waves. The second pass may involve similar or lower levels of impulsive loadings in a similar grid of footprints interspersed between the first grid. This pass is meant primarily to improve regions at intermediate depths and between the footprints from the first pass. The third pass is usually a light leveling pass for the near-surface regions and to level out the ground surface.

DC is often most effective in granular soils. However, there have also been cases of its successful usage on unsaturated clayey soils. It is normally not considered to be applicable to saturated clayey soils since the low permeability of the soil would prevent moisture egress from the soil skeleton during compaction. Although there have been a few reported cases of its use in saturated clayey soils, with vertical drains, its effectiveness is likely to be highly dependent upon the permeability of the soil. Clay with very low permeability are unlikely to be improvable by DC. One important consideration in the use of DC is the vibration from the impacts and its possible effect on surrounding structures and on archaeological remains within the ground. For this reason, DC is not often used in the vicinity of sensitive sites.

Cross-References

- ▶ Compaction
- ▶ Compression
- ▶ Ground Preparation
- ▶ Soil Properties

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Earthquake

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Synonyms

Earth tremor; Temblor

Definition

Quake	Vibration of a medium
Earthquake	The intense shaking of the Earth's surface caused by seismic waves resulting from the sudden release of the stored elastic strain energy in the Earth's crust (or, sometimes, upper mantle), which are usually generated naturally but are sometimes induced by human activities.

Introduction

An earthquake is the shaking of the Earth's surface caused by seismic waves from sudden energy release in the inner Earth's crust. Generally, the shaking severity of the earthquake can range from barely felt to very violent. Due to past strong earthquakes, buildings have been extensively destroyed; nuclear waste has leaked from a nuclear power plant; co-seismic landslides have been triggered in mountain areas; and tsunamis have been triggered when the epicenter of a large earthquake is located offshore. These earthquake-induced disasters have caused a great number of casualties and loss of properties. To date, earthquakes are the second most destructive contributors of natural disaster for human beings.

Distribution of the Global Earthquakes

Most earthquakes are associated with boundaries between tectonic plates. But significant earthquakes also occur within plates (e.g., New Madrid 1811) and on so-called passive margins (e.g., Lisbon 1755 and Charleston 1886). Some earthquakes are also linked to isostatic uplift following deglaciation or volcanic activity. The global distribution of earthquakes occurs in zones called seismic belts. These are basically located at the borders between tectonic plates where there are strong seismo-tectonic processes. In the seismic belts, epicenters are closely spaced but are also scattered outside those belts (see Fig. 1). There are three main seismic belts: the Circum-Pacific seismic belt ("Ring of Fire"), Alpide belt, and the Oceanic Ridge belt. Most major tectonic earthquakes occur in the Circum-Pacific seismic belt (USGS 2015).

The depth of the earthquakes is often limited to tens of kilometers. Earthquakes that have a focal depth of less than 70 km are classified as shallow-focus earthquakes; earthquakes with a focal depth ranging from 70 to 300 km are commonly termed intermediate-depth earthquakes; earthquakes with a greater focal depth between 300 to 700 kilometers are classified as deep-focus earthquakes which generally occur in subduction zones (USGS 2005). About 90% of the world's earthquakes (USGS 2012a) and 81% of the world's largest earthquakes (USGS 2014) occur along the Circum-Pacific seismic belt. Five to six percent of earthquakes and 17% of the world's largest earthquakes have occurred in the Alpide belt which extends from Java to the northern Atlantic Ocean via the Himalayas and southern Europe (USGS 2013). The earthquakes in the Oceanic Ridge seismic belt are all shallow-focus earthquakes which usually have low magnitude and are generally distant from human populations.

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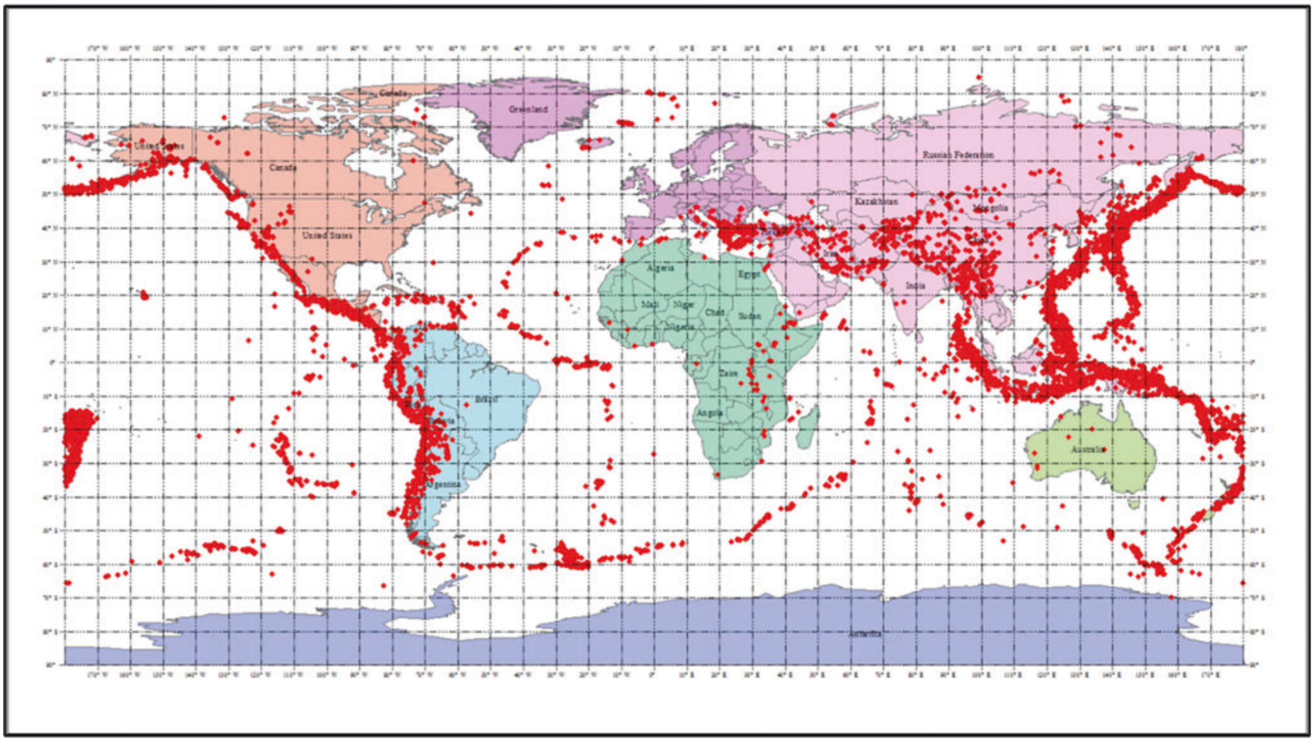


Earthquake, Fig. 1 Distribution of the global earthquakes ($M_L > 6$, Earthquake data from 1900 to 2015, from http://www.usgs.gov/)

Earthquake Classification and Induced Causes

An earthquake can be induced by both natural and anthropogenic forcing. On this basis, earthquakes are often classified into two categories: natural earthquakes and induced earthquakes. The number of the natural earthquakes is much greater than that of induced earthquakes. However, as human populations become larger, so do the impacts of natural earthquakes, and as large-scale human activities increase, so does the number of induced earthquakes attracting more attention from scientists worldwide.

Natural Earthquakes

It has been proved that natural earthquakes result from ruptures of faults mainly due to tectonic activity. Fault surfaces often have asperities and are initially locked. Under tectonic thrust, tectonic plates continue to move relatively leading to increased stress and, thus, stored strain energy in the fault system. When the stress is high enough to break through the asperity, the locked fault surfaces suddenly slide past each other and abruptly release the stored energy (Ohnaka 2013). This process leads to a form of stick-slip behavior. The energy is released into the rock masses in the form of radiated elastic strain seismic waves, frictional heating of the fault surface, and cracking of rock. This process of gradual build-up of strain and stress punctuated by occasional sudden failures and earthquake is referred to as the elastic-rebound theory (Reid 1910). It is estimated that only 10 percent or less of total energy produced by an earthquake is converted as radiated seismic energy. Most of the energy released by an earthquake contributes to powering the earthquake fracture growth or generating heat by friction. Therefore, earthquakes lower the Earth's available elastic potential energy and raise its temperature, though these changes are negligible compared to the conductive and convective flow of heat out from the Earth's deep interior (Spence et al. 1989).

In nature, there are three main types of faults, that is, normal, reverse (thrust), and strike-slip faults. It has been reported that all three types may cause earthquakes. The two walls of a fault can produce dip-slip or strike-slip motion depending on the orientation of the fault plane relative to the dip or strike of a succession. For a dip-slip type, the displacement along the fault is in the direction of dip with a vertical component movement. For a strike-slip type, the displacement along the fault is in the direction of strike with a horizontal component movement. Many earthquakes originate from a hybrid mode with both a dip-slip and strike-slip type, known as oblique slip. The three types of faults have a hierarchy of stress levels. Reverse faults have the highest stress levels, strike-slip faults intermediate, and normal faults the lowest (Schorlemmer et al. 2005). The difference in stress levels of the three faulting environments determines the differences in stress drop during faulting, and stress drop contributes to differences in radiated energy. For normal faults, the rock mass is pushed down in a vertical direction under the weight of the rock mass itself so the greatest principal stress equals the gravity of the upper walls. In the case of a thrust fault, the upper wall escapes in the direction of the least principal stress so the upper wall moves upward; thus the overburden equals the least principal stress. Strike-slip faulting lies in the intermediate state between the other two types described.

Induced Earthquakes

Human activities can produce induced earthquakes. With increased large-scale human activity over the past few decades, impacts on the Earth's environment have also increased. There are four main activities that may trigger earthquakes: reservoir filling behind a high dam, drilling and injecting liquid into wells, oil drilling, and mining subsidence (Madrigal et al. 2008). The first three activities can change the volume and pressure of liquid in the fault system. The increase of the pressure can probably increase the movement rate on a fault and strengthen the power of the earthquake (National Geographic 2009). In the mining process, millions of tons of rock are often removed by means of blasting (excavation). As a result, the stress level of the fault system changes reactivating faults, causing roof collapse, and inducing tremors (Trembath 2009).

Seismic Scale

Because different earthquakes usually have different magnitudes of released energy and effects on the Earth's surface, it is necessary to have seismic scales to calculate and compare the severity of earthquakes. There are two types of scales commonly used by seismologists to describe earthquakes. One is the magnitude scale which is used to describe the original force or release energy of an earthquake. The other is the intensity scale associated with describing the intensity of shaking occurring at any given point on the Earth's surface.

Magnitude Scale

The magnitude scale is used to describe the magnitude of the earthquake, which can be calculated from records of vibration waves away from the epicenter. Seismologists often assign a magnitude number to quantify the energy released by an earthquake. To date, there are more than 20 methods adopted to measure magnitude scale. Among them, the Richter magnitude scale $M_{L,}$ also called local magnitude scale, developed by the seismologists Charles Francis Richter and Beno Gutenberg (1935), is used worldwide.

The Richter magnitude is determined from the logarithm of the amplitude of waves recorded by seismographs, which can be calculated by the following formula (Ellsworth 1991):

$$\mathrm{M}_L = log_{10}^{A} - log_{10}^{A_0} = log_{10}[A/A_0]$$

where A(μm) is the maximum excursion of a Wood-Anderson seismograph located 100 km away from the epicenter and A_0(μm) is the maximum amplitude of the seismic wave of a magnitude 0 which is received by the seismograph away from the epicenter. Due to the limitation of the Wood-Anderson seismograph, the Richter magnitude is no longer applicable when the magnitude is larger than around 6.7 or the epicentral distance is larger than 600 km. Therefore, the surface wave magnitude M_s, the body wave magnitude M_b, and the moment magnitude scale M_w were introduced to make up for the limitation of the Richter magnitude.

Intensity Scale

The intensity scale is used for measuring the intensity of an earthquake and describing its effect on the ground surface and buildings. According to the degree of the damage of the building and the change of the ground surface, seismologists evaluate the earthquake intensity of different regions and draw intensity contours as descriptions of the damage level. For a specific region, the intensity scale depends on the magnitude of the earthquake, the focal depth and distance away from the epicenter, and also the engineering geology conditions of the site and the characteristics of the building. To date, numerous intensity scales have been developed and are used in different regions of the world. To take an example, the Mercalli intensity scale (USGS 2013) is selected to illustrate the scaling of the damage intensity for the earthquake. Table 1 shows the magnitude scale and corresponding modified Mercalli intensity scale. The average earthquake effects of different Mercalli intensities are also given.

Comparison Between the Two Seismic Scales

Although the two seismic scales are fundamentally different, they are equally important, and both are widely used by seismologists to describe an earthquake. The magnitude scale is usually expressed using an Arabic numeral to characterize the size of an earthquake via measuring indirectly the energy released. By contrast, intensity scale is usually expressed by a Roman numeral, which represents the severity of the shaking caused by an earthquake. The intensity value is determined based on the local effects and potential for damage produced by an earthquake on the Earth's surface. For a given earthquake, its release energy is unique, which can be only described by one magnitude. However, due to varied circumstances such as distance from the epicenter, local soil conditions, and hydrogeological conditions, different effects of the earthquake on the Earth's surface are involved. Thus different intensities may be calculated at different points for one earthquake. The two types of scale are essential inputs to hazard mapping.

Earthquake, Table 1 The Richter magnitude scale and the Mercalli intensity scale

Magnitude	Description	Mercalli intensity	Average earthquake effects	Average frequency of occurrence (estimated)
Less than 2.0	Micro	I	Microearthquakes, not felt, or felt rarely. Recorded by seismographs	Continual/several million per year
2.0–2.9	Minor	I to II	Felt slightly by some people. No damage to buildings	Over one million per year
3.0–3.9		II to IV	Often felt by people, but very rarely causes damage. Shaking of indoor objects can be noticeable	Over 100,000 per year
4.0–4.9	Light	IV to VI	Noticeable shaking of indoor objects and rattling noises. Felt by most people in the affected area. Slightly felt outside. Generally causes none to minimal damage. Moderate to significant damage very unlikely. Some objects may fall off shelves or be knocked over	10,000 to 15,000 per year
5.0–5.9	Moderate	VI to VIII	Can cause damage of varying severity to poorly constructed buildings. At most, none to slight damage to all other buildings. Felt by everyone	1000 to 1500 per year
6.0–6.9	Strong	VII to X	Damage to a moderate number of well-built structures in populated areas. Earthquake-resistant structures survive with slight to moderate damage. Poorly designed structures receive moderate to severe damage. Felt in wider areas, up to hundreds of miles/kilometers from the epicenter. Strong to violent shaking in epicentral area	100 to 150 per year
7.0–7.9	Major	VIII or greater	Causes damage to most buildings, some to partially or completely collapse or receive severe damage. Well-designed structures are likely to receive damage. Felt across great distances with major damage mostly limited to 250 km from epicenter	10 to 20 per year
8.0–8.9	Great		Major damage to buildings, structures likely to be destroyed. Will cause moderate to heavy damage to sturdy or earthquake-resistant buildings. Damaging in large areas. Felt in extremely large regions.	One per year
9.0 and greater			Near or total destruction – severe damage or collapse to all buildings. Heavy damage and shaking extend to distant locations. Permanent changes in ground topography	One per 10 to 50 years

Based on USGS (2012b)

The Effects of an Earthquake

As mentioned above, part of the energy released in an earthquake propagates into the rock mass in the form of a seismic wave. Arriving at the ground surface, the seismic waves induce ground motions. Thus, the ground surface deforms, which affects the stability of the rock mass, the soil mass, and the buildings and engineered structures and poses serious threats to people's lives and properties.

Shaking and Ground Rupture

Earthquakes mainly produce shaking and ground rupture that cause more or less severe damage to buildings and other engineered structures. Generally, the severity of the shaking and rupture depends on the combination of several factors, that is, the earthquake magnitude, the distance from the epicenter, and the local geological and geomorphological conditions.

Ground acceleration is taken as a measure of ground shaking. When propagating in different geological and geomorphological conditions, the seismic wave may be amplified or attenuated. Site conditions have a significant effect on the shaking and rupture. Even if the earthquake strength is low, for some special local geological, geomorphological, and geo-structural conditions, high-intensity shaking of ground surface can be still induced as a site or local amplification effect.

The earthquake can also tear the ground surface and produce ground rupture (see Figs. 2 and 3), which is a visible break and displacement on the Earth's surface along the trace of a fault. For a major earthquake, the size of the rupture can reach an order of several meters. Ground rupture is a major risk for large engineering structures such as dams, bridges, and nuclear power stations and requires careful mapping of existing faults to identify which are active faults and likely to break the ground surface within the life of the structure (USGS 2005).

Soil Liquefaction

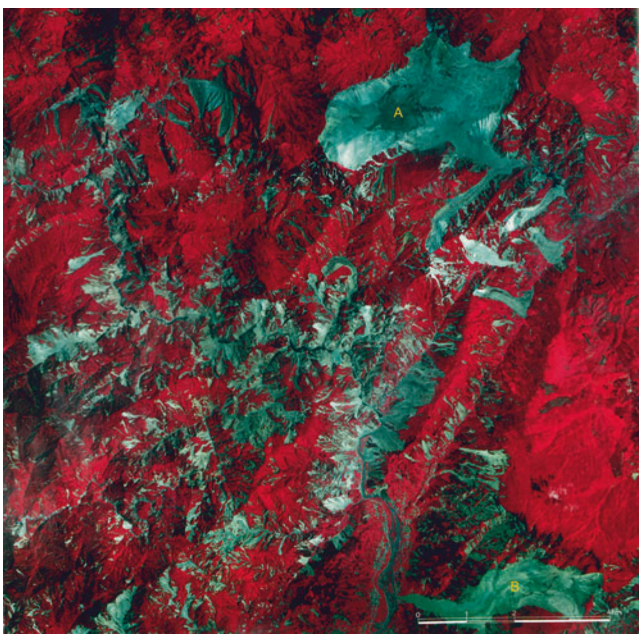
When the seismic waves propagate through saturated or partially saturated granular soil or sand in the shallow subsurface of the ground, the dynamic loading causes loose sand to gradually decrease in volume, whereas the pore water pressure increases, which consequently reduces the effective stress. When the effective stress of the soil is reduced to approximately zero, it loses its shear strength. As a result, the soil transforms from a solid state to liquid state causing soil liquefaction. Mobilization of the liquefied material gives rise to sand boils and waterspouts (see Fig. 4). Because the soil suddenly loses its strength and transforms into a liquid state, engineered structures on the soil such as buildings and bridges tilt, sink, and may finally collapse (see Fig. 5).

Earthquake, Fig. 2 Historic photographs taken in the aftermath of the San Francisco earthquake of 1906. (**a**) Offset of fence located ~1 km northwest of Woodville, California. View is northeast. Fence is offset in right-handed fashion by a distance of 2.6 m (Photograph taken by G. K. Gilbert. ID. Gilbert, G.K.2845 ggk02845. Courtesy of the US Geological Survey). (**b**) Offset of road and fence, with horse and buggy for scale. Road located between Upper and Lower Crystal Springs Reservoirs, currently Highway 92 (Photograph courtesy of Bancroft Library, University of California, Berkeley). (**c**) Train overturned by the earthquake at Point Reyes Station. This locomotive was standing on a siding when the April 18 earthquake pounded the region with seismic shockwaves (Photograph taken by G. K. Gilbert. ID. Gilbert, G. K. 3400 ggk03400. Courtesy of US Geological Survey) (From Davis and Reynolds (1996))

Earthquake, Fig. 3 Surface ruptures induced near the epicenter by the Yushu earthquake of April, 14, 2010 (Photograph provided by Yongshuan, Zhang, from the Chinese Academy of Geological Sciences; view is northwest)

Earthquake, Fig. 4 Sand boils and waterspouts located in the south of Gengzhuang Qiao, Ningjing County, during Xingtai earthquake that occurred on March 8, 1966, Ms 6.8 (From IGCEA (1983))

Earthquake, Fig. 5 Tilted apartment buildings at Kawagishi-cho, Niigata, Japan. The soils beneath these buildings liquefied during an earthquake in 1964 and provided little support for the building foundations (From http://geomaps.wr.usgs.gov/sfgeo/liquefaction/aboutliq.html#niigata)

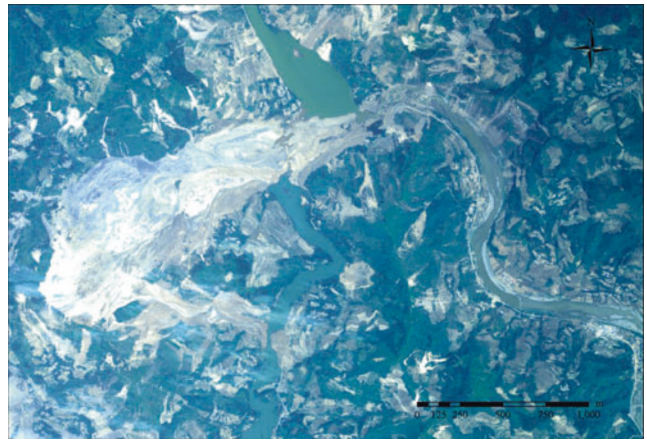


Earthquake, Fig. 6 Numerous landslides and rock falls triggered by the Wenchuan M_S 8.0 earthquake of May 12, 2008. (**a**) Daguangbao landslide; (**b**) Wenjiagou landslide (From Guo (2009))

Liquefaction is most likely to occur in loose to moderately saturated granular soils with poor drainage, such as silty sand or sand and gravel capped or containing seams of impermeable sediments, in both natural deposits or anthropogenic deposits in reclaimed land.

Severity of Damage

Generally, the severity of damage to the ground surface and built structures depends on the condition of the substrate ground under same seismic force, that is, the damage is least on bedrock, moderate on stiff soil, and most serious on soft soil. After the San



Earthquake, Fig. 7 Co-seismic landslides and dammed lakes in Donghekou, China, caused by Wenchuan earthquake of May 12, 2008

Francisco earthquake in 1906, it was found that the difference between the seismic intensities in different substrates can be as much as three levels. The depth of soft sediment has an obvious effect on the earthquake damage. As early as 1923, when a great earthquake happened in Kanto, Japan, it was observed that buildings on thicker alluvial deposits had more serious damage. Additionally, groundwater conditions have a significant effect on the seismic intensity. The saturation level of the soil mass influences the propagation velocity of the seismic wave, such that lower groundwater depth leads to greater seismic intensity. When the depth of the watertable ranges from 1.0 m to 5.0 m, the effect is most obvious gradually fading away when the depth is greater than 10.0 m (Li and Yang 1994).

Earthquake-Induced Landslides

As a dynamic load is suddenly imposed on slopes, seismic waves can produce slope instability resulting in earthquake-induced, or co-seismic, landslides. In recent decades, earthquake-induced landslides have become one of the most destructive geological hazards posing major threats to lives and properties. Sometimes, seismically induced landslides block rivers and form dammed lakes. For example, the Wenchuan earthquake that occurred on May 12, 2008, in China induced about 15,000 landslides and formed about 257 dammed lakes (see Figs. 6 and 7). Some of the resulting dams fail leading to flooding.

Before an earthquake, slopes may be stable or metastable. When the earthquake wave propagates into the slope, it produces accelerations of the rock and soil material, which significantly changes the gravitational load on the slope. The vertical seismic accelerations are applied to the slope upward, which decrease the normal downward load acting on the slope. On the other hand, the horizontal accelerations produce shear forces due to the inertia of the landslide mass. These processes induce slope failure and landsliding when the acceleration is high enough. In mountainous areas, the terrain has a significant effect on the acceleration distribution of the slope. Usually, the geomorphic effect increases the magnitude of the ground accelerations. Therefore, this process is usually much more serious in mountainous areas. This process can be termed topographic amplification. It has been found that the maximum acceleration usually appears at the crest of the slope or along the ridge line (He and Lu 1998). Thus, characteristically earthquake-induced failures occur at the top of slopes.

Similar to co-seismic landslides, earthquake-induced avalanches are a less common but dangerous type of catastrophic slope failure (Chernous et al. 2004). Many casualties have been caused by catastrophic avalanches when a snowpack with an unstable inner structure is disturbed by an earthquake (O'Leary and Rangers 1968) such as that which affected Mount Everest on April 25, 2015, that killed trekkers and climbers.

Tsunami

Tsunami is the rapid movement of large volumes of water due sometimes to earthquakes, which behave as long-wavelength and long-period sea waves. Ordinarily, subduction zone earthquakes less than magnitude 7.5 on the Richter scale do not cause tsunamis, although some instances of this have been recorded. Most destructive tsunamis are caused by earthquakes of magnitude 7.5 or more (Noson et al. 1988). The propagation velocity of the tsunami can reach 700–800 km/h. Generally, it only takes a few hours for the tsunami to propagate across the ocean with limited energy dissipation. Away from the coastline, the water wave initially has a long wavelength with a wave height often of less than 1 m. But, when it arrives at shallow areas near the coastline, the wavelength decreases whereas the height increases abruptly. In the large events, wave heights can be up to around 10 m forming a water wall with huge energy. The formation of the tsunami is mainly controlled by the submarine topography, the coastline geometry, and the characteristic of the wave. Tsunamis are generally made up of a series of waves with periods that range from minutes to hours. The global distribution zone of the tsunami is basically consistent with the seismic zone. To date, about 200 destructive tsunamis have been recorded globally. About 80% occurred in the Circum-Pacific seismic belt. These powerful tsunamis often impact the coastal area, destroy embankments, and flood the land. As a result, they cause a large number of casualties and major losses of properties. The destructive power of a tsunami is enormous, and a large event can affect parts of an entire ocean basin. It has been reported that there were at least 230,000 people killed in the 2004 Indian Ocean tsunami which affected 14 countries: one of the deadliest natural disasters in human history.

Measuring and Locating Earthquakes

Seismic waves produced by the rupture of the fault propagate into the Earth's interior, which can be recorded by seismometers installed in the monitoring stations. Generally, monitoring can be undertaken at a great distance. Earthquakes produce three different types of seismic waves with different propagation velocities, that is, longitudinal P-waves (shock or pressure waves), transverse SV and SH-waves (both body waves), and surface waves (Rayleigh and Love waves).

According to the density and velocity of the Earth's medium, it is estimated that the propagation velocity of the seismic waves ranges from 3 km/s up to 13 km/s. P-waves propagate much faster than the S-waves in the Earth's interior, with the ratio of P-wave velocity to the S-wave velocity at 1.67. The Rayleigh and Love waves travel near the ground surface. The propagation velocity of the Rayleigh wave is slightly less than the S-wave, which ranges from 2 km/s to 5 km/s. Love waves travel with a lower velocity than P or S-waves, but faster than Rayleigh waves. Figure 8 shows the representative seismograms for a distant earthquake.

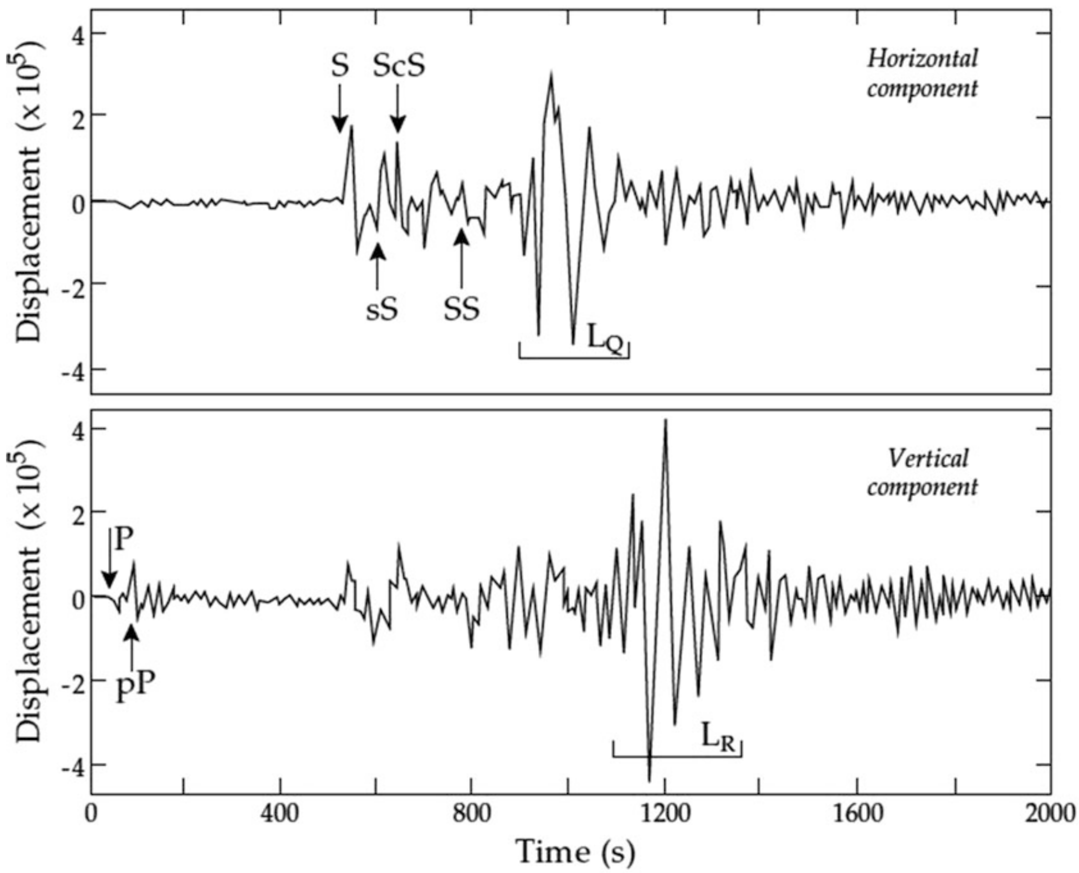
Making full use of the differences in travel time from the epicenter to the seismic stations, the distance from epicenter and the seismic stations can be measured. Meanwhile, these differences can usually be used to image both sources of earthquakes and structures within the Earth. Also, the depth of the hypocenter can be computed roughly.

Based on the recorded seismic waves and the distance from the epicenter and the seismic stations, the magnitude scale of the earthquake can be calculated. The locations where earthquakes occur can be also determined. Standard reporting of earthquakes includes the magnitude, date and time of occurrence, geographic coordinates of the epicenter, depth of the epicenter, geographical region, distances to population centers, location uncertainty, a number of parameters that are included in USGS earthquake reports (number of stations reporting, number of observations, etc.), and a unique event ID (Geographic Org 2013).

Prediction and Preparedness

Prediction of the times and places in which earthquakes occur is the most challenging work for seismologists. Until now, scientifically reproducible predictions cannot yet be made to a specific time despite considerable research efforts by seismologists (Ruth 2001). However, it is likely that the probability of a fault segment rupture might be established, during the next few decades, for well-understood faults (USGS 2003).

Although it is difficult to predict the occurrence time and place of the earthquake, preparations should be made to reduce or relieve earthquake damage. Establishment of earthquake warning systems is needed for geological disaster protection and prediction, particularly for the major engineering structures such as high dam hydroelectric and nuclear power stations, subways, or railway tunnels. Earthquake engineering measures should also be taken to predict the effect of shaking on buildings



Earthquake, Fig. 8 Broadband seismograms of an earthquake in Peru recorded at Harvard, Massachusetts. (*Top*) the SH body wave and Love (LQ) surface wave are prominent on the horizontal component record. (*Bottom*) the P and SV body waves and the Rayleigh (LR) surface waves are clear on the vertical component record (Lowrie 2007)

and other engineering structures. On the other hand, earthquake engineering aims to design such structures to minimize the risk of damage. Furthermore, existing structures can be modified by seismic retrofitting to improve their resistance to earthquakes.

Summary

As a frequent phenomenon, an earthquake is the tremor of the ground surface caused by the seismic waves produced by the sudden rupture of faults. There are three types of faults producing earthquakes, that is, the normal fault, the strike-slip fault, and the reverse (thrust) fault. Different types of faults can induce earthquakes with different intensities. The earthquake can be triggered by the natural forcing or by anthropogenic forcing. Two types of scales are applied to describe the intensity of an earthquake. One is the magnitude scale which is used to measure the energy release of the fault systems; the other is the intensity scale which is used to describe the effect of an earthquake on the ground surface and buildings. The global distribution of earthquakes mainly occurs in three types of belt, that is, Circum-Pacific seismic belt ("Ring of Fire"), Alpide belt, and the Oceanic ridge seismic belt. Strong earthquakes can result in intensive shaking and rupture of the ground surface, soil liquefaction, the collapse of buildings and engineering structures, landslides, and tsunami which often cause losses of human life and properties. Prediction of the times and places in which earthquakes occur is still the most challenging work, and an earthquake warning system should be established and the anti-seismic measures should be strengthened to reduce or relieve earthquake damage.

Cross-References

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Earthquake Intensity

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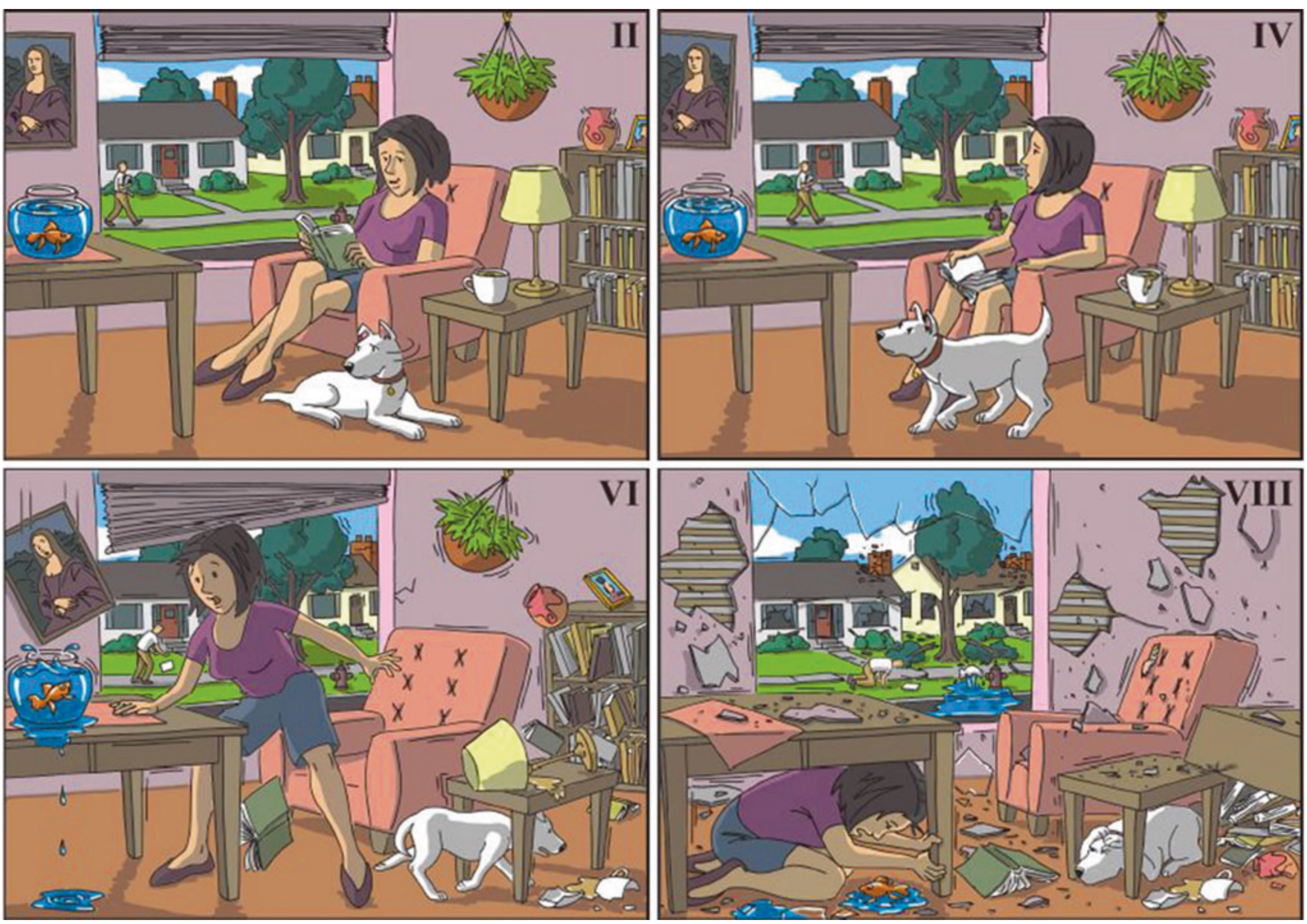
Definition

The severity and effects of ground shaking, for a given earthquake at a specific location, on humans, man-made structures and the natural environment, for instance, how the vibrations were felt by people (e.g., not felt, light, strong, intense), and the impact on the contents and components of buildings.

Worldwide, the principal earthquake intensity scales are the Modified Mercalli (Wood and Neumann 1931), Japan Meteorological Agency (Japan), and European Macroseismic (Grünthal 1998) scales. All intensity scales are "bounded" with a set range – for example, the Modified Mercalli (MMI) scale uses Roman numerals from I to XII (see Fig. 1). As examples, MMI II indicates: "Felt only by a few people"; MMI IV indicates "Felt indoors by many, outdoors by few, frightened no one," MMI VI indicates "Strong shaking, Felt by all, indoors and outdoors, a few instances of fallen plaster," MMI VIII indicates "Severe shaking, fall of chimneys, walls, . . . heavy furniture overturned." Traditionally, seismologists relied on newspaper accounts or returned mail questionnaires to estimate the impact of earthquakes at different localities. Since the early 2000s, citizens have been encouraged to answer questions online (e.g., The United States Geological Survey's "Did You Feel it?" questionnaire; Wald et al. 1999). In these questionnaires, individuals choose the description that best corresponds to the felt effects and observed impacts of the shaking where they were at the time of the earthquake. Based on these answers, a computer program rates the reported earthquake impact on the MMI scale. By plotting each MMI report on what is called an isoseismal map, one can determine the area where the earthquake was felt (called the "felt area"), as well as the areal extent of damage at a certain intensity level. The correspondence that exists between the MMIs and the levels of ground motions



Earthquake Intensity, Fig. 1 Cartoons that illustrate the impact that correspond to progressively higher Modified Mercalli Intensities (MMI) II, IV, VI, and VIII (velocity, acceleration) allows one to use recorded vibrations to infer intensities and vice versa.

Because the intensity scale does not rely on instruments, the effects and sizes of earthquakes that may have occurred hundreds (or even thousands) of years ago can be estimated if there is sufficient surviving documentation. From either the felt area, the damaged area, or the maximum intensity, the magnitude and locations of preinstrumental earthquakes can be estimated. Great care must be taken in converting one into the other: intensities depend on a number of factors, some due to the earthquake itself, such as the magnitude, focal depth, directivity of ground motions, while some are due to conditions where the observer was at the time of the earthquake, such as distance from the epicenter, local geology, topography, building type and condition, and on the individual (e.g., at rest, moving, in a car).

Earthquake intensity scales (as described above) have applications for evaluating geological hazards. For example, empirical relationships have been developed between MMI and landslide potential as a function of distance (e.g., Keefer 2002). There are also important engineering geology applications from "intensity measures" that are based on instrumental recordings. As one example, Arias intensity and other "intensity scales" (Kramer and Mitchell 2006 and references therein) utilize the strength and duration of ground shaking to evaluate liquefaction potential.

Cross-References

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Earthquake Magnitude

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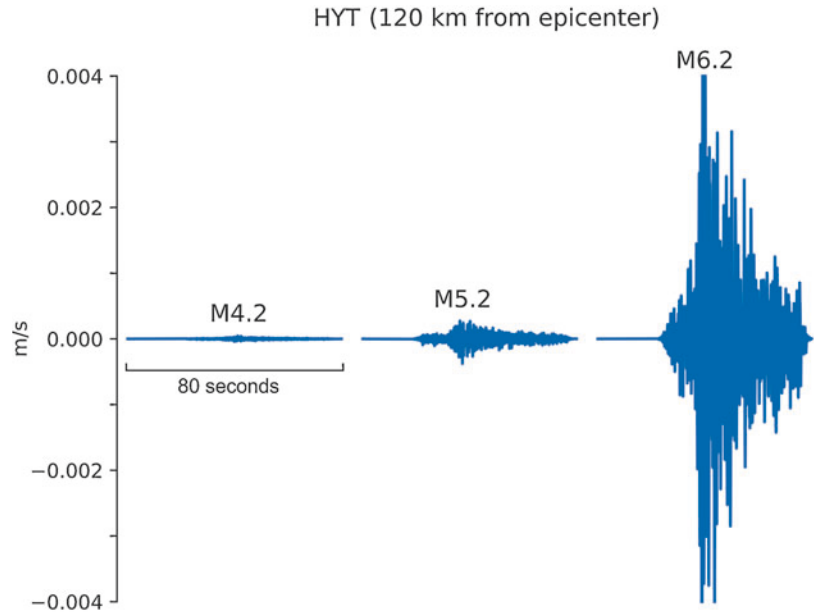
Definition

Earthquake magnitude (M) describes the energy release (or size) of an earthquake.



Earthquake Magnitude, Fig. 1 Seismic recordings illustrating the change in ground shaking as a function of earthquake magnitude. All traces are of the same duration (80 s) and plotted at the same amplitude scale

There are many types of earthquake magnitude scales, with the vast majority based on recorded seismic waveforms (Bormann 2002). Magnitude scales are logarithmic, correct for distance from the earthquake, and are unbounded (the smallest earthquakes are less than zero, and the largest recorded event to date is the 1960 M9.5, Chile earthquake).

Some magnitude scales are based on measurements of shorter-period body waves (primary (P) or secondary (S)-waves), and some are based on longer-period surface waves. The original (and most famous) magnitude scale, the "Richter scale" was developed in the 1930s for California earthquakes (Richter 1935). The modern and most commonly used magnitude scale is moment magnitude, Mw (Hanks and Kanamori 1979). It is based on seismic moment (Mo) release, which directly relates to the fault rupture area and amount of slip.

It is important to note that each unit increase in magnitude represents a 10-fold increase in amplitude of shaking (Fig. 1) and a 32-fold increase in energy release. For example, a M7 earthquake releases ~1000 times as much energy as a M5 earthquake and has shaking that is 100 times stronger. Earthquakes can generally be felt starting at M 2–3. Earthquakes may cause minor damage starting at M ~4–5, and earthquakes of M7 or larger are considered major and can be felt (and have the potential to cause damage) up to 100's of km away. Small earthquakes are much more frequent than large earthquakes – for example, there are, on average, about 1.3 million M2–2.9 earthquakes around the world each year, compared to 15–20 M 7–7.9 events.

Scientists are limited to the instrumental recording period (since the late 1800s) for the accurate estimation of earthquake magnitude. Prior to that time, they rely on written and oral reports that describe the earthquake's "intensity." Intensity describes the effects of an earthquake on humans or the environment and requires no instrumental records. Earthquake magnitude has important applications for engineering geology, including numerous empirical relationships developed between magnitude and potential for triggering of landslides and liquefaction. Specifically, a historical (and global) review of earthquake-triggered landslides (including rockfalls, delayed-initiation landslides, lateral spreads and flows) as a function of earthquake magnitude is provided by Keefer (2002). For example, an M7 earthquake can be expected to generate lateral flows at distances of ~80 km and landslides at distances of ~170 km. A relationship between areas of potential liquefaction and earthquake magnitude is provided by Wang et al. (2006). Based on their work (and references therein), an M7 earthquake may cause liquefaction to hypocentral distances of ~160 km.

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- Shear Strength
- Shear Stress
- Soil Laboratory Tests
- Soil Mechanics
- Soil Properties
- Strength
- Surface Rupture
- Tension Cracks
- Tsunamis

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Effective Stress

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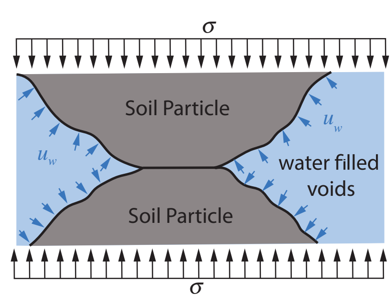
Definition

The (total) stress (σ) applied to a dry soil is transmitted through the contacts of the soil particles that compose the structure. When the voids between the soil particles are filled or partially filled with water then the water will transmit a portion of total stress (Fig. 1). The amount of stress transmitted by the pore water is equal to the **pore water pressure** (u_w) (Fig. 1). The stress transmitted between the soil particles is the effective stress (σ′) (Terzaghi 1920; Bishop 1960; Skempton 1961).

$$\sigma' = \sigma - u_w$$

The deformation of the soil structure is a result of the stress imposed on the structure, and this stress is σ′. The **shear strength** of soils is predominantly a result of interparticle friction, and the frictional strength that can be mobilized between these particles is a result of stresses carried by the soil particles, and is thus defined by σ′.



Effective Stress, Fig. 1 Definition of the total stress (σ) and pore pressure (u_w) which are used to calculate the effective stress

Cross-References

- ▶ Hydraulic Action
- ▶ Pore Pressure
- ▶ Shear Strength

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Elasticity

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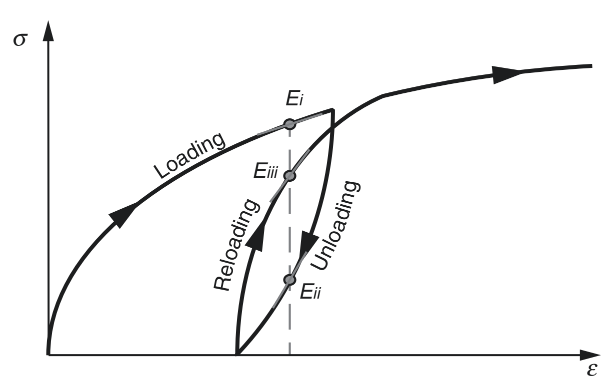
Definition

Elasticity is the ability of a material to deform under an applied load, such that the resulting deformation is recoverable (*elastic*) once the load is removed.

Introduction

The following is a presentation of the mathematical formulation for elasticity, a contrast between elastic and plastic deformation, and the application of elasticity to rock and soils (Fig. 1).



Elasticity, Fig. 1 Moduli (Ei, Eii, and Eiii) evaluated at the same strain for different portions of an unload and reload cycle and thus differing stress history

Elasticity of Rock and Soil

The earliest formulation of a mathematical description of elasticity resulted from experiments conducted by Hooke and published in Hooke (1675). This formulation stated that the deformation of a body is directly proportional to the applied loading. More contemporary applications of these results are presented as Hooke's law, representing it in terms of stress (σ), strain (ε), and Young's modulus (E) (Love 1906; Wood 1990).

$$\Delta\sigma = E\Delta\varepsilon$$

Within a continuum, both σ and ε may be represented as tensors such that they vary with spatial orientation. When σ and ε are tensors, then E is replaced with a compliance matrix. The simplest formulation for an isotropic material is presented below. Where γ is the shear strain, τ is the shear stress, and ν is the Poisson's ratio (Love 1906).

$$\begin{bmatrix} \delta\varepsilon_{xx} \\ \delta\varepsilon_{yy} \\ \delta\varepsilon_{zz} \\ \delta\gamma_{yz} \\ \delta\gamma_{zx} \\ \delta\gamma_{xy} \end{bmatrix} = \mathrm{E}^{-1} \begin{bmatrix} 1 & -\nu & -\nu & 0 & 0 & 0 \\ -\nu & 1 & -\nu & 0 & 0 & 0 \\ -\nu & -\nu & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 2(1+\nu) & 0 & 0 \\ 0 & 0 & 0 & 0 & 2(1+\nu) & 0 \\ 0 & 0 & 0 & 0 & 0 & 2(1+\nu) \end{bmatrix} \begin{bmatrix} \delta\sigma_{xx} \\ \delta\sigma_{yy} \\ \delta\sigma_{zz} \\ \delta\tau_{yz} \\ \delta\tau_{zx} \\ \delta\tau_{xy} \end{bmatrix}$$

Elasticity is limited in the representation of deformation of a material. Elastic strain often occurs concurrently with non-recoverable (*plastic*) strain. Typically, the proportion of strain that is plastic is small at lower strains and increases with increasing strain. Thus, the representation of a material as solely elastic is more realistic at relatively small strains (Wood 1990, Terzaghi et al. 1996).

E is a result of the history of stresses that the material has been subjected. The reloading of a material through stress states that it has previously been subjected to will be governed by an E that is often significantly different than E observed during the first loading the material through this stress state and potentially from other loading cycles that may have occurred (Wood 1990, Terzaghi et al. 1996).

Elastic models are commonly used in the estimation of the deformation behavior of soil and rock. Moduli for these materials are strongly related to stress history. The stress state of both soil and rock is often defined by **effective stresses** (σ'), the same deformation behavior can be interpreted to be a result of either σ and σ', and, thus, this results in different Young's moduli with E relating to change in total stress and E' relating to the effective stress (Wood 1990, Terzaghi et al. 1996).

The stress-strain response of soils is nonlinear for all but *very* small strain, and analyses conducted with linear elasticity require significant judgment in the selection of moduli and in the interpretation of the results. The differentiation between plastic and elastic strain may not be necessary for the calculation deformation under monotonic loading, and the use of a non-linear elastic model may provide reasonable results.

For the interpretation of soil and rock behaviors, it is often useful to divide the modulus of the material into a shear modulus (G) and a bulk modulus (K); both may be represented as a function of E and ν. G relates shear stress to shear strain, and K relates the compressive stress to the volumetric strain. As the pore water is unable to resist shear, the whole of the shear stress is carried by the soil particle interactions; thus, G is the same whether interpreted in terms of σ or σ'. Alternatively, K is limited to the change in volume of the voids within the soil, which is in turn governed by the ability of the pore water to drain from that space. For conditions where the water is not able to drain, then K is effectively infinite; where the water is allowed to drain, then K is a result of the stresses on the structure of the soil particles and thus relates the effective compressive stress to the volumetric strain and is commonly referred to as the drained bulk modulus (K') (Wood 1990; Terzaghi et al. 1996).

Summary

Elasticity is the ability of a material to deform under an applied load, such that the resulting deformation is recoverable once the load is removed. This is in contrast to plastic deformation which is not recoverable. Mathematical descriptions are based on the magnitude of deformation being directly proportional to the applied loading. Elasticity of rock and soils is often defined in terms of effective stress and divided into a shear and volumetric components.

Cross-References

► Consolidation
► Deformation
► Poisson's Ratio
► Rock Properties
► Soil Properties
► Strain
► Stress
► Young's Modulus

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Engineering Geological Maps

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Synonyms

Engineering geological models; Environmental geological maps; Geohazard maps; Geotechnical maps; Urban geological maps

Definition

Many people, when asked to describe a "map," would probably refer to a topographical one that is essentially factual, two dimensional, and, depending upon the age of the person asked, either digital or printed on paper. Geological maps, in general, and engineering geological maps, in particular, are far more varied, covering a range of topics, being interpretative and factual and, increasingly, digital.

Until about the 1990s, engineering geology was often understood to be the application of geology to civil engineering design and construction. However, the definition of what engineering geology covers has broadened to include all parts of the development process from the identification of land for a wide range of engineering, environmental, and conservation uses, through the process of obtaining permission to use the land for a specific purpose to building and construction on, or in, the ground. As a result, engineering geological maps have increased in their scope to cover all aspects of the gathering and spatial presentation of geological information for development, construction, regeneration, and conservation.

An engineering geological map was defined by Commission No. 1 of the International Association of Engineering Geology as: "...a type of geological map which provided a generalized representation of all those components of a geological environment of significance in land-use planning, and in design, construction and maintenance as applied to civil and mining engineering." (Anon. 1976).

More recently, González de Vallejo and Ferrer (2011) said that engineering geological maps present geological and geotechnical information for land-use planning, development, regeneration and conservation, and to plan, construct, and maintain buildings, engineering structures, and infrastructure.

They often provide data on the characteristics and properties of the artificial ground and the natural soils and rocks of a specific area to enable its behavior to be evaluated and to forecast geological and geotechnical problems.

However, as computing power increases, the two-dimensional map, which represents a three-dimensional object, has been replaced by two and a half and true three- and four-dimensional models that show change with time.

Historical Introduction

While William Smith (1769–1839) is regarded by many as the first geologist to produce stratigraphical geological maps as well as the first engineering geologist (Terzaghi 1948; Forster and Reeves 2008), the maps that he did produce were not engineering geological ones; rather, he applied what we would now regard as basic geological principles to solve a number of engineering construction problems. It was not until Henry Penning (1838–1902) published a short text book on engineering geology (Penning 1872) that the subdiscipline was formally recognized. This book described the principles of engineering geology and also how to produce geological maps – but not *engineering* geological maps.

One of the earliest engineering geological maps was produced by Woodward (1897) and republished with modifications 9 years later (Woodward 1906). The map of London, UK, and the surrounding area was at a scale of approximately 1:253 440 (4 miles to 1 inch). Culshaw (2004) observed that what distinguished it from the conventional stratigraphical maps of the time was that geological units from the Upper Cretaceous to the Holocene were grouped by lithology into three "series":– sandy series, gravelly series, and clayey series. Geotechnical, hydrogeological, geoenvironmental, and geohazard conditions for members of each series are discussed in the memoirs.

In the twentieth century, engineering geological maps began to appear more frequently, particularly in central and Eastern Europe. These maps related to engineering, land-use planning, and geohazards (Dearman 1991). The II World War also saw the development and use of engineering geological mapping to identify, for example, landing sites, groundwater resources, and potential grass strip airfields for the D-Day landings (Rose and Clatworthy 2008).

In the 1960s and 1970s, engineering geology began to develop as a science and it deemed helpful to codify engineering geological maps. This was achieved mainly by working parties of the Engineering Group of the Geological Society of London (EGGS) (Anon. 1972) and the Commission on Engineering Geological Mapping (Commission No. 1) of the International Association of Engineering Geology (IAEG) (Anon. 1976). These two publications sought to provide guidance to different types of engineering geological maps and how to make them. Their recommendations were constrained by the fact that almost all engineering geological maps at this time were produced on paper and, therefore, were limited in terms of the scale of reproduction by the scale of topographical base maps. Both reports focused mainly on maps for use as part of the site investigation process prior to engineering construction.

The methodologies developed by Anon. (1972, 1976) were put into practice across the world. Not surprisingly, Dearman and coworkers were active in the UK (e.g., a major engineering geological mapping and geotechnical databasing project in north-east England [Dearman et al. 1979]). In Brazil, Zuquette and Gandolfi (1990) developed the mapping methodology for application in the different geological conditions there. In 1979, the IAEG held a symposium in Newcastle upon Tyne, UK, to "discuss the methods, application and usefulness of mapping in engineering geological terms to planning, design and construction in civil engineering" (Anon. 1979). The Proceedings of the Symposium were published in Volumes 19 (1979) and 21 (1980) of the Bulletin of the International Association of Engineering Geology (papers from Sessions 1–4) and in volume 12, Part 3 (1979) of the Quarterly Journal of Engineering Geology (papers from Sessions 5–6). A wide range of papers discussed eight aspects of engineering geological mapping:

- Regional maps for planning purposes
- Hazard mapping in risk evaluation for engineering structures
- Civil engineering site mapping practice
- Hydrogeological mapping at the engineering site scale
- Land and sea floor geophysical mapping for engineering structures
- Use of computers in mapping
- Terrain evaluation and remote sensing
- Engineering geomorphological mapping

Looking back what stands out from this highly active period in the development of engineering geological mapping is the relative crudeness of some of the maps and the recognition of the value of computer techniques.

The IAEG Engineering Geological Mapping Commission No. 1 also set itself the task of producing four further reports relevant to engineering geological mapping (Dearman et al. 1979):

- Semiquantitative classifications
- Symbols and patterns
- Environmental aspects
- Use of computer techniques in preparation of engineering geological databanks (databases) and maps

Two reports on classification were published (Matula et al. 1979; Matula et al. 1981a) and one on symbols (Matula et al.

Engineering Geological Maps, Table 1 Different approaches to the ordering of engineering geological descriptive terms

Ordering of engineering geological descriptive terms	
Matula et al. (1979)	Anon. (1972)
Rock name	**Rocks**
Mineral composition Texture (grain size) Color Weathered state Degree of jointing Relative density Consistency Strength Deformability Permeability Durability	Color Grain size Texture and structure Discontinuities within the mass Weathered state Alteration state Minor lithological characteristics
	Rock name Estimated mechanical strength of the rock material Estimate of mass permeability Other terms indicating special engineering characteristics
	Soils Color *In situ* Strength and structure (including discontinuities) Weathered state Alteration state Minor lithological characteristics and additional descriptive terms
	Soil name Estimated mass behavior to ground water flow Other terms indicating special engineering characteristics

1981b). Some papers on environmental aspects (Golodkovskaja 1979; Radbruck-Hall 1979) and computer techniques (Radbruck-Hall et al. 1979) were also produced but no formal reports.

Of particular importance was the publication in the Symposium Proceedings of a paper by Commission No. 1 of the IAEG on "Classification of rocks and soils for engineering geological mapping. Part 1: Rock and soil materials" (Matula et al. 1979). An ordering for terms in an engineering geological soil or rock description was established. The order differs from that suggested by Anon. (1972). The two are compared in Table 1. Whereas Anon. (1972) gives examples of soil and rock descriptions, Matula et al. (1979) do not. The various IAEG Commission No. 1 reports also recommended classification tables for:

- Soil and rock grain size
- Grain shape
- Color (following the Rock Color Chart of the Geological Society of America)
- Degree of weathering of rock material
- Weathering grades for the rock and soil mass
- Discontinuity spacing
- Roughness of discontinuity surfaces
- Aperture of discontinuity surfaces
- Rock mass block shape
- Rock block size
- Grading chart for soils
- Relative density of sand and gravel
- Definition of sand, gravel, cobble, and boulder composite types and for soils in general
- Consistency of cohesive soils
- Undrained shear strength of soils
- SPT "N" values
- Classification of rocks in terms of RQD and velocity index
- Strength of rocks
- Deformability of rocks (in terms of deformation modulus and modulus of compressibility)
- Permeability
- Unit weight for soils and rocks
- Porosity for soils and rocks
- Degree of saturation
- Plasticity of soils in terms of liquid limit and plasticity index
- Sonic velocity of soils and rocks

Many of these classifications have been superseded, for example, by the International Society for Rock Mechanics' series of books on "Suggested Methods for Rock Characterisation." The latest, the "Orange Book," updates previous versions and is for the period 2007–2014 (Ulusay 2015).

Maps for Engineering Construction

Anon. (1972, 1976) considered engineering geological maps mostly from a civil engineering perspective. These two extensive publications set the global standard for engineering geological mapping. Bill Dearman was Chair of the EGGS Working Party and Editor of the IAEG Commission No. 1 report and so strongly influenced both. At that time, engineering geology was defined in Dearman's book on engineering geological mapping as: "...the discipline of geology applied to civil engineering, particularly to the design, construction and performance of engineering structures interacting with the ground in, for example, foundations, cuttings and other surface excavations, and tunnels." (Dearman 1991). The book was published at a key time for engineering geology, in that the scope and definition of engineering geology was changing and the rapid development of computer software in relation to two and three-dimensional representation meant that within a few years of the publication of the book, engineering geological maps would no longer be constrained by the necessity of being printed on paper. These changes are considered further below.

Anon. (1972) only classified engineering geological maps in terms of scale:

Engineering Geological Maps, Table 2 Classification of engineering geological maps according to their scale (After González de Vallejo and Ferrer 2011)

Type and scale	Content	Mapping method	Applications
Regional <1:10 000	Geological data, lithological groups, tectonic structures, regional morphological features, large areas affected by processes. General information and interpretations of geotechnical interest	Aerial photography, previous topographical and geological maps, existing information and field observations	Preliminary studies and planning, general information on the region, and types of material and geomorphological processes present
Local Desk study phase 1:10 000 to 1:500	Description and classification of soils and rocks, structures, morphology, hydrogeological conditions, geodynamic processes, location of possible construction materials	Aerial photography, ground trothing field surveys, measurements and other field data	Planning and viability of works and detailed site reconnaissance. Basic design
Local Ground investigation phase 1:5000 to 1:500	Material properties, geotechnical conditions and other aspects important for the carrying out of a specific construction project	All previous data plus data from boreholes and trial pits, geophysical methods, *in situ* and laboratory tests	Detailed information on sites and geological-geotechnical problems. Detailed design and analyses

- Regional engineering geological maps at a scale of 1:10,000 or smaller
- Local engineering geological plans at a scale larger than 1:10,000
- 1:5000 as a general scale for many purposes, such as extended or compact sites, small reservoirs, large dam sites, borrow areas, tunnels, airfields, parts of transport systems, port facilities
- 1:1250 for more detailed representation of important compact sites such as bridges, tunnel portals, dam sites, parts of linear transport systems, large buildings
- 1:500 to 1:100 or larger for recording the details of trenches, pits, and other excavations

González de Vallejo and Ferrer (2011) revised these subdivisions as shown in Table 2.

However, Anon. (1976) classified engineering geological maps in terms of *purpose* and *content* as well as *scale*. The classification is summarized in Table 3.

Since the development of digital mapping, this type of rather rigid classification of engineering geological maps has largely become irrelevant and such classification terms are little used today. In particular, the idea of scale has become rather flexible in that digital maps can be increased or decreased in scale quite easily. However, this can create its own problems if users expand a map to a larger scale without fully appreciating the potential implications.

Engineering Geological Maps, Table 3 Classification of engineering geological maps (After González de Vallejo and Ferrer 2011 from Anon. (1976), Hrašna and Vlčko (1994), Proske et al. (2005))

Criterion	Type
Purpose	*Special*: Providing information either on one specific aspect of engineering geology or for one specific purpose
	Multipurpose: Providing information covering many aspects of engineering geology for a variety of planning and engineering purposes
Content	*Analytical*: Giving details of, or evaluating, individual components of the geological environment. Their content is, as a rule, expressed in the title, for example, map of weathering grades; jointing map; seismic hazard map
	Comprehensive: Two kinds – maps of engineering geological conditions depicting all the principal components of the engineering geological environment or maps of engineering geological zoning, evaluating and classifying individual territorial units on the basis of uniformity of their engineering geological conditions. The two types may be combined on small-scale maps
	Auxiliary: Presenting factual data and are, for example, documentation maps, structural contour maps, isopachyte maps
	Complementary: Geological, tectonic, geomorphological, pedological, geophysical, and hydrogeological maps. They are maps of basic data that are sometimes included with a set of engineering geological maps
Scale	*Large-scale*: 1:10,000 and greater
	Medium-scale: Less than 1:10,000 and greater than 1:100,000
	Small-scale: 1:100,000 and less

Description and Classification of Rocks and Soils

In parallel with the development of research into engineering geological mapping in the 1960s, 1970s, and 1980s, engineering geologists and soil and rock mechanics engineers started to develop improved systems for the standardized description and classification of soils and rocks. These systems are beyond the scope of this contribution but information can be found elsewhere in this encyclopedia. These various descriptive systems and classifications are subject to almost continuous development and improvement. The latest versions can be found in European, Australian, and other national and international standards and in publications of organizations such as the International Association for Engineering Geology and the Environment (IAEG), the International Society of Soil Mechanics and Geotechnical Engineering

(ISSMGE), and the International Society for Rock Mechanics (ISRM). A very useful and readable introduction to soil and rock description and classification has been produced by Norbury (2016).

Geohazard and Georisk Maps

A geohazard map is one that shows some or all of the following for one, or more, of the wide ranges of different geohazards:

- The number of geohazards
- Their extent
- Their likelihood or susceptibility
- The degree of hazard that they cause

Whereas maps showing the distribution of geohazard events, such as landslides or dissolution features, are relatively simple, with the locations of geohazard occurrences shown as points on small-scale maps and as areas on larger scale ones, hazard susceptibility maps are usually more complex. Most are derived by assessment of a number of geoenvironmental factors using one of a large number of available analytical tools. The maps produced show relative susceptibility often using qualitative terms such as very low, low, medium, high, very high, and combinations thereof. According to Varnes (1984), geohazard maps show the probability of a particular geohazard within a given area and a defined period of time. However, Fell (1994) modified this definition by defining geohazard (in terms of landslides) as the landslide magnitude (M) times the probability (P), where M and P are defined in Tables 4 and 5. The hazard (H) = (M × P) and is defined in Table 6. For other geohazards, separate specific tables of magnitude would have to be devised, leading to geohazard-specific definitions of hazard (H).

Geohazard maps can cover areas ranging in size from a whole country (or indeed continent) to a location of a few hundred square meters. Their purpose is to provide information to various stakeholders to lessen or mitigate the impact of these hazards. Geohazard maps are usually prepared by a range of applied geoscientists, depending upon the type of hazard being investigated.

Classification of Geohazards

The type of geohazard map produced is dependent upon the nature of the geohazard being mapped. Geohazards can be subdivided into three main groups.

Primary Natural Geohazards

Primary geohazards, such as earthquakes and volcanic eruptions:

Engineering Geological Maps, Table 4 Landslide magnitude (M) classification (Fell 1994)

M	Description	Volume (m^3)
7	Extremely large	>5,000,000
6	Very large	1,000,000 – 5,000,000
5	Medium-large	250,000 – 1,000,000
4	Medium	50,000 – 250,000
3	Small	5000 – 50,000
2	Very small	500 – 5000
1	Extremely small	<500

Engineering Geological Maps, Table 5 Landslide probability (P) classification (Fell 1994).

P	Description	Annual
12	Extremely high	1
8	Very high	0.2
5	High	0.05
3	Medium	0.01
2	Low	0.001

Engineering Geological Maps, Table 6 Landslide hazard (H) classification (Fell 1994)

H	Description
≥30	Extremely high
≥20, <30	Very high
≥10, <20	High
≥7, <10	Medium
≥3, <7	Low
≥2	Very low

- *Appear* to be cyclical in their occurrence
- Have return periods determined by analysis of past events
- Affect regions
- Are controlled by *regional* geology
- Are generally unpredictable as the geological processes are not well enough understood
- Are best mitigated by engineering design and focus on secondary geohazards

The hazard is generally represented as the probability of a certain magnitude event occurring. The problem with this approach is that the probability of occurrence is based on data from very short (in geological terms) periods of time.

Secondary Natural Geohazards

Secondary geohazards, such as landslides and dissolution:

- Are often triggered by primary hazards
- Have return periods that are difficult to determine by analysis of past events because of limited data and non-steady state conditions

- Affect sites and districts
- Are controlled by *local* geology
- Are partially "predictable" from an understanding of geological processes
- Are best mitigated by land-use planning, insurance, and site-specific engineering

The hazard can be represented deterministically because the geological processes that control these geohazards are relatively well understood. Whereas records of past events are often relatively sparse and cover short periods of time, evidence for events for some geohazards that took place tens of thousands of years ago can be observed.

Geohazards Caused by Human Activity
There are four broad causal types:

- Extraction of minerals and its after-effects
- Engineering activities on, or just below, the ground surface
- Alteration of surface water or groundwater conditions
- Waste materials placed in, or on, the ground

Some of these geohazards have very little geological control unless events in the Anthropocene are regarded as geologically as well as human controlled. The methods for mapping some of these geohazards are still developing.

Landslide Hazard Maps

Chacón et al. (2006) provided a report for the IAEG Commission on Engineering Geological Maps on landslide maps. They divided landslide maps into four types:

- Inventory maps
- Susceptibility maps
- Hazard maps
- Risk maps

Of these four types, the first two are probably the most common. Inventory maps vary mainly depending upon scale. For example, small-scale maps might only show landslide locations whereas larger scale maps might classify the landslides and distinguish between landslide sources and deposits.

Susceptibility maps show areas where landslides are more, or less, likely to occur. Such maps are created by analysis of a wide range of potential triggering factors such as geology (particularly lithology and structure), climatic conditions (e.g., rainfall), hydrology and hydrogeology, seismicity, geomorphology (e.g., slope steepness and aspect), vegetation, and anthropogenic factors (e.g., proximity to roads or drains and agricultural activity). The various factors in an area are zoned and then the various factors are analyzed together to produce the final susceptibility map. The analytical techniques used have become increasingly complex and sophisticated (e.g., Meng et al. 2016).

By comparison, for debris flow susceptibility mapping, Mingyuan et al. (2016) used different factors that influenced susceptibility including basin area, the length, curvature, and average gradient of the main channel, elevation range of the catchment, average slope angle, drainage density, loose material volume, and supply length ratio, proportion of poor vegetation in the watershed area and basin roundness. Again, complex analytical techniques were used.

At the present time, it is not possible to identify which analytical techniques work best and in which particular environmental situations. However, as both digital analytical techniques and digital mapping methods develop further in the years ahead, identification of both the most straightforward and accurate approaches is likely to become more difficult before it becomes easier.

Hazard maps differ from susceptibility maps in that they take account of the fourth dimension – time. The difficulty is that to produce them, information is required about landslide occurrence over time. In northern Europe, the vast majority of landslides have occurred in roughly the last 10,000 years (during the Holocene) following the approximate end of the Ice Age. However, information on when, within that period of time, landslides occurred is limited to the last few hundred years. Also, climatic and anthropogenic conditions have changed. Consequently, it is difficult to determine a meaningful probability of landslide occurrence.

Similarly, landslide risk mapping, which includes assessment of the vulnerability of an area and the elements at risk as well as hazard assessment (see above), is difficult to carry out quantitatively with a high degree of confidence.

Subsidence Hazard Maps

Marker (2010) produced a further report for the IAEG Commission on Engineering Geological Maps on maps of hazards arising from subsidence into cavities ("hazard" is used here in a general sense meaning threats to people, structures, and infrastructure). The cause of the subsidence described is either natural – mainly dissolution of more soluble rocks, such as limestone, chalk, gypsum, and halite ("salt"), marine erosion, and mass movement – or the result of the mining of a wide range of minerals. Marker (2010) identified the same types of maps as Chacón et al. (2006) did for landslides – inventory, susceptibility, hazard, and risk. However, with reference to abandoned mine workings, he noted the value of documentation maps that show areas that may include mined ground and mine entrances and which, hence, provide useful background information for the planning of ground investigations. Such maps, which can be regarded as a type of inventory map, are a starting point for more detailed and accurate mapping of mining areas.

The identification of subsidence susceptibility is often more straightforward than for landslide susceptibility mapping, in that the extent of either more soluble rocks or potentially exploitable minerals is more obvious from geological maps produced by national geological surveys and similar organizations. However, neither the full extent of dissolution nor of underground mining is observable from the surface (unlike landsliding). Consequently, methods to determine the susceptibility to subsidence of an area are much less sophisticated than for landslides. Edmonds et al. (1987) developed the concept of the "subsidence hazard rating (SHR)" for natural and artificial cavities in the Chalk of southern England.

For artificial cavities (mining):

$$SHR_A = (R + L)S$$

where R is a regional factor indicating the varying number of cavities per unit area, L is the locational factor indicating the variation of type and purpose of cavities between different locations, and S indicates the relative stability of the cavities. R and S are on scales of 1–10 and L on a scale of 1–5.

For natural cavities:

$$SHR_N = (G_1 + G_2 + H_1 + GM_1 + GM_2)H_2$$

where G_1 relates to the proportion of metastable features associated with different lithostratigraphic units in the chalk, G_2 relates to the nature of post Cretaceous deposits overlying the chalk, H_1 relates to the influence of the water table, H_2 relates to topographical relief and drainage, GM_1 relates to former drainage paths, and GM_2 relates to the presence of glacial deposits.

From the overall values of SHR, areas under investigation can be zoned.

Forth et al. (1999) produced limestone dissolution susceptibility maps for part of the Algarve coast in Portugal. Similar to landslide susceptibility mapping, they used a series of factors relating to the type of dissolution features present, slope angle, the nature of tension cracks, fissuring, presence of rockfalls, estimated strength, and vegetation cover. Each factor was allocated a series of weighting factors which were then summed so that the area could be zoned in terms of degree of influence on hazard.

Relatively few attempts have been made to map risk due to subsidence. Ragozin and Yolkin (2003) carried out a hazard and risk study in the Tartar Republic of Russia involving Upper Permian carbonate and carbonate-sulfate rocks overlain by Pliocene-Quaternary silty-clayey sediments. The hazard maps were combined with measures of physical and economic vulnerability to create risk maps. Marker (2010) suggested that there was a need for more research on the level of risk associated with areas subject to subsidence.

Georisk

A geological risk can be defined as the combination of the probability, or frequency, of occurrence of a defined geohazard and the magnitude of the consequences of occurrence to health, property, and the environment. Geological risk is often represented by the following equation:

$$\text{Risk } (R_s) = \text{Hazard}(H) \times \text{Elements at Risk}(E) \times \text{Vulnerability}(V)$$

where:

H (hazard) is: *the probability of occurrence of a particular magnitude of geohazard within a specified area*, *within a given period of time*, or *the susceptibility of the ground to a particular hazardous process*

E (elements at risk) is: *the total value of population, properties, artefacts, infrastructure, amenity, etc., within the specified area under consideration – also known as the "exposure"*

V (vulnerability) is: *the proportion of the "elements at risk" affected detrimentally by the hazard (that is, the significance of the loss represented as either a percentage or on a scale of 0–1) – also known as the "potential to suffer harm"*

Fell (1994) provided classification tables for H, V, and R_s for landslides (Tables 6, 7, and 8).

Whereas this definition can be relatively easily understood, it is quite difficult to measure quantitatively because of the incompatibility of the terms relating to the likelihood of geohazard occurrence and their consequences. It is much easier to express risk in qualitative terms plotting likelihood against consequences as shown in Table 9.

"Likelihood" is best determined by geologists, seismologists, and other Earth scientists whereas "consequences" will be usually assessed by geographers, land-use planners, structural engineers, economists, social scientists, and others depending on the nature of the geographical area being assessed in terms of buildings, structures, infrastructure, and the distribution of people and services.

Engineering Geological Maps, Table 7 Landslide vulnerability (V) classification (for property loss, not loss of life) (Fell 1994)

V	Description
≥0.9	Very high
≥0.5, <0.9	High
<0.5	Medium
≥0.05, <0.1	Low
<0.05	Very low

Engineering Geological Maps, Table 8 ≥0.1, Landslide specific risk (R_s) classification (Fell 1994)

R_s	Description
≥0.1	Very high
≥0.02, <0.1	High
≥0.005, <0.02	Medium
≥0.001, <0.005	Low
≥0.0001, <0.001	Very low

Engineering Geological Maps, Table 9 Plot of likelihood of geohazard occurrence against the consequences of the occurrence to give a qualitative measure of risk

LIKELIHOOD	CONSEQUENCES: Negligible	Marginal	Critical	Catastrophic
Certain	High	High	Extreme	Extreme
Likely	Moderate	High	High	Extreme
Possible	Low	Moderate	High	Extreme
Unlikely	Low	Moderate	Moderate	High
Rare	Low	Low	Moderate	Moderate

Elements at risk and vulnerability maps are relatively uncommon in the literature, though maps showing the vulnerability of buildings to earthquake damage exist for Porto in Portugal (Moura et al. 2009) and maps showing spatial hazards and risk including vulnerability indicators such as population density, infrastructure, economic potential, or elements at risk were produced in 2008 for Central Java by a collaboration between the Geological Agency of Indonesia (Badan Geologi) and the German Federal Institute for Geosciences and Natural Resources (BGR) (https://www.whymap.org/EN/Themen/Zusammenarbeit/TechnZusammenarb/Downloads/indonesien_informationsblatt_riskmapping.pdf?__blob=publicationFile&v=2).

Groundwater risks can be evaluated within a "source-pathway-receptor" framework in which the lack of any one of the three elements means that there is no risk. In other words, for a risk to exist, there needs to be a source of contamination, the pollution from which can reach the groundwater table, one or more flow-paths by which polluted groundwater can migrate and downstream objects or processes that would be adversely affected by the polluted groundwater. Waters et al. (2005) described such a study for the Swansea-Neat-Port Talbot area of South Wales (UK). This is a coastal area with a long history of heavy industry (including steel-making and chemical processing) that left a legacy of groundwater, water-course, and land contamination. *Source* areas were sampled to determine their soil geochemistry for 14 potentially contaminating elements. The hydrogeology was investigated to determine potential *pathways* for contaminants. Four hydrogeological zones were identified and mapped – Zone 1, where the water table fluctuated because of river flows and tidal movements; Zone 2, where groundwater flowed through valley side glacial deposits; Zone 3, where perched aquifers were present; and Zone 4, where there were discrete perched aquifers above an elevation of 150 m. From these studies, and the maps produced, potential *receptor* areas for contaminants could be identified.

Unfortunately, the development of georisk maps has been slow. A decade or so ago, van Westen et al. (2006) observed, with respect to landslides, that while "*quantitative risk assessment on a site investigation scale or for the evaluation of linear features is feasible*. . .. *the generation of quantitative risk zonation maps*. . .. *seems still a step too far.*" They questioned whether there was even a need from development and emergency planners for such maps at a medium scale (1:10,000–1:50,000). Finally, they recommended that "*the various components of landslide risk assessment should be integrated in risk information/management systems which should be developed as spatial decision support systems for local authorities dealing with risk management.*" This would be true for other geohazards too. However, as pointed out by Culshaw and Price (2011), local authorities are not necessarily the best places to hold and develop such information systems in the long term.

Going Forward

It is interesting that Marker and Culshaw (2002) noted in a 10 year forward look that, at that time, there were four main trends in geohazard mapping:

- A move from geohazard inventory maps toward those that provide interpretative information for planning of land-use or for relative evaluations of risk
- Inventory maps being replaced by digital databases linked to maps within a GIS environment
- Increased recognition of the nonspecialist as an important user
- Increasingly widespread use of information technology

They also noted, as discussed by van Westen et al. (2006), that there was a general lack of risk maps and particularly a lack of research into how risk could be quantified meaningfully and presented on maps.

Whereas the four trends have been sustained, there remains a paucity of published georisk maps. Either this need is still to be delivered or the task is not one for engineering geologists, in particular, and Earth scientists in general.

Urban/Planning Maps

Whereas civil engineering professionals have been seen as the main customers for engineering geological maps, the development process, including the making of decisions on how land is used, begins long before building and construction takes place. As a result, engineering geological maps that present information that helps land-use planners to understand potential geological constraints on, and resources for, development have been produced over many decades but particularly in the 1980s, 1990s, and the 2000s. As there may be many potential constraints on, and opportunities for, development, provided by the nature of the ground, such maps tend to be provided in sets, each map of a set focusing on a different theme. The maps can be subdivided into those that provide essentially factual information ("Element Maps" of Smith and Ellison 1999), those that provide interpreted information ("Derived Maps"), and those that summarize the content of the other two types in terms of issues of particular importance to development ("Potential Maps").

In the UK, Culshaw and Ellison (2002) and Smith and Ellison (1999) noted that, from the users' perspective, information was needed particularly for the:

- Provision of land suitable for development
- Protection and development of mineral resources
- Protection and development of water resources
- Protection of agricultural land
- Provision of waste disposal sites
- Control of pollution and contamination
- Control of flooding
- Conservation of sites

These various information requirements could be summarized as being necessary to understand the geological constraints on, and resources for, development and conservation. As the urban geology mapping program evolved in the UK between 1982 and 1996 (when it ended), the number of maps and reports produced for each city increased, culminating in the following (Smith and Ellison 1999):

- Summary maps – either one map showing key planning issues or two maps showing opportunities for and constraints on development
- A summary report for planners
- Thematic maps showing a range of Earth science themes
- A technical report providing detailed descriptions of the geotechnical and geological aspects
- A digital database with basic Earth science information and capable of incorporation into a GIS (geographical information system)

Engineering geological maps for use in land-use planning are usually referred to as urban geological maps because they are mostly prepared for use in towns and cities. The research that has underpinned the development of urban geological mapping has taken place in a number of countries, often funded by national, state, regional governments, or city councils. The work has been summarized in various publications such as Eyles (1997) (Canada) and Smith and Ellison (1999) (UK). The Association of Environmental and Engineering Geologists published a CD-ROM entitled "Geology of the cities of the world" containing 12 papers on American cities, 6 on South African cities, and 1 each on cities in Canada (Toronto), Egypt (Cairo), Hong Kong, Italy (Rome), and New Zealand (Christchurch) (Anon. 2006). Most of the papers were published in Environmental and Engineering Geoscience between 1981 and 1995 or in the Proceedings of the Symposium on the Engineering Geology of Cities in South Africa in 1981. However, these papers were produced to a format specified by the AEG rather than by potential users. The format used was briefly discussed by Culshaw and Price (2011). Publications from the "LANDPLAN" series of conferences, and later the International Working Group on Urban Geology (IWGUG), have encouraged research and made available a range of different approaches and map types from many countries. More recently, Culshaw and Price (2011) summarized the development of urban geology to that point in time.

Despite the work of the IWGUG, between its formation in 1992 and its closure in 2008, development of urban geological mapping methods is now going through a period of stasis. This is despite the rapid development of digital three-/four-dimensional mapping/modeling techniques. As a postscript, Culshaw and Price (2011) suggested a number of actions needed in the near future:

- Long-term collection, digitization, and storage of urban geo-data
- Creation of 3D–4D models of the shallow subsurface and the attribution of these models with relevant parameter data – linking the databases to the model
- Assessment of the uncertainty associated with the models and explanation what it means
- Carrying out cost-benefit studies to demonstrate the value of urban geo-information
- Integration of physically based 3D ground models with process and socioeconomic models to assess the

vulnerability/resilience of the urban subsurface to future environmental change
- Better communication with users and understanding of their needs
- Persuasion of the policy makers and politicians who, ultimately, control spending on geo-information in cities, that continuing to do so is cost-effective and environmentally beneficial

At the heart of these proposals is the need for the continuous collection of geoinformation and its storage in, preferably, digital databases that are maintained in the long term. Traditionally, this has been the role of national and regional geological surveys.

Maps for the Siting of Waste Disposal Sites

Engineering geological maps for the siting of waste disposal facilities are a particularly specialized type of map. The IAEG's Commission No. 1 on Engineering Geological Mapping provided a comprehensive report giving advice on how to prepare this type of map to identify sites suitable for inert wastes, municipal wastes, and special wastes (but not radioactive wastes) (Proske et al. 2005). Map examples from 25 sites were examined; these mainly came from locations in Europe and North America. The authors listed the steps necessary to evaluate the suitability of an area for a waste disposal site:

1. Selection of the geological factors having the most significant effect on the suitability of land for waste disposal
2. Ranking of these geological factors into classes according to their importance
3. Regrouping of the geological factors in terms of land suitability
4. Preparation of a land suitability map
5. Selection and evaluation of alternative sites within the mapped area using quantitative methods
6. Selection of the best site

The map or maps are developed in steps 1–4 and then they are used in steps 5 and 6. The main geological factors that should be taken into account in preparing a map for the identification of sites suitable for waste disposal are:

- The underlying geology
- The ground and surface water conditions
- Earth surface processes (geohazards – slope stability, flooding, etc.)

Overall, the process of identifying suitable sites involved the . . ."comparison of a set of relevant criteria to rule out generally unsuitable areas or to weight the factors to give a relative suitability index."

The map examples given by Proske et al. (2005) showed various ways by which the evaluation process was carried out. However, there is no assessment or comparison of how successful each of the approaches was in terms of site selection, probably because the various reviewed published papers did not provide this information.

From Analogue to Digital and From 2D to 4D

Geological maps, and hence engineering geological maps, are a two-dimensional representation of a three-dimensional object (parts of the Earth). Because these maps are intended to inform users, they were usually reproduced on a convenient medium to enable communication. Inevitably, this medium was paper. However, paper is inconvenient in that a master map needs to be produced by hand and then this can be copied using any of a number of printing processes. Until relatively recently (the 1990s can be considered, perhaps, as the turning point), this meant that multiple copies of printed maps were relatively expensive to produce and difficult to update easily without reprinting. Also, the maps are two-dimensional and the third dimension has to be interpreted. Geologists have realized the limitations of two-dimensional geological maps almost since they were first developed in the eighteenth and nineteenth centuries.

Sopwith's Models

In the first half of the nineteenth century, when William Smith was producing his iconic geological maps of Britain and its counties, Thomas Sopwith began producing a series of three-dimensional wooden models to illustrate geological structure. He was not the only one to do this but his models are perhaps best known. Other modelers included Elias Hall and John Farey (Turner and Dearman 1980). The popularity of these models demonstrates the early realization of the importance of understanding the three-dimensional variation of geology.

Fookes' 1997 Glossop Lecture

Peter Fookes' paper on geological model, prediction and performance (Fookes 1997) helped engineering geologists begin the process of moving from the two-dimensional map toward the three-dimensional model. He described the geological model as "*A representation of the geology of a particular location. The form of the model can vary widely and include written descriptions, two-dimensional sections or plans, block diagrams, or be slanted towards some particular aspect such as groundwater or geomorphological processes, rock structure and so on.*" The model's purpose was "*to counter any unawareness of ground conditions.*" Fookes developed a series of conceptual ground models that could, for example, represent different environmental conditions such as hot deserts, different geological conditions such as igneous rock associations in a wet temperate climate, or specific sites for, say, quarrying or construction.

Toward 3D Engineering Geological Models

The development of digital mapping/modeling over the last 20–30 years has revolutionized all forms of map-making including engineering geological mapping. One of the consequences is that the rather rules-based approach of Anon. (1972, 1976) to producing engineering geological maps has almost disappeared. This is probably because it is *relatively* easy to produce a wide range of three-dimensional digital maps/models to show the information and interpretations required by users.

It is noticeable that as maps/models become increasingly three-dimensional, the terminology is changing and for most geologists, the term "map" tends to mean two-dimensional whereas "model" means three (or four) dimensional. There may be unforeseen consequences arising from this change. When three-dimensional geological models were unavailable digitally, one defining skill of a geologist was to be able to mentally visualize both two-dimensional geological maps and the ground itself in three-dimensions. The increasing availability of digital three-dimensional geological models may have the unintended consequence of geologists beginning to lose this essential skill.

Geach (2016) has pointed out another concern about digital, three-dimensional models, namely, the limitations of geospatial interpolation. This is the technique by which one-dimensional data points are linked together to produce continuous geological surfaces. Geach (2016) pointed out that because there are various interpolation techniques, the results produced by each technique may vary, even though they used the same initial data.

Another advantage of the increasing sophistication of digital mapping/modeling is that it is also possible to reproduce the fourth dimension – the effects of change with time. Hydrogeologists have led this development because of their need to model the movement of groundwater and of any associated pollutants, but another application is modeling changes in landscape, for example, caused by landslides, land subsidence, bulging during increased volcanic activity or tectonic activity in seismically active areas. With the development of high-resolution remote sensing techniques, such as LIDAR (LIght Detecting And Ranging) and InSAR (Interferometric Synthetic Aperture Radar), the ability to carry out four-dimensional modelling of changes to ground surface shape has become increasingly important.

The development of digital three/four-dimensional engineering geological models and their uses are discussed elsewhere in this encyclopedia. More detailed discussion of the transition from Fookes' (1997) analogue conceptual models to digital three-dimensional models can be found in papers by Culshaw (2005) and Royse et al. (2009).

Conclusions

Although what we would now recognize as "engineering geological maps" were produced from around the end of the nineteenth century, they were not really identified as such until the 1960s and 1970s when this type of geological map was defined and classified. Engineering geological maps were defined by their content and also by how the sub-discipline of engineering geology, itself, was defined. Initially, engineering geology was seen to be the application of geology to civil engineering and, so, many engineering geological maps were created to identify geological problems relevant to engineering design and construction.

Over time, and particularly from the 1990s onward, the scope of engineering geology broadened. Engineering geology is now seen (after Bob Tepel's definition in the December 2012 edition of AEG News, page 6) as "discovering, defining, and analyzing geologically-sourced risks or conditions that impact, or might impact, humans as they utilize and interact with their built and natural environments." Engineering geological maps now cover a much broader range including geohazard maps, maps for planning and development in urban areas, environmental geological maps covering issues such as pollution and contamination, and the siting of waste disposal sites, as well as the more traditional geotechnical maps.

However, digital technology was developing fast with the creation of geographical information systems (GIS) and the gradual move toward three and four-dimensional modeling. If a map is seen to be a piece of paper showing elements of the geology, then engineering geological mapping is a dying art. However, to define maps in such a way is meaningless as the techniques used to produce engineering geological maps can similarly be used to produce digital three and four-dimensional engineering geological models. It can be argued that only the means of portraying the information has changed. If our engineering geological maps no longer have to have a rectangular boundary, so much the better!

Cross-References

▶ Classification of Rocks
▶ Classification of Soils
▶ Databases
▶ Engineering Geology
▶ Engineering Geomorphological Mapping
▶ Geohazards
▶ GIS
▶ Groundwater
▶ Hazard Mapping
▶ Infrastructure
▶ International Association of Engineering Geology and the Environment (IAEG)

▶ Land Use
▶ Landslide
▶ Lidar
▶ Mining Hazards
▶ Probabilistic Hazard Assessment
▶ Risk Mapping
▶ Rock Mass Classification
▶ Subsidence

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Engineering Geology

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Definition

An applied discipline of geology that employs knowledge of geologic principles and processes to support how humans utilize their environment (Johnson and DeGraff 1988; IAEG 2015).

Characteristics

Because it is important to determine how society is affected by and affects the natural environment, engineering geologists must investigate and study the ground using sound principles of inquiry (AEG 2015). Engineering geologists contribute their knowledge to solving engineering and environmental issues arising from the interaction of human works and activities with the natural environment (AEG 2015). Therefore, engineering geologists frequently work as part of interdisciplinary organizations that include other professionals such as engineers, planners, hydrologists, and construction specialists.

Engineering geologists play a key role in maintaining or creating the buildings, roads, dams, and other structures that modern society depends upon. An engineering geologist needs to develop a thorough understanding of rock and soil including their structural geologic setting and the weathering processes affecting the area over time (Johnson and DeGraff 1988). By developing this understanding, the engineering geologist can provide important information affecting design and construction. Specific findings may be about characteristics of the underlying rock and soil affecting their suitability as a construction material or their capability for supporting overlying structures. The presence, or movement, of subsurface water that may need to be controlled is also valuable information influencing design and construction that the engineering geologist can determine. The information which the engineering geologist must develop and provide should always be focused on the needs of the particular project being undertaken (Johnson and DeGraff 1988).

Another primary role of engineering geology is assessing geologic hazards including seismic shaking, liquefaction, subsidence, sinkhole development, flood zones, and landslide activity including debris flows. The emphasis is on preventing or limiting the adverse effect of various hazards to society (Lindell 2013). With public safety a primary concern, engineering geologists are called upon during active events to identify the area being threatened, determine factors important to taking effective mitigating actions, and advise emergency response agencies. Before active events occur, engineering geologists work with planners, engineers, and other government specialists to identify potential hazards for emergency management (Lindell 2013). Engineering geologists provide information needed by engineers to make sure the infrastructure our society depends on is properly designed (Johnson and DeGraff 1988). A similarly close working relationship that exists with environmental geologists promotes effective solutions to contaminated soils and polluted waters (AEG 2015). Both disaster preparedness and land use planning benefit from development of maps showing differences in hazard potential, defining the nature of potential hazards, and predicting the likelihood of future hazard events (Marker 2013).

Cross-References

▶ Bedrock
▶ Earthquake

E

▶ Foundations
▶ Geohazards
▶ Geotechnical Engineering
▶ Ground Shaking
▶ Hazard Assessment
▶ Land Use
▶ Landslide
▶ Mass Movement
▶ Residual Soils
▶ Risk Assessment
▶ Risk Mapping
▶ Sinkholes
▶ Site Investigation
▶ Water

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Engineering Geomorphological Mapping

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Definition

Engineering geomorphic mapping is the process of creating a graphical representation of geomorphic features for an engineering application. The mapping may be used to identify, classify, quantify, and visualize geomorphic features for development planning and site characterization purposes.

Introduction

Geomorphology is the study of landforms and the processes involved in their formation. Engineering geomorphic mapping provide a geographically referenced depiction of landforms and surficial processes. Geomorphic maps may be produced for the following reasons:

- To provide an understanding of the landscape and the processes that formed and continue to modify the landscape
- To provide a geographically referenced description of the landscape and the identification of problematic landscape features
- To provide a map of the landscape to be used as the basis for a derivative mapping product (Cooke and Dornkamp 1990)

The engineering geomorphologist situates engineering works within a landscape context. The engineering geomorphologist contributes to solving engineering problems by assessing current conditions and predicting future conditions. An understanding of how the landscape formed and continues to develop is fundamental for predicting its future. Engineering geomorphic mapping might be done for the following reasons:

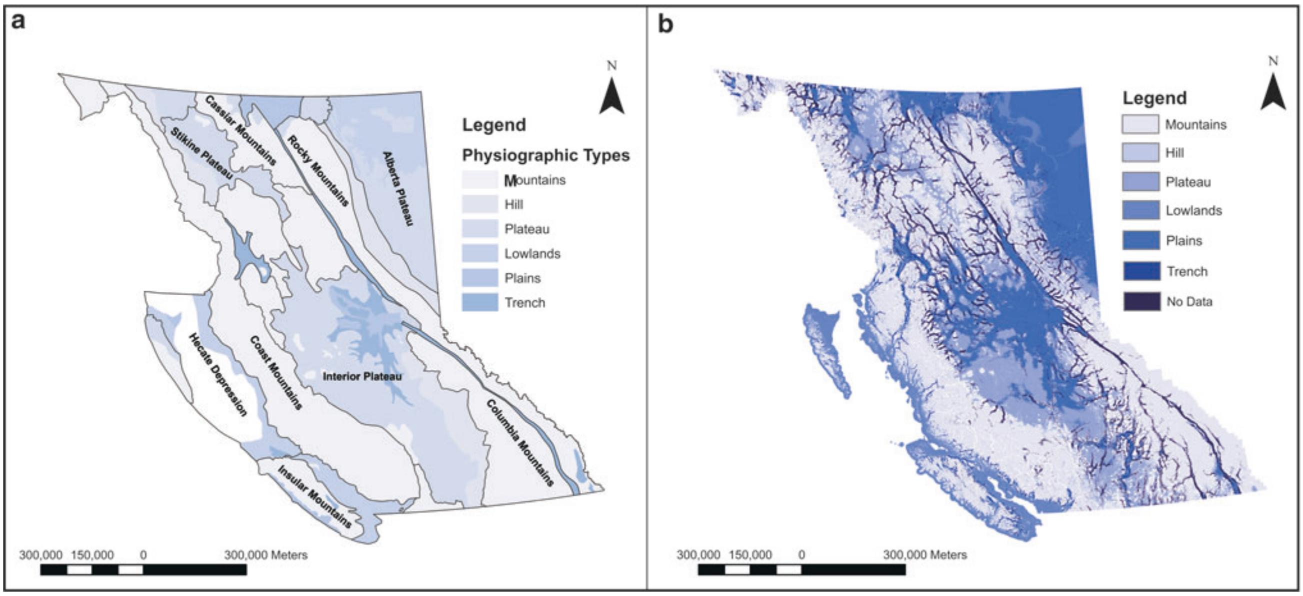
- To identify existing geotechnical and hydrotechnical hazards and conditions and to provide predictions on potential conditions
- To perform landscape risk evaluations for predevelopment planning and post-development risk mitigation purposes
- To characterize existing foundation materials and hydrogeological and drainage conditions for development planning purposes

Today, geomorphic maps are used to depict features at scales ranging from the global to the site level. The processes developed for geomorphic mapping can be universally applied and are being used on Mars as a means of understanding its geologic history.

Geomorphic Mapping Techniques and Concepts

The process of geomorphic mapping includes the following steps:

- Identifying the purpose and scale of the mapping
- Setting the mapping criteria (qualitative and/or quantitative) and choosing a classification scheme
- Gathering background information and data



Engineering Geomorphological Mapping, Fig. 1 Small-scale physiographic map of British Columbia, Canada. (**a**) Vector format map adapted from Holland (1976) and reclassified into broad physiographic types. (**b**) Raster format created using a maximum likelihood classification with an input 1 Ha Digital Elevation Model and five terrain derivatives (including 8 km neighborhood topographic position index, 2 km neighborhood topographic position index, slope, plan curvature, and profile curvature) to define physiographic type

- Mapping the features by interpretations or modeling
- Validating the map with field data and possibly laboratory data
- Producing the mapping products using GIS and cartographic techniques
- Producing supporting materials such as reports, legends, presentations, etc.

Purpose for Engineering Geomorphic Mapping

Engineering geomorphic mapping is the depiction and characterization of geomorphic features for application in engineering works. Engineering geomorphic mapping is particularly useful for linear infrastructure (roads, railways, powerlines, and pipelines), as these often cross complex terrain and are exposed to variety of geomorphic hazards and conditions. Likewise, these developments can impact other geomorphic values downslope, if the existing geomorphic conditions are not fully appreciated prior to construction. Geomorphic mapping is also useful for site-specific developments to identify and characterize existing or potential geotechnical and hydrotechnical hazards, as part of the predevelopment planning stage or for post-development risk mitigation efforts. Forestry operations will routinely use geomorphic mapping as a means of minimizing their environmental footprint and protecting their infrastructure investments.

Concepts of Scale

Geomorphic features vary with scale. The mapping scale is chosen according to:

- The purpose or intended use of the mapping
- The spatial patterns and scale of geomorphic features of interest
- The available imagery and base mapping data
- The resources available for field work

As the purpose for engineering geomorphic mapping is for its application to engineering works, the scale of mapping is generally 1:25,000 or larger. However, other scales can provide useful information for planning purposes (e.g., small-scale maps provide an understanding of the distribution of macroscale geomorphic features, such as mountain ranges, plateaus, plains, and basins, and are useful during the development planning stage). Figure 1 is a small-scale physiographic map of British Columbia, Canada, indicating major geomorphic features.

Concepts of Multiple Dimensions

Geomorphology varies in multiple dimensions. Features have location, length, area, depth, and change over time. The basic geomorphic map displays features on a 2D plane. Some maps use symbology or attributions to indicate thickness of stratigraphic units, providing information on the third dimension. The third dimension can also be displayed using cross sections and block diagrams. Modern interactive mapping technologies allow three-dimensional rendering of maps on screen. Three-dimensional site characterization is an important component of engineering geomorphic maps.

The fourth dimension, time, can be captured using a series of maps or by mapping change using symbology or attribution. More recent interactive mapping technologies use a slider bar that allows the map user to scroll dynamically thought a time sequence of maps for a given geographic area.

Classification

Most geomorphic mapping uses a classification system to group similar materials, landforms, and processes into mappable units. Engineering geomorphic classification can be based on slope, slope curvature, landform genesis, material properties, and active or potential geomorphic processes. Quantitative measures relating to geomorphology can be displayed as continuous values but are often grouped into classes. Classification systems can be predefined and used widely across a jurisdiction or project specific, where the legend and classification is created by the mapper for a particular project or group of projects.

Some classification systems use a finite number of mapping units that are defined and described and then mapped across the landscape. This approach is appropriate for less complex landscapes or for identifying specific geomorphic features for focused projects.

The project-specific legend enables mappers to determine how to define polygons based on the local environment and the requirements for the mapping project. The method is versatile and can produce results that effectively communicate the geomorphic conditions at a particular site. It also allows the mapper to define units that are easy to describe and are representative and mappable at the project scale.

Other systems classify different aspects of geomorphology (such as lithology, drainage, process, genesis, and landform) and allow a combination of defined categories to represent the content of a mapped unit. The combination of these symbols can create an almost unlimited number of options allowing for a great deal of flexibility to describe the geomorphic landscape but, as a result, decrease the probability of reproducing the same symbol for the same unit between map areas and mappers. These classification systems are appropriate for multipurpose maps that aim to capture a wide diversity of geomorphic features and inform a variety of land use decisions.

Some types of geomorphic mapping divide up the entire landscape, whereas others delineate only specific features. Full landscape mapping divides the landscape into relatively homogeneous areas or areas of repeated patterns or complexes. All areas of the map must fit into a particular category or represent the unit described by the polygon label. Mapping criteria are used to apply mapping principles consistently across the study area. These criteria are used to lump similar areas and divide areas that differ according to the set criteria and the legend. Surficial geology maps and terrain maps are examples of full landscape mapping. Specific feature maps use polygons, lines, and points to highlight particular features across the landscape or to map one particular feature in great detail. The extreme is the binary map showing areas that meet or do not meet very narrow geomorphic criteria.

Representation of Complex Polygons

Often areas can have multiple geomorphic features that are too small to be mapped as separate polygons at the desired scale without the map appearing cluttered. As a result, complex polygons that include multiple geomorphic features are required. Complex polygons are handled in several ways. Often lengthy map unit descriptions are provided in the legend, which indicate the occurrence of lesser units. Some classification systems allow multiple components to be indicated in the map label with percentages or relative proportion of the different geomorphic units described.

Representation of Stratigraphy

Applied engineering geomorphology requires knowledge of the underlying stratigraphy for comprehensive hazard or site characterization (e.g., a relatively stable and well-drained alluvial terrace might lay over a weak glaciolacustrine unit). Stratigraphy is indicated in several ways. Legend descriptions will often indicate subsurface units. Some systems provide a means of noting subsurface units in the polygon labels (e.g., British Columbia (1997) has stacked stratigraphic units indicated by horizontal lines).

Hierarchical

Hierarchical classification schemes group landforms that have similar geomorphic genesis but which are visible at different scales. It relies on the idea that small-scale landforms are composed of a variety of larger-scale features. The United States Department of Agriculture, Forest Service (Haskins et al. 1998) created a geomorphic classification system that uses hierarchical map units to describe geomorphic process and landforms at different scales. These classification schemes can be applied across many scales and can be adapted to include as much or as little detail as necessary for the mapping objectives.

Field Techniques

A necessary stage of geomorphic mapping is the verification of map unit accuracy. This is done via field validation, through which data are collected in order to confirm whether the classification is reliable or not. Data observed in the field can be geographically referenced using a Global Positioning Systems and combined with mapped data, providing a comprehensive view of the landscape. Post-validation, the mapper may use the information to adjust their classifications or model. Subsequent field work may be required, to further improve accuracy. This cycle of validation and adjustment may be done as many times as required by the mapper to produce satisfactory results.

Pits, Exposures, and Cores

Field-based interpretations of stratigraphy, structures, and sedimentology are used to verify map units initially defined by interpreting remotely sensed data. Field-based efforts utilize existing exposures, excavated test pits, or materials brought to the surface by drilling or augering. Natural or anthropogenic processes often create exposures that can be useful sites for geomorphic analysis. Examples of these include road cuts, mining pit walls, river-eroded escarpments, and landslide scarps. These exposures can provide an extensive perspective of the stratigraphy. However, pre-existing exposures may not be favorably located; in which case, other options need to be utilized.

Test pits are commonly excavated where no pre-existing exposures are available. Test pits provide very limited depth penetration – generally a maximum of 6–7 m when using an excavator and less than 1 m when dug manually. Small-scale sedimentary structures and material fabric may be destroyed during excavation. The walls of machine dug pits are often unstable, making detailed observations perilous or impossible. In addition, as test pits only expose the stratigraphy at one location, a series of test pits may have to be excavated to assess trends or stratigraphic changes.

Drilling and augering are used when information must be gathered at greater depths than test pits can attain. The result is essentially a one-dimensional view of the subsurface materials. Like test pits, interpolation is required between drill sites, to achieve a broad landscape perspective. For surficial geological investigations, an auger is often used as an economical means of attaining data. The augering process destroys any material fabric and preferentially collects the finer portion of the subsurface materials, complicating interpretations. A sonic drill provides improved results to what an auger can achieve. The sonic drilling method vibrates a tool into the subsurface. The method provides a better representation of the actual grain size distribution, but the fabric is also destroyed.

Geophysics

Geophysics provides a subsurface perspective of the landscape. Geophysical data can be gathered using terrestrial-based techniques or from shipborne or aerial-borne platforms. Data acquired from drilling can be augmented using down-hole geophysical techniques. Geophysics will generally not provide a definitive interpretation of subsurface stratigraphy, and the geophysical data will have to be associated with stratigraphic data from other sources.

Global Positioning Systems

Since the late 1990s, Global Positioning Systems (GPS), which use satellites for accurately locating field sites, have been extensively used by geomorphic mappers. GPS data can be improved by using a base station at a fixed location to rectify GPS location drift. GPS coupled with a field-based Geographical Information System is an effective means of field locating office-derived information, allowing boundaries to be adjusted and interpretations to be verified.

Remote Sensing Products

The early history of geomorphic mapping is closely tied to the availability of aerial photography following World War II. Remote sensing was developed for military purposes, first using airborne sensors and subsequently using satellite-borne sensors. Both methods of data collection are still in use today. The evolution of geomorphic mapping is strongly tied to the quality, coverage, and availability of imagery. Data acquisition techniques and computer processing abilities have improved significantly since the advent of remote sensing, allowing for a wide variety of imagery types covering many geographic areas and resolutions. There are several types of remote sensing products and imagery used in geomorphic mapping.

Stereo Aerial Photographs

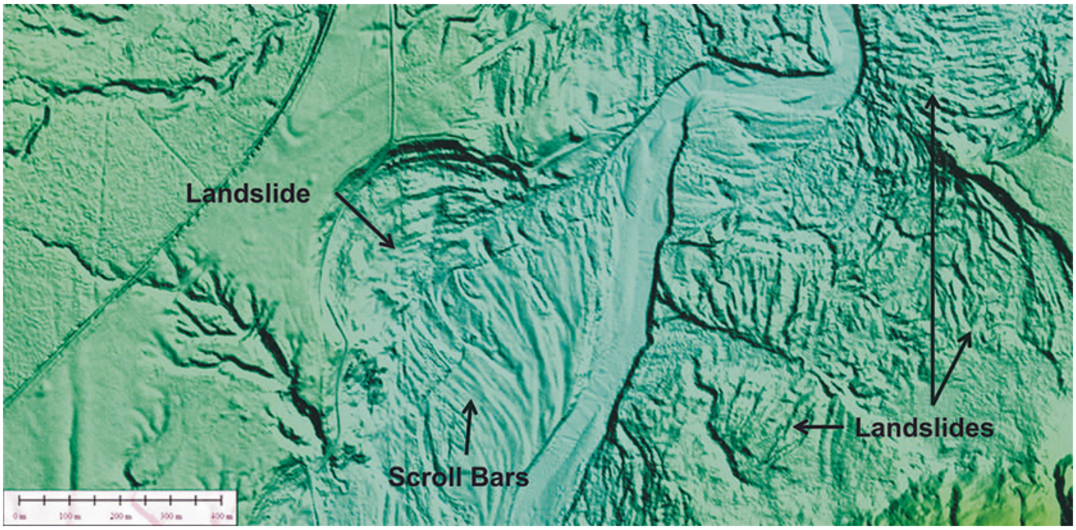
Stereo aerial photographs are one of the principal tools used by geomorphologists. The photographs provide a historical perspective of the landscape going back to the early 1900s. The time series that photography provides has proved indispensable for change detection in landslide, fluvial, and glacial geomorphic research. The resolution of modern aerial photography generally far exceeds space-borne imagery due to closer proximity of the aircraft to the Earth's surface. Whole landscape analysis using aerial photography is time-consuming and difficult as a result of infrequent repetition times between subsequent projects and the limited extent of the area captured in an image.

Satellite Imagery

Satellite imagery provides repeated image capture of most locations on the Earth at a much higher frequency than provided by aerial photographs. Modern satellite images capture spectral data beyond the visible spectrum. Modern satellites are capable of image capture in submeter resolution vastly improving their versatility as an engineering geomorphic tool. The footprint of an image can be much greater than that of an aerial photograph facilitating whole landscape analysis. Satellite imagery has its limitations in that stereo imagery is infrequently acquired, atmospheric distortion degrades the quality of the image, the resolution of the images are considerably poorer than what can be produced using an aerial platform, and cloud cover frequently obstructs optical data capture.

Digital Elevation Models

Digital Elevation Models (DEMs) are the digital representation of topographic data. DEMs are an invaluable engineering



Engineering Geomorphological Mapping, Fig. 2 Hillshade bare-earth Digital Elevation Model showing the Pine River valley, British Columbia, Canada. The image depicts a thick glacial lacustrine deposit with large retrogressive spreading landslides from both banks. Also evident are fluvial scrollbars from the meandering Pine River. Image provided courtesy of the Government of British Columbia

geomorphic analysis tool in that they provide a perspective of the landscape without the obstruction of vegetation (Fig. 2). This is referred to as a bare-earth perspective. DEMs can be generated by digitizing contour maps, using data from space-borne radar satellite platforms, generating surfaces from aerial or satellite optical images using photogrammetric techniques, or using a lidar system. The method by which data are collected will influence the DEM's resolution and accuracy.

Lidar DEMs provide the most accurate and highest resolution representation of the Earth's surface currently available. Lidar data are collected by using lasers to determine the distance between an emitter, at a known location, and the Earth's surface. Modern lidar systems emit laser pulses at very high frequencies so that in all but the most densely vegetated environments, some of the pulses will contact the ground enabling the generation of a bare-earth DEM. Lidar data can achieve sub-decimeter DEM resolution, enabling the rendering of micro-topographic features.

Geographic Information Systems (GIS) provide various means for the visualization of the bare-earth landscape including manipulating sun locations to alter landscape shading or using a gradation of colors to signify changes in elevation or slope gradient. Modern GIS applications either have an internal remote sensing software extension or allow for the peripheral use of a third-party remote sensing software. The use of remote sensing software can greatly improve the visualization and interpretation of DEM data. Remote sensing software allows for the creation and visualization of bare-earth DEMs in stereo, which vastly improves the accuracy and utility of engineering geomorphic mapping from DEMs.

Multitemporal DEMs can be used for the detection and assessment of ground deformation to sub-decimeter accuracy, with applications for landslide activity detection and land settlement monitoring. This can be done using terrestrial, aerial, or space-borne sensors. For example, Differential Interferometric Synthetic Aperture Radar allows for the detection of submeter surface deformation from satellites, by comparing data from multiple data capture occurrences.

Geographic Information Systems

Traditionally, geomorphic maps use points, polygons, and lines to convey information on geomorphic attributes. Cartographic techniques are used to symbolize and label the features to bring meaning to the map. More recently, Geographic Information Systems (GIS) allow exploration of geomorphic features and attributes in a more interactive manner. They allow for symbolization and analysis of geomorphic features using both qualitative and quantitative attributes. Advances in remote sensing, computing power, and big data have increased the use of raster (GRID) data to map, visualize, and analyze both categorical and continuous geomorphic data.

Geographic Information Systems provide a platform which enables all data collected to be analyzed, synthesized, and displayed. GIS allows for tremendous opportunity to improve the versatility and, by extension, the utility of geomorphic maps. As such, geomorphic mapping should be done in a manner that fully integrates GIS into the process. Mapping using quantitative and qualitative classifications with a limited number of categories is best suited for GIS display and analysis.

GIS uses two types of data formats to describe geographic information: vector and raster. Vector data uses X and Y coordinates to compose points, polygons, and lines. It defines feature centers and edges well. Raster data uses a matrix or grid of regular-sized squares called pixels or cells.

This format is efficient at storing and analyzing large datasets. Raster format is most appropriate for continuous variables that vary predictably across the landscape. It is commonly used to store imagery and quantitative variables such as climate, slope, elevation, curvature, aspect, ice direction, soil pH, and material thickness. Figure 1 shows a comparison of raster and vector formats.

Both data types allow for multiple geomorphic descriptors (or attributes) to be associated with a geographic location (cell, polygon, line, or point). Vector features have an associated attribute table to describe the characteristics of the point, line, or polygon. A raster stack allows for multiple descriptors to be attached to a single pixel. This allows the user to choose what attributes are important for the work being done and create themed maps for specific purposes. It also allows complex legend types that maximize the benefits of open and predefined systems and can also accommodate project-specific legend elements.

GIS gives the user significant flexibility in their mapping projects. It enables users to adjust class thresholds (e.g., adjusting the slope class thresholds) in response to an evolving understanding of parameter importance in landscape geomorphic analyses. GIS also enables progressively more detailed data to become visible at increased magnification or the converse at less magnification. This technique allows for scale-independent geomorphic mapping, effectively limited only by the scale of the primary data used for the mapping exercise. Additionally, it provides a framework to manage metadata and allows for data validation and the management of the classification hierarchy, codes, and values.

Delineating Vector Features

Geomorphic features can be identified on a map using polygons, lines, and points. The scale of the mapping dictates the minimum polygon size below which features are identified with lines and points. For example, specific features too small to be mapped as separate terrain polygon at the scale of the mapping (e.g., landslide scarps) are often indicated by a line along the top of the escarpment with hatch marks extending in the direction of landslide movement (see Dearman (1991) for a comprehensive list of symbols).

Raster Mapping

The raster format uses a grid of cells to assign attributes to a landscape. Typically, each cell is given a value that represents some feature of the surface. Values can be categorical, where each number represents a type of surface, or continuous, where the attribute is related to some quantitative feature or process on the land. Remotely sensed data such as aerial imagery and digital elevation models are created in raster format. These data can be manipulated into secondary data products (e.g., slope gradient created from a DEM). Satellite and aerial imagery can be reclassified based on spectral signatures into land cover maps. Often, interpretation of raster imagery and DEMs will involve delineating observations in vector format. Raster mapping of geomorphic features can be done through raster math operations in a multiple criteria evaluation. This involves a number of primary raster images that can be used to define land characteristics. However, basic raster math is done on a pixel by pixel basis and does not usually recognize overall geometry of adjacent pixels. As such, raster operations are limited to defining pixel characteristics, as opposed to large-scale landforms or processes.

Harnessing the Power of Raster and Vector

Combining raster and vector techniques can be a powerful tool in geomorphic mapping. Raster techniques can be used to consistently apply thresholds and statistical methods. Computed raster modeling tools provide consistency and repeatability for mapping geomorphic patterns that adhere to model assumptions. Automated mapping and modeling tools can be incredibly useful when dealing with large datasets and when feature boundaries are clearly definable. However, the trained human eye and the experience of geomorphologists will still discern and delineate complex geomorphic patterns, relationships, concepts, and features better than even the most sophisticated computer algorithms can achieve.

Automated Mapping

Geomorphic mapping projects are incorporating tools and techniques for automating the mapping process. These techniques can significantly reduce mapping time, cost, and subjectivity. Automated mapping can use expert-driven models where a mapper provides rules or criteria that are applied across the study area to classify the landscape into geomorphic categories. Other automated methods use statistical and machine learning techniques. In both methods, the maps are reproducible and with an independent validation dataset so the accuracy can be reported. Ultimately, the choice of using automated or manual techniques often comes down to the time and funds available for the project, the nature of the features being mapped, and the available input data for the study area. However, automated mapping techniques can be easily distracted by noise in the dataset, which can lead to over or underestimation of features. Expert knowledge is also needed to select appropriate training data and model inputs. As with manual mapping, automated mapping is an iterative process. Field verification and geomorphic knowledge is required to produce a defensible process, and independent field validation data are required to evaluate map accuracy. Collecting spatially accurate and reliable point data in order to produce statistically defensible maps can be time-consuming and expensive in areas where there are insufficient existing data. This can negate some of the cost and time savings over manual methods of mapping.

One machine learning method of classification used for automated mapping is Random Forests (Breiman 2001).

Random Forests is a multiple decision tree algorithm, in which bootstrap sampling is used to choose random selections of training sites to create multiple decision trees. A portion of the training data set is set aside to use to report the output error rate. Sites of known classification are used to teach the computer which categories relate to which input value. The class assigned by the majority of decision trees is assigned to a pixel. It is a robust and repeatable method that has built in error reporting measure included in the classification. Random Forests classifier has been used to map surficial material (soil parent material type) (Bulmer et al. 2016) and landslide susceptibility (Stumpf and Kerle 2011).

Automated mapping has been applied with varying degrees of success in a number of cases. Relevant applications include landslide mapping (Booth et al. 2009; Stumpf and Kerle 2011; Tarolli et al. 2012) and landform extraction (Asselen and Seijmonsbergen 2006; Robb et al. 2015). In general, the projects use landscape parameters (such as slope, texture, and elevation percentile) in conjunction with overall pixel geometry to classify each pixel as a geomorphic feature. This outlines the importance of clear definitions of landforms and relevant parameters.

Cartographic Techniques

The final step to a geomorphic mapping exercise is to create a clear and concise map product to report the results. Necessary cartographic elements include symbolization, a legend, a scale bar or representative fraction, and a base map. Consideration of symbol color, size, and shape will contribute to making the map unbiased and intuitive. A legend should be organized and should contain all symbols relevant to the message the map maker is trying to convey. A scale or representative fraction relates ground units to map units and is used to give context to the map viewer. A base map is also important for context. It can be made from simple geometric shapes (i.e., lines representing roads, polygons representing water bodies or buildings, and so on), as a terrain map using a DEM or contour lines, or using imagery to display ground data. Base maps are often made to be light colored or transparent, so as to not obscure data of interest. Maps should also have a north arrow if cardinal information is not inherently obvious. Supplementary material, including reports, block diagrams, cross sections, photographs, stratigraphic sections, sketches, tables, data sources, and other information, may be included to further enhance the cartographic design and interpretability.

Types of Geomorphic Maps

Terrain Maps

Terrain maps are a qualitative form of geomorphic mapping that subdivides the landscape based on various terrain attributes such as material texture, surficial material (geology), surface expression, and geomorphic process (Fig. 3) (British Columbia 1997). The terrain map can be used as the basis for several other mapping products (e.g., archeological potential maps, soil maps, terrain hazard and risk maps, ecosystem maps, and vegetation inventory maps).

Terrain Hazard Maps

A terrain hazard map is a geomorphic process map which considers pre-existing hazards or the potential of a hazard occurring as a result of anthropogenic or natural disturbance. These maps are often derived from terrain maps or focus in on known problematic terrain feature (see Schwab and Geertsema 2010).

Landslide Maps

A landslide map is a specific type of terrain hazard or process map which involves delineating specific landslide elements (e.g., scarps, grabens, tension cracks, movement vectors, lateral and transverse ridges, breaks in slope, and various other landslide features). The illustration of these features helps researchers to understand the kinematics of a landslide, which then goes toward the selection of an appropriate model to describe the geotechnical properties of the landslide, and determine the nature of the hazard. Dearman (1991) provided a comprehensive list of symbols which can be used in landslide mapping. Cruden and Varnes (1996) define landslides types.

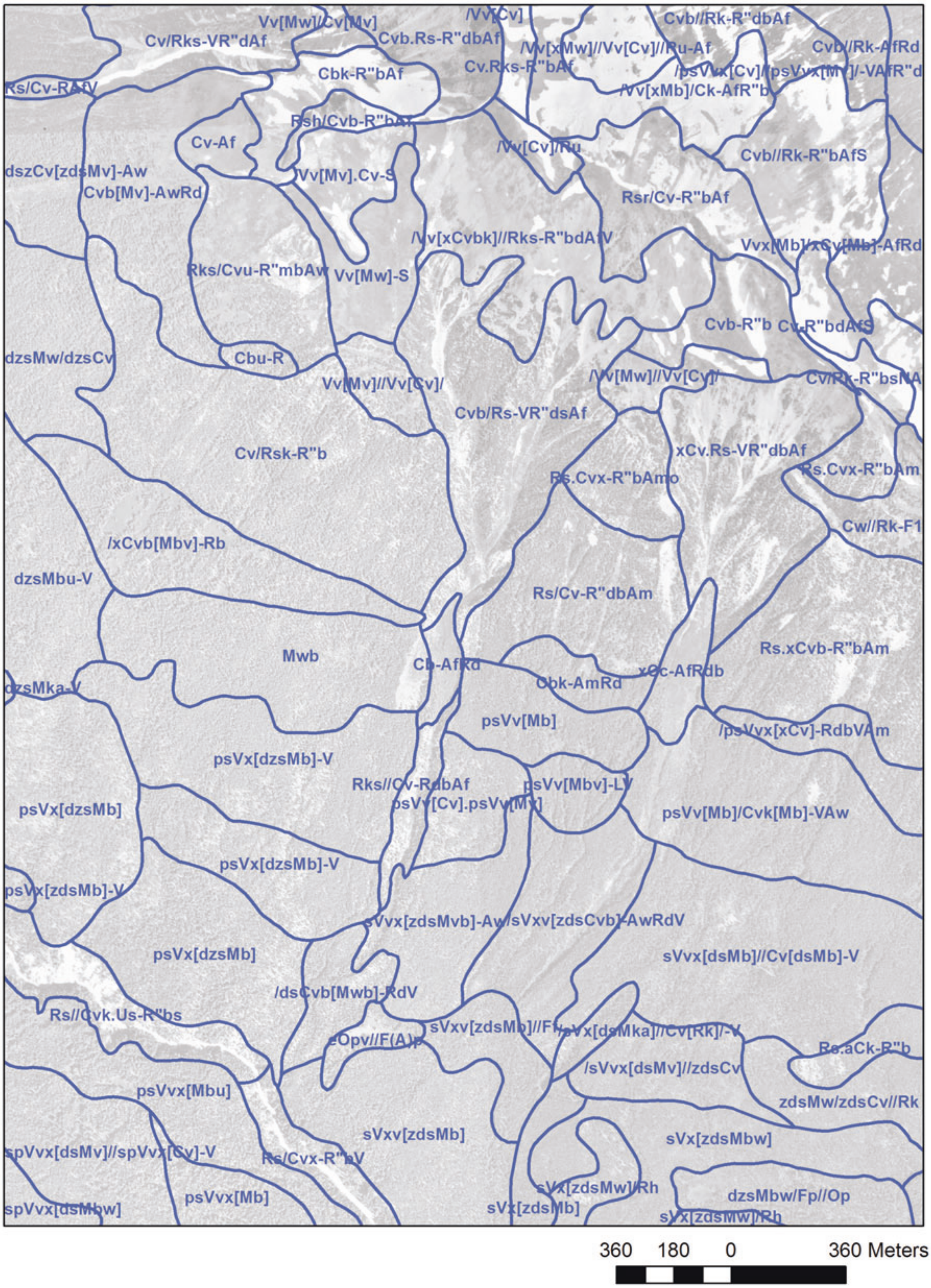
Relationship to Other Forms of Mapping

Many other forms of mapping have some relationship to or overlap in subject matter with engineering geomorphic mapping. These include surficial geological maps, lithological and structural geological maps, aggregate maps, and engineering geological maps. Geomorphic maps often contain information that is relevant to these other map types, and these other maps often include information that a geomorphic mapper will draw from.

The surficial geological map will show the occurrence of surficial geological units – that is – sedimentary deposits that have not been lithified. Surficial geology is often a primary rationale for polygon delineation in geomorphic mapping, especially in formerly glaciated landscapes. The geomorphic map will normally further define the landscape beyond what is normally done for a surficial geological map (e.g., by slope gradient, surface expression, and geomorphic processes).

Lithological and structural geological maps pertain to rock and rock processes. These maps overlap in content with the engineering geomorphic maps when the lithology or geologic structures represent significant drivers of geomorphic processes (see Cruden 2003).

Aggregate (often gravel) maps' emphasis is on the occurrence, extent and depth (to define a volume), and material properties of aggregate deposits. An aggregate map may also



Engineering Geomorphological Mapping, Fig. 3 Terrain stability map of the Bridge-Noel-Hurley landscape (J. M. Ryder and Associates (2001)). Mapping methodology follows British Columbia (1997, 1999) with slight modification to incorporate the use of a GIS define areas where certain aggregate products (e.g., paving grade aggregate) can be produced given the properties of the available aggregate. A good quality aggregate, or bedrock, source is integral to the success of most civil engineering projects.

Engineering geological maps will often contain many of the elements of an engineering geomorphic map to the point where a distinction between the two mapping efforts is not always clear. Engineering geology is the application of the geology discipline to civil engineering problems (Dearman 1991). The engineering geological maps are generally larger in scale and place more emphasis on the engineering properties of the underlying soils and rock.

Elements of Engineering Geomorphic Maps

Surficial Geology (Genetic Material)

Surficial geology is a common attribute of most engineering geomorphic maps, especially in areas that were formerly glaciated. The surficial geology will infer possible conditions that might be encountered in an area where development is being considered. For example, road construction across a silty lacustrine deposit may have to consider the potential of a soft base or a development-triggered landslide and may have to take measures to ensure that drainage does not result in unacceptable levels of soil erosion. Engineering geomorphic maps should also identify surficial geological units that underlie the surface unit (e.g., a fluvial fan which is underlain by glacial marine clay).

Geology

Geology is included in an engineering geomorphic map when lithology or geologic structures represent a significant geomorphic process driver. This can occur when persistent discontinuities, or bedding planes, disadvantageously intersect the topography, resulting in unstable slopes (see Cruden 2003). In areas that were not glaciated, thick weathered bedrock can represent a significant stability and erosion hazards, and the occurrence of weathered bedrock can provide a basis for geomorphic polygon delineation.

Slope

A basic element of an engineering geomorphic map is the delineation of polygons based on slope gradient. The slope gradient is the simplest consideration upon which to create terrain polygons, and the process can be automated using a Geographic Information System (GIS) with accurate Digital Elevation Models.

Slope gradients are often grouped into classes that reflect different likelihoods of associated hazards (snow avalanche, debris flow, landslide). The Terrain Classification System for British Columbia (1997) defined five slope classes: plain (0–3°), gentle (4–15°), moderate (16–26°), moderately steep (27–35°), and steep (greater than 35°). Slope classes can be defined based on legislative thresholds. These thresholds may require specific design elements to be included in a civil engineering project, or more detailed analysis be undertaken on slopes beyond a specified gradient.

Surface Expression

The surface expression will convey considerable information about the formative processes of a landform and potentially materials that comprise that landform. As such, it is often included as a mapping element. Surface form can also convey information on the possible hydrogeological regimes (e.g., convergent slopes will concentrate groundwater), which provides information about the potential engineering prescriptions that will be required when modification to the landscape is being considered.

Geomorphic Processes

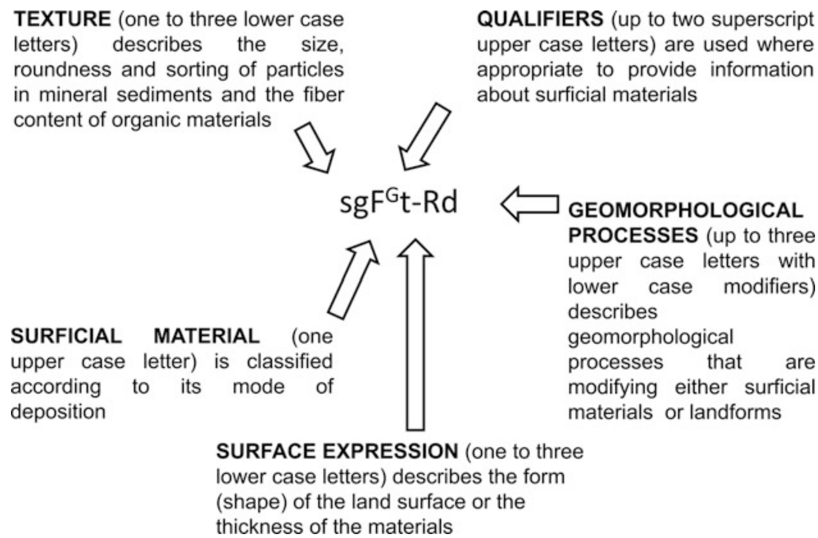
The geomorphic process is a description or a listing of the geomorphic process that could affect the site of a civil engineering development. These include existing and potential hazards from within the development site (e.g., soft foundation, unstable slopes, drainage erosion) and geomorphic hazards that could occur in an adjacent area that could affect the engineering development (e.g., a debris flow).

Examples and Case Studies

British Columbia Terrain and Terrain Stability Mapping

British Columbia (BC), Canada, has successfully developed and applied a predefined legend mapping methodology across its very large (945,000 km^2) and complex landscape (BC 1997; Resource Inventory Committee 1996). This methodology has also been applied in the Yukon Territory (482,000 km^2), Canada, with minor modifications (Lipovsky and Bond 2014). This terrain mapping approach is a procedurally comprehensive mapping methodology developed to describe the very complex and diverse landscape of BC, Canada. Its use could be extended well beyond the boundaries of the Province.

There are two primary documents which describe the methodology: Terrain Classification System for British Columbia (BC 1997) and Guidelines and Standards for Terrain Mapping in British Columbia (Resource Inventory Committee 1996). The Terrestrial Ecosystem Information Digital Data Submission Standards: Database and GIS Data Standards (Resources Information Standards Committee, 2015) is designed to be used in conjunction with BC (1997) and Resource Inventory Committee (1996). The BC terrain mapping approach has seen widespread use across Canada.



Engineering Geomorphological Mapping, Fig. 4 An example of a polygon label following the Terrain Classification System for British Columbia (1997). This label example indicates a sandy-gravel, glaciofluvial terrace, which is subject to a rapid debris flows

The Terrain Classification System for British Columbia (1997) provides a basis on which other mapping products are built off of including terrain stability mapping. The two principal documents for terrain stability mapping are Terrain Stability Mapping in British Columbia: A Review and Suggested Methods for Landslide Hazard and Risk Mapping (Resource Inventory Committee 1996) and the Mapping and Assessing Terrain Stability Guidebook (BC 1999).

The Terrain Classification System for British Columbia uses a predefined legend, where polygon information is provided by a standard series of symbols representing the surficial materials, surface expression, material texture, and geomorphic processes (Fig. 4). For each polygon, a minimum of the surficial material and material expression is required (e.g., Mb is a moraine blanket (blanket being >1 m thickness)). The approach also allows for complex polygons with up to three surficial materials types or stacked surficial units. Each polygon symbol must be unique from its neighboring polygons. Figure 5 provides a list of the terrain types and corresponding symbols. Figure 6 provides a list of surface expression terms and corresponding symbols. The reader is directed to BC (1997) for a detailed description of these terms.

Geomorphic process is indicated when a large area of a polygon is impacted by a geomorphic process or where there are a number of occurrences of one type of geomorphic process that are too small to map individually. The methodology also provides a number of geomorphic subclasses that can be used in conjunction with the geomorphic process symbols to further clarify the nature of the geomorphic process (e.g., a debris flow is indicated as Rd (R, rapid mass movement; d, debris flow)).

In addition to the polygon labels, the Terrain Classification System for BC (1997) includes a number of mapping symbols, which can be used to delineate geomorphic process. These symbols are used where the indication of the feature is deemed by the mapper to be important to the mapping purpose but where the feature is too small to be mapped as a separate polygon.

Material Name	*Map Symbol*
Anthropogenic Material	**A**
Colluvium	**C**
Weathered Bedrock (in situ)	**D**
Eolian Material	**E**
Fluvial Material	**F**
Glaciofluvial Material	$\mathbf{F^G}$
Ice	**I**
Lacustrine Material	**L**
Glaciolacustrine Material	$\mathbf{L^G}$
Morainal Material (Till)	**M**
Organic Material	**O**
Bedrock	**R**
Undifferentiated Materials	**U**
Volcanic Material	**V**
Marine Material	**W**
Glaciomarine Material	$\mathbf{W^G}$

Engineering Geomorphological Mapping, Fig. 5 List of surficial material terms and symbols used in the Terrain Classification System for British Columbia (1997)

The Terrestrial Ecosystem Information Digital Data Submission Standards: Database and GIS Data Standards (BC 2015) includes templates, validation tools, data dictionaries, and systems for managing project metadata. The corporate GIS that manage these datasets provide links to project reports and related data. It allows for projects across the Province of BC to be validated, analyzed, and interpreted. This allows for provincial layers and queries to be performed to make

Surface Expression Name	*Map Symbol*
moderate slope	a
blanket	b
cone(s)	c
depression(s)	d
fan(s)	f
hummock(s)	h
gentle slope	j
moderately steep slope	k
rolling	m
plain	p
ridge(s)	r
steep slope	s
terrace(s)	t
undulating	u
veneer	v
mantle of variable thickness	w
thin veneer	x

Engineering Geomorphological Mapping, Fig. 6 List of surface expression terms and symbols used in the Terrain Classification System for British Columbia (1997)

comparisons and to develop standardized interpretive products. The system also allows for project-specific attributes and codes to be added to the table. New attributes require a field name, definition, and data type. Numeric values can also specify an allowable range. Coded values must include a code, code name, and code definition. This has provided opportunity for adding new predefined elements to the database and allows a level of responsiveness to the classification system.

United States Department of Agriculture, Forest Service, Geomorphic Classification System

The United States Department of Agriculture (USDA), Forest Service, developed a hierarchical mapping method for use in all the US National Forests. The description of this methodology below was entirely taken from Haskins et al. (1998). The system is hierarchical, in that it defines landforms at different scales, as follows: *geomorphic process*, *landform*, *morphometry*, *and geomorphic generation.*

The system uses *geomorphic map units* to summarize areas of similar process and landform composition. A *geomorphic map unit* is a classification scheme used to display relatively homogenous areas of land. Each *geomorphic map unit* is unique in its composition. A *geomorphic map unit* will have a corresponding description that summarizes its *geomorphic process*, *landform*, *morphometry*, and *geomorphic generation* attributes, as well as any smaller landform inclusions of importance.

The *geomorphic process* describes the primary force acting on the landscape and can be further divided into *geomorphic process type* and *geomorphic subprocess*. The *geomorphic process type* is the broad geomorphic process responsible for landform genesis (e.g., fluvial, glacial, tectonic, etc.). The *geomorphic subprocess* is a more narrow description of the process type. For example, the mass wasting *geomorphic process type* can be further defined into fall, topple, slide, lateral spread, flow, or complex movement *geomorphic subprocess*.

A *landform* is simply a naturally formed feature on the Earth's surface characterized by a recognizable shape. It is directly connected with a single geomorphic process. An example is an alluvial fan A subdivision of the landform is the *element landform*.

Morphometry describes the shape, dimensions, and configuration of landforms. It is the measurable component. Indices of *morphometry* include relief, elevation, symmetry, slope gradient, drainage density, and so on.

Geomorphic generation identifies the process that formed each landform and the status of the process. The status can be active (developing), dormant (developed under different influences that are related to cyclic climate or tectonic forces), or relict (developed in previous geologic periods, where the process is unlikely to begin again).

The *geomorphic process*, *landform*, *and morphometry* attributes have been utilized in the USDA, Forest Service, Terrestrial Ecological Unit Inventory methodology (Winthers et al. 2005).

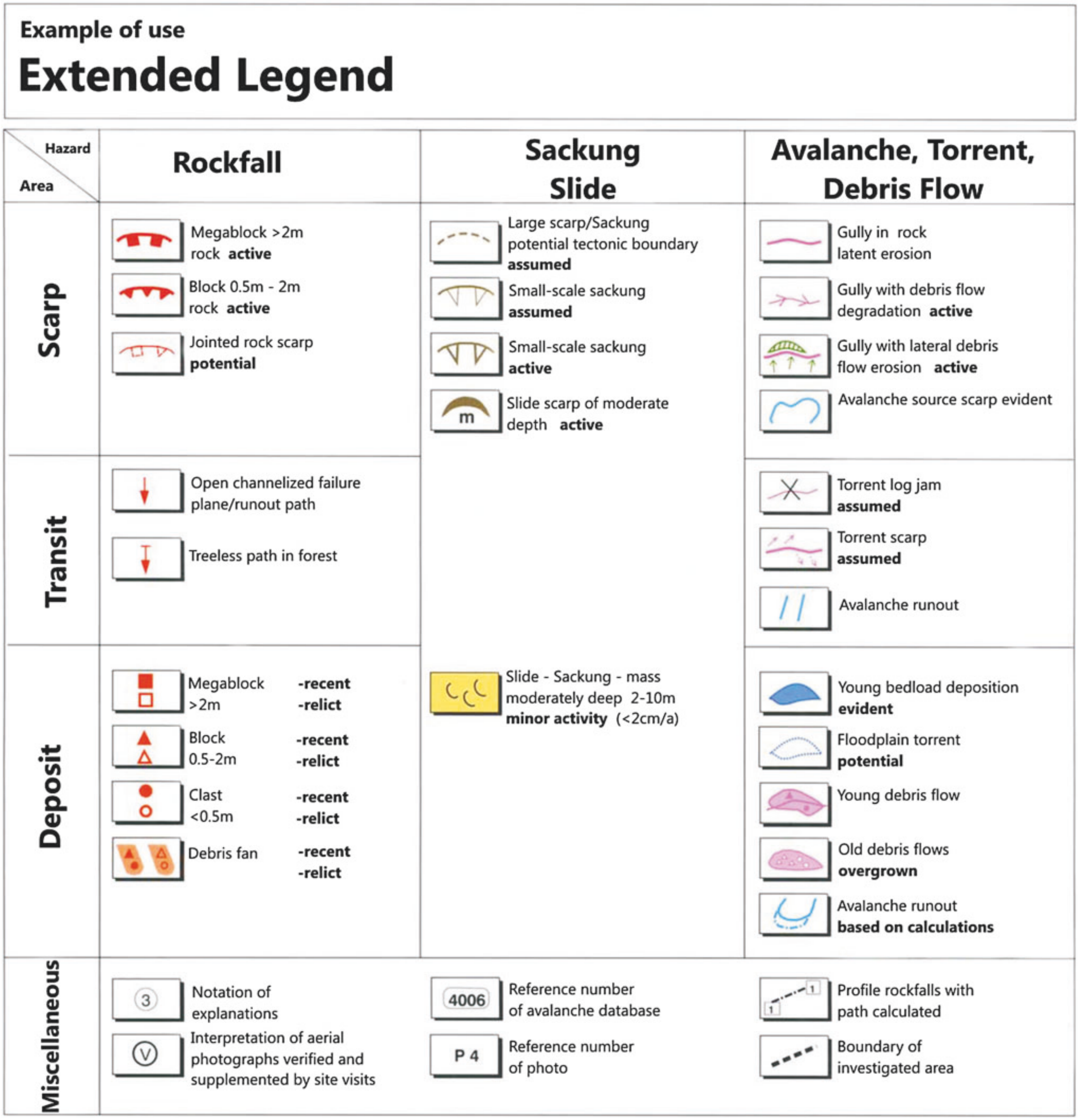
Australian Land-System Mapping

The land-system mapping methodology was initially developed in Australia by the Division of Land Resources and Regional Survey of the Commonwealth Scientific and Industrial Research Organization (DeWit and Bekker 1990) and was later applied in several other countries in Africa, Latin America, and Asia, following the adoption of the methodology by the United Kingdom Ministry of Overseas Development, Land Resources Division. In these countries, development planning was being hampered by a lack of appropriate scaled baseline data including topographic, geologic, and soils maps (Cooke and Doornkamp 1990).

The approach is hierarchical. Land-system polygons ranged in size from tens to hundreds of square kilometers in area, within which a repeated pattern of physiography, geology, geomorphology, soils, topography, and vegetation was evident (DeWit and Bekker 1990; Cooke and Doornkamp 1990). Each land-system polygon could be further divided into smaller, more homogeneous polygons, called land units. Indications of rare but important attributes of the larger polygon could also be included in the legend description. The mapping scale used in land-system mapping was typically 1:500,000 to 1:1,000,000 (Cooke and Doornkamp 1990).

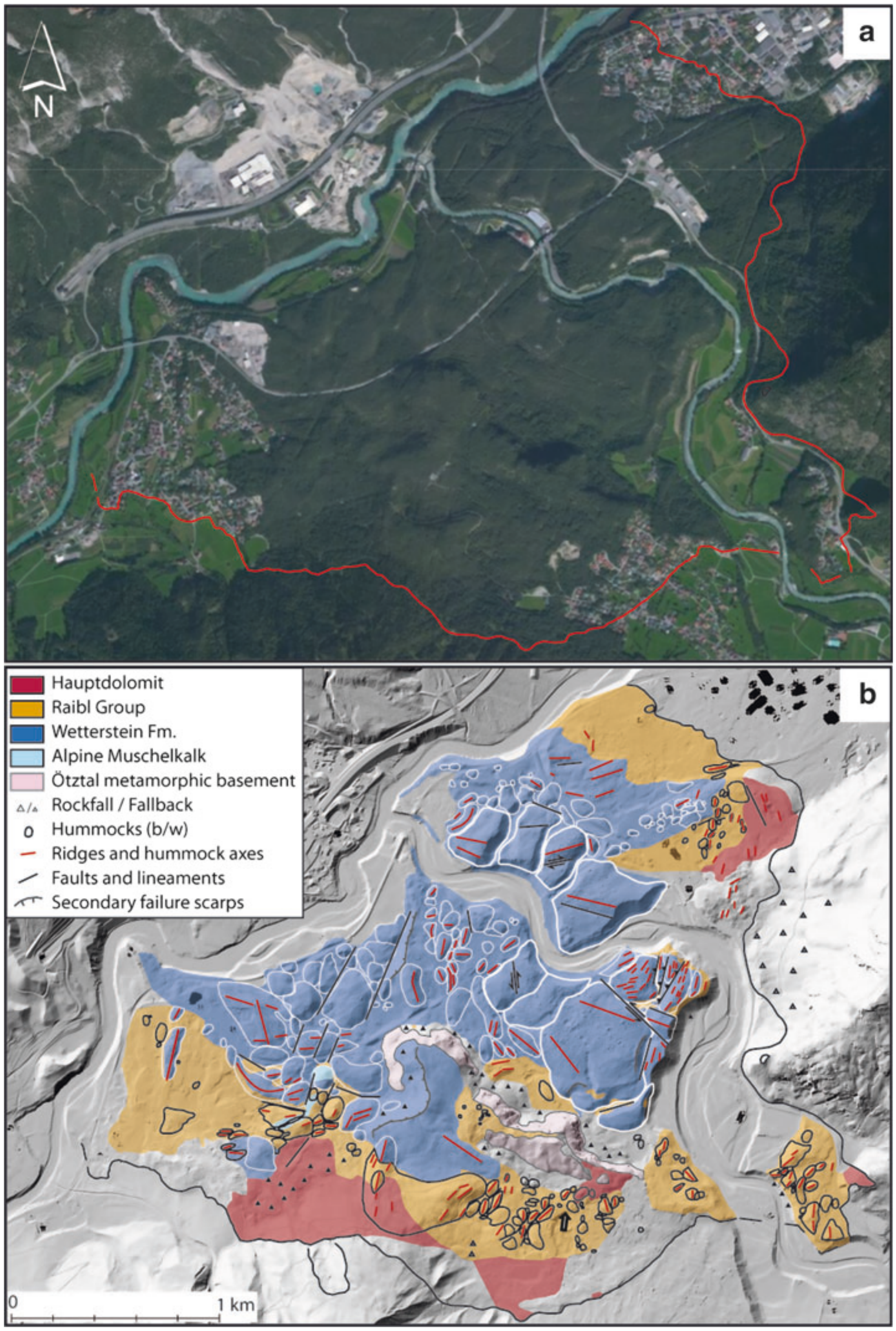
Perspectives from Austria, Germany, and Switzerland

Specific mapping guidelines may result from concerted regional case studies. Zangerl et al. (2008), for example, created process-oriented guidelines for mapping mass movement deposits following an interdisciplinary research project on



Engineering Geomorphological Mapping, Fig. 7 Mapping symbols for natural hazards as implemented by the Swiss Federal Office of Environment, Forest, and Landscape (BUWAL 1995). Two sets are suggested, one for general overview maps at smaller scales (*left*) and another for more detailed maps at 1:5,000, for example (*right*)

bedrock landslides in Tirol, Austria. These very comprehensive guidelines (thus far only available in German) encompass a broad spectrum of analyses, starting with field mapping of the unstable rock slope, aided by geophysics, borehole data, and volume calculations based on DEMs. The type of deformation (fall, slide, topple, flow, spread) and its spatial extent (e.g., location(s) of movement, number of moving units) are determined. Dynamic processes (i.e., time-dependent deformation) are monitored and integrated into what is already known from the previous analyses. Next, causal process chains between meteorological, hydrogeological, and mass movement processes are determined. Finally, a comprehensive model, including geometry, kinematics, material properties, time-dependent deformation, trigger, stabilizing and destabilizing factors, and numerical analyses, is constructed. Zangerl et al. (2008) describe four types of mapping involved in these guidelines:



Engineering Geomorphological Mapping, Fig. 8 (**a**) Digital orthophoto (2009; resolution 2–2.5 cm) of the Tschirgant rockslide-rock avalanche deposit (Tyrol, Austria); *red outline* shows mapped limit of the deposit. Only features larger than several 10s of meters (large hummocks and a bedrock ridge) can be crudely mapped due to vegetation covering obscuring any smaller features. (**b**) Lidar-derived hillshade image with 1-m resolution (flights between 2006 and 2010) shows the bare earth and facilitates geomorphic mapping at unprecedented scales Dufresne et al. (2016). Both orthophoto and lidar image are provided by the Federal Government of Tyrol (www.tiris.gv.at)

(1) geologic, (2) geomorphologic, (3) hydrogeologic, and (4) geotechnical mapping. These guidelines are aimed at providing a basis for effective planning of monitoring techniques and protective measures, as well as for predictive tools.

Hillslope risk assessment mapping by the Bavarian Geological Survey (LfU, Germany) also combines field mapping with results from kinematic and numerical modeling (e.g., LfU 2014). This approach has been applied throughout the entire state, resulting in a full coverage risk map of the federal state of Bavaria.

The Swiss Federal Office of Environment, Forest, and Landscape (BUWAL 1995) published a kit of mapping symbols for precise and unified documentation of dispositions, triggers, and effects of all potential natural hazards in Switzerland. For general overview maps, a “minimal legend” is given, whereas for maps created for specific purposes (e.g., reforestation or infrastructure projects), an extended legend is proposed (Fig. 7). The latter may be adjusted and upgraded as needed (BUWAL 1995).

Case Study: Rock Avalanche Morphometric Mapping for Emplacement Dynamics Reconstruction

Recent lidar surveys (flights between 2006 and 2010) by the Tyrolean government, Austria (Tiris 2015), provided high-resolution coverage of the entire province. Comparing generic aerial photographs of the study site (Fig. 8a) with lidar DEMs (Fig. 8b) shows the high value of the latter for morphometric mapping and consequently in reconstructing the dynamics of mass movement deposits. Using aerial photographs, only the very largest hummocks may be discerned roughly, whereas using lidar DEMs allows for the identification and mapping of details down to the decimeter scale. Many morphologic features can thus be mapped prior to field reconnaissance, better guiding field mapping, as well as planning and sampling strategies. Two emplacement modes could thus be identified from the map alone: linear rock sliding (large hummocks in spatial jigsaw-fit arrangement) and radial rock avalanche spreading (distributed smaller hummocks, partially aligned) (Dufresne et al. 2016). Whereas some standard symbols were used in places (Fig. 8b), such as the aforementioned line with hatch marks for (in this case secondary) failure scarps, other symbols were customized to best express processes that are not commonly captured on maps. Attention was paid to optimizing clarity in choice of color and symbol to enable a rapid overview of important geomorphic features relevant to landslide processes. For example, red lines were chosen for ridge crests, which, in their longitudinal extent, indicate rock avalanche motion direction and changes thereof. The same black lines were used for lineaments along topographic depressions regardless of the processes of formation. The rationales for using the same symbol were to (a) keep the map simple and legible and (b) allow for various interpretations of their origins open to local discussion provided in the accompanying manuscript and to developing insights into landslide processes (through, e.g., numerical modeling).

Summary

Engineering geomorphic mapping is used to assess the landscape for engineering purposes. Geomorphic mapping might be done to identify geotechnical and hydrotechnical hazards, to evaluate risk prior to or after development, or to characterize surficial materials and drainage patterns.

There are broad strategies used by engineering geomorphologists to achieve a project's mapping objectives. The process starts with clarifying the purpose and deciding on the scale and mapping methodology to be used. Both office-based and field-based mapping techniques are required to achieve an accurate depiction of real ground conditions. The office-based mapping will use aerial photographs, satellite images, or Digital Elevation Models and can involve either manual or automated mapping techniques. The field-based mapping can involve exposure, test pit, or drill core analysis, often supplemented with geophysical data. Field and office data can be analyzed, and mapping products can be generated using a Geographic Information System.

Cross-References

▶ Aerial Photography
▶ Engineering Geological Maps
▶ Engineering Geomorphology
▶ Geohazards
▶ GIS
▶ Lidar
▶ Photogrammetry
▶ Remote Sensing
▶ Risk Mapping

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Engineering Geomorphology

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Definition

Engineering geomorphology is the study of the Earth's morphological features and their processes of formation with special attention to their engineering properties and behavior aiming to provide solutions to complex problems and needs of engineers, development planners, environmentalists, and decision makers. It combines knowledge and methodological approaches from geomorphology, engineering geology, and geotechnical engineering considered as a branch of applied geomorphology.

Characteristics

The strong potential of this subdiscipline is determined by the combination of geomorphological site evaluation and description of dynamic processes with engineering characterization of deformation, strength, and hydrological properties of the involved materials. Thus it provides results suitable for the identification of engineering solutions. The geomorphological approaches include landform mapping combining field survey with interpretation of remotely sensed images (e.g., aerial photographs, satellite images) and digital elevation models prepared using various techniques (e.g., satellite image processing, LiDAR, structure from motion). Engineering geomorphology also uses the geomorphological knowledge of short and long-term landform dynamics in order to produce synthetic maps describing different environmental phenomena, whereas the engineering disciplines add description of their possible effects on anthropogenic structures and activities. Moreover, they bring in engineering approaches to characterize and assess the performance and hydrological properties of rocks and soils constituting respective landforms as a basis for engineering solutions. Studies focus on different landform systems (e.g., slope, river, coastal, and karst systems) with hazard assessment being usually the final step (Fookes et al. 2007) to provide a basis for mitigation of hazards and risks, often using spatial capabilities of GIS.

Engineering geomorphology is important in meeting new challenges of a changing climate and anthropogenic landscape changes and is vital for human development in Arctic and high mountain regions (Giardino and Marston 1999) that are subject to frequent mass movements and require river system management.

Cross-References

▶ Designing Site Investigations
▶ Engineering Geological Maps
▶ Engineering Geomorphological Mapping
▶ Environmental Assessment
▶ Geohazards
▶ Geotechnical Engineering
▶ Landforms
▶ Landslide
▶ Mass Movement
▶ Surveying

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Engineering Properties

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Synonyms

Biological characteristics; Chemical properties; Physical properties

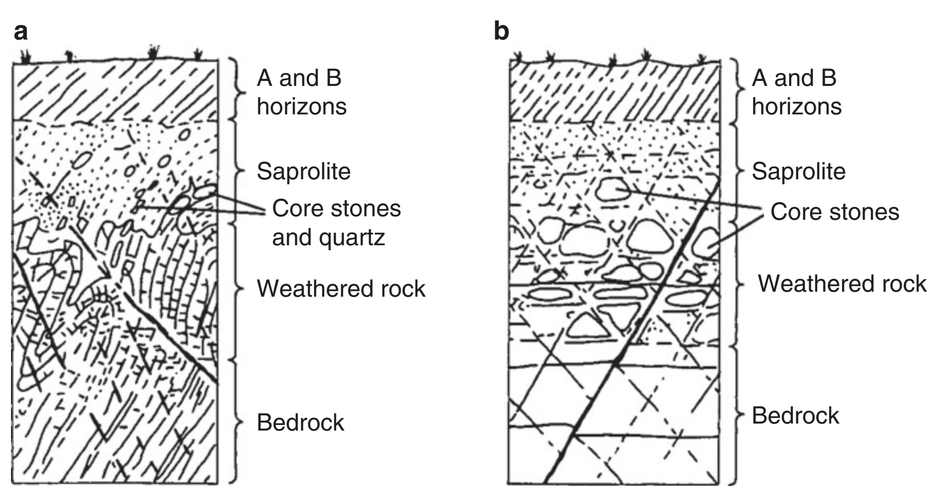
Definition

Engineering properties, considered broadly, are physical, chemical, and biological characteristics that are observable, measureable, and influence behavior to the extent that they are important in engineering analyses and design, and in performance of components, systems, or processes.

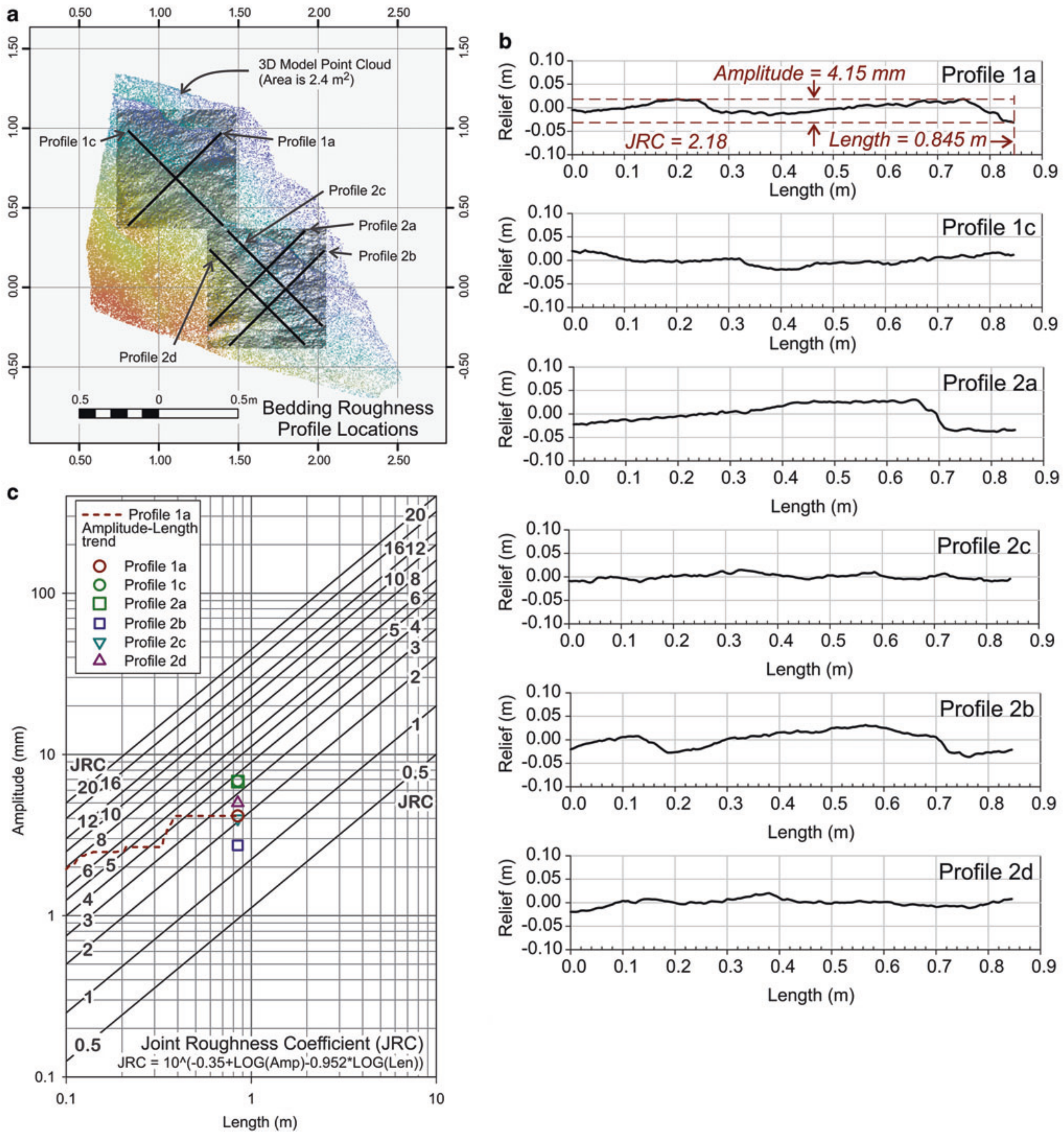
Introduction

Properties of soil and rock, more so than properties of manufactured materials, such as steel and concrete, are important in engineering geology. In most cases, the *in situ* properties of the materials are important in engineering analysis and design; however, applications such as compacted fill and riprap require properties of reconstituted masses or excavated fragments. Subsurface soil or rock properties can be determined during field investigations with *in situ* testing, during laboratory programs with tests on samples collected in the field, or by estimation based on back-analysis of field performance and measurements. Soil and rock properties are controlled by geologic conditions that can be expressed with geologic models that include the origin of deposits and formations, and the history of deformation, alteration, weathering, and erosion (Fig. 1). Engineering properties may vary considerably from point to point within individual strata, and almost certainly from strata to strata or formation to formation at a site being characterized. For these reasons, values of properties used to characterize a site should not be averaged across multiple strata (NYSDOT 2013).

Typically, standardized expressions of properties are desirable so that terms carry the same meaning from site to site and geologist to geologist. Weathered rock in the diagrams in Fig. 1 is described by Obermeier and Langer (1986) as having more than 10% core stones that are sufficiently strong and durable that a hammer is needed to break them; saprolite retains relict features of bedrock structure in material that is completely weathered to soil-like consistency, containing less than 10% core stones. The weathered rock and saprolite descriptions are functional and meaningful but are not referenced to a classification system, such as USBR (1998), USACE (2001), Sabatini et al. (2002), or those mentioned in USACE (1994), which notes that each project may need site-specific zoning or rock mass classification because rock mass discontinuities take on different significance at different places in a project depending on orientations of slope surfaces or tunnel alignments.



Engineering Properties, Fig. 1 Diagrams illustrating typical weathered profiles on crystalline rocks of the Piedmont Province, Fairfax County, Virginia, USA. (**a**). Foliated metasedimentary rocks. (**b**) Massive metamorphic and igneous rocks (From Obermeier and Langer (1986, Figure 8))



Engineering Properties, Fig. 2 Example of direct measurement of bedding-plane roughness using a point cloud on a bedding surface in sandstone. (**a**) GIS view of the bedding surface showing locations of six profile lines in perpendicular directions; coloration in the point cloud represents relief in the z-value of 3D model in arbitrary coordinate space approximately perpendicular to the bedding plane. (**b**) Lines of relief for each of the six profile lines; relief is the residual of z from a linear regression calculation of point cloud z-values to de-trend the z-profile. (**c**) Plot of joint roughness coefficient (JRC) on an amplitude-length graph; the amplitude-length trend of Profile 1a is shown with the short dashed red line demonstrating that the JRC value for a short length generally is higher than for a long line

Index Parameters and Performance Parameters

Engineering properties of soil and rock can be grouped into index properties and performance properties. Index properties for soil include unit weight, water content, grain size distribution, porosity, plasticity and liquidity (collectively known as Atterberg Limits), specific gravity of mineral solids, organic content, grain shape, pH, and chemical content. The Unified

Soil Classification System (ASTM 2011) is based on grain size distribution and Atterberg Limits results, making it a hybrid representation of an index for soils. Index properties for rock material include petrographic analyses, unit weight, absorption, porosity, punch-penetration index, and Cerchar abrasivity index.

Performance properties of soil include unit weight, shear strength, compressibility, ultrasonic velocity, hydraulic conductivity, electrical resistivity, dielectric constant, thermal conductivity, shrink-swell characteristics, and type and concentration of minerals that react corrosively to buried metal and concrete. Performance properties of rock material include the same properties listed for soil, as well as point load strength, unconfined compressive strength, tensile strength, Young's modulus, Poisson's ratio, shear modulus, solubility, slake durability, abrasion resistance, and crushed or broken-fragment shape. Performance properties of rock masses include discontinuity orientation, discontinuity length and spacing, and discontinuity surface condition. Discontinuities consist of bedding planes, joints, fractures, shear zones, faults, and lithologic contact surfaces. Discontinuity surface conditions consist of planarity, roughness, aperture, filling material, and wall-rock compressive strength. For some projects, mineralogy and chemical characteristics, such as pyrite or other sulfide minerals, are important for evaluating the potential for seepage to be strongly acidic.

Properties of rock masses tend to be controlled by the discontinuities in conjunction with the shape and size of excavations or constructed works to be made in or supported by the rock mass. The amplitude-length method of calculating the joint roughness coefficient (Barton 2013) is used as an example of an engineering property of a rock mass (Fig. 2) that tends to vary with scale and may vary with direction, as well as with position on the joint or bedding plane.

Conclusions

Engineering properties of Earth materials are important for engineering geologists to know or to be able to find in reference books or other resources. This knowledge enables engineering geologists to communicate effectively with engineers and other non-geologists about geologic aspects of site characteristics that control performance of the ground under different existing or proposed land uses.

Cross-References

- ▶ Angle of Internal Friction
- ▶ Atterberg Limits
- ▶ Bulk Modulus
- ▶ Characterization of Soils
- ▶ Classification of Rocks
- ▶ Classification of Soils
- ▶ Coefficient of Uniformity
- ▶ Conductivity
- ▶ Density
- ▶ Dilatancy
- ▶ Durability
- ▶ Elasticity
- ▶ Liquid Limit
- ▶ Mechanical Properties
- ▶ Modulus of Deformation
- ▶ Modulus of Elasticity
- ▶ Plastic Limit
- ▶ Plasticity Index
- ▶ Poisson's Ratio
- ▶ Rock Field Tests
- ▶ Rock Laboratory Tests
- ▶ Rock Mass Classification
- ▶ Shear Modulus
- ▶ Shear Strength
- ▶ Shear Stress
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Mechanics
- ▶ Velocity Ratio
- ▶ Young's Modulus

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E

Environmental Assessment

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Definition

A process for considering the environmental effects of a project on a par with the projects' technical and economic feasibility (Scheuer 2005; EPA 2016)

Characteristics

The environmental assessment process was explicit in the National Environmental Policy Act (NEPA) enacted by the US Congress in 1969 and signed into law on January 1, 1970. The purposes of NEPA were declared to include:

> To declare a national policy which will encourage productive and enjoyable harmony between man and his environment; to promote efforts which will prevent or eliminate damage to the environment and the biosphere and stimulate the health and welfare of man; to enrich the understanding of the ecological systems and the natural resources important to the Nation.... Sec. 2 [42USC § 4321] (EPA 2016)

Similar provisions to assess developmental measures for their impact on the environment have since been adopted in many other countries. In 1973, the European Union initiated efforts to provide a framework within which the member state develops and enforces applicable environmental laws including environmental assessment (Scheuer 2005). The European Union adopted measures for environmental assessment having a broader effect than those of the United States. As exemplified by the Scottish Environmental Assessment Act of 2005, assessment is required of "All Scottish public bodies and a few private companies operating in a 'public character' (e.g., utility companies) within Scotland..." (Scottish Government 2017). In this respect, environmental assessment is similar to that undertaken under NEPA within the United States. The more important difference is the application of the European Union environmental assessment requirements, as illustrated by the Scottish example, to "assess, consult and monitor the likely impacts of their plans, programmes and strategies on the environment." This is a much broader application of environmental assessment which is termed as Strategic Environmental Assessment (SEA) (Scottish Government 2017).

Nationally, NEPA is applied to actions affecting federally managed lands or being projects fully or partially funded by the Federal government (EPA 2016). The environmental consequences of the proposed action are documented in either an environmental assessment (EA) or an environmental impact statement (EIS) (EPA 2016). The National Environmental and Planning Agency of Jamaica requires environmental assessment as part of the process for issuing permits for a specified list of enterprises, construction, and development within prescribed areas (NEPA 2017). Because their National Resources Conservation Authority Act of 1991 also identifies the entire island of Jamaica as a prescribed area, permits are effectively needed for all the listed elements (NEPA 2017). Whereas environmental assessments in the United States generally affect activities involving the Federal government, environmental assessment in Jamaica tends to focus on subject actions being undertaken in the private sector. So, a fundamental difference in environmental assessment among different countries is how broadly it affects developmental actions and its application to the public and private sectors.

However, to produce an environmental assessment satisfying the legal requirements embodied in law and regulation requires having guidance documents detailing how to conduct and document the impact of a project and its alternatives. Whether carrying out an environmental assessment under the authority of NEPA in the United States or for a strategic environmental assessment (SEA) in a European Union country such as Scotland, the guidance requires scoping for issues and consulting with the affected stakeholders. Consequently, an important aspect of carrying out an environmental assessment is an emphasis on public involvement enabling access by individual citizens and public interest groups into the process (Scheuer 2005; EPA 2016; Scottish Government 2017). Public scrutiny tends to sharpen the details put forth in describing and defining projects and their alternatives.

As noted earlier, the requirement to undertake an environmental assessment of a project and its alternatives will apply to different entities depending on laws and regulations for the jurisdiction where the project will take place. NEPA placed the requirement on all branches of the US government prior to any major federal action that significantly affected the environment (CEQ 2007; EPA 2016). Actions are broadly understood to include the issuance of permits or similar authorizations needed by a project, providing direct funding, and projects actually undertaken by federal agencies as part of their authorized work. Many individual states instituted similar environmental assessment requirements calling for environmental assessment for actions undertaken by state government entities. For instance, California was inspired by the passage of NEPA to enact the California Environmental Quality Act (CEQA) (CNRA 2016).

Components of Environmental Assessment

The environmental effect of a project can manifest in three different ways. It may be a direct effect. An example might be constructing a reservoir on a river that empties in the ocean. The reduced water flow from the reservoir may be insufficient for the downstream fish populations to continue to thrive. The environmental effect may be indirect. Once that same reservoir is built, less sediment is carried by the river to the ocean. Consequently, currents running along the coast begin to erode beaches up the coast from the mouth of the river. The third environmental effect is cumulative. If two or three more reservoirs are constructed upstream from the first reservoir, there may be years when the surface runoff of the drainage basin is nearly all stored in the upper reservoirs leaving the lower one with insufficient volume to continue releases to maintain downstream fish populations. For each alternative being considered in an environmental assessment of a project, the direct, indirect, and cumulative effects should be determined (CEQ 2007).

While the examples demonstrate negative impacts of the reservoir, it should be recognized that a project may also have positive effect. For example, the multiple reservoirs in the earlier example may lead to more groundwater recharge in the vicinity of each reservoir than would take place if the river was free flowing. Consequently, the direct, indirect, or cumulative effects of any project alternative on the environment can be either negative or positive. It should also be evident that assessing environmental effects will require an interdisciplinary team of specialists to make these determinations. A reservoir project might (1) provide an opportunity for motorboat recreation, (2) supply water to power a hydroelectric plant, (3) reduce spawning gravel used by fish populations downstream, (4) require migrating deer herds to swim to continue their annual journey, or (5) induce slope instability in adjacent valley walls. The interdisciplinary team for this hypothetical project should clearly have, at least, a recreational specialist, a hydroelectric engineer, a wildlife biologist, an aquatic biologist, and a geologist. It might be argued that consultancies could address each of these issues and provide the effects information to a smaller team. However, this multidisciplinary team would lack the ability to collaboratively and thoughtfully examine possible means for balancing certain adverse effects while maximizing beneficial ones. An interdisciplinary team can benefit when consultancies facilitate making sufficient data available for use by a particular team specialist.

Under NEPA, an initial evaluation of environmental effects for a project will not only identify and describe the direct, indirect, and cumulative effects that may arise but also their significance (CEQ 2007). Despite having identifiable effects on one or more aspects of the local environment, it is possible that the initial environmental assessment may yield a finding of no significant effect for the action being proposed. This finding usually would be for only one of the alternatives and may require adding mitigations or making design modifications to reduce or eliminate one or more of initially identified adverse effects. Such an alternative would be identified as the preferred one. An environmental assessment report would document the alternatives, data, and analysis along with the basis for the no significant effect finding (CEQ 2007). The preferred alternative would then move forward as the basic project design with the environmental assessment being completed.

If the initial environmental assessment (EA) produces a significant effect finding, the project would require a much more extensive evaluation of environmental effect in the form of an environmental impact statement (EIS) (CEQ 2007). Preparing an environmental impact statement would also require an interdisciplinary team. However, the team might be enlarged by adding specialists who can examine the critical environmental concerns in greater detail. This detail often requires additional data gathering for completeness or to resolve uncertainty identified during the initial assessment. Like the initial environmental assessment, the environmental impact state will include description of the affected environment or project area and the proposed action alternatives. There is always a no-action alternative which essentially describes what will continue to be the situation should no-action alternative be instituted (CEQ 2007). The no-action alternative serves as a baseline for comparing the degree to which any action alternative does or does not achieve the stated purposes of the project.

The environmental impact statement is commonly issued in draft form to solicit public comment and mandatory review from various governmental entities (CEQ 2007). This input is used to identify a preferred alternative in the final environmental impact statement. The document will also identify other alternatives which were not considered in detail under the environmental impact statement and the reasons for their exclusion from consideration. Whereas the environmental impact statement contains all the data and analyses leading to the identification of a preferred alternative, a separate document termed a record of decision will summarize the analysis and conclusions leading to the selection of a particular alternative (CEQ 2007). It is this document, rather than the environmental impact statement, where the decision on how the project is to proceed is made. The record of decision will also summarize the environmental impacts associated with other proposed alternatives, the no-action alternative, and those alternatives not considered in detail.

There will be ongoing informal and formal communication with the public during an initial environmental assessment and an environmental impact statement, should one be needed (CEQ 2007). Communication will focus on informing affected or potentially affected groups about the project. Regulations derived from NEPA and similar state laws have requirements for public notification about project actions and public input opportunities (CEQ 2007). Early in the process, there are often public meetings or other forums where the public directly or through elected officials can express concerns about possible environmental effects. This public input often identifies economic, safety, and other concerns in addition to environmental effects. Depending on the size or scope of project, the public affected by a project may include regional or national groups in addition to local groups and residents. Efforts to communicate and resolve concerns raised by the public as individuals or groups do not preclude their taking legal action should the final decision about a project alternative not be to their liking. Milner et al. (2010) provide an illustration of this point related to land management of national forest in the United States.

An Example of Engineering Geology in Environmental Assessment

In 2006, reactivated movement of the Ferguson Rockslide caused rock to bury part of California State Highway 140 along the Merced River, one of the west-draining rivers of the Sierra Nevada, California (Harp et al. 2008). This section of the Merced River is within a steep-walled canyon about 10 km west of an entrance to Yosemite National Park. After several months, one-way traffic with limitations on vehicle size was restored by a detour to the opposite side of the river via two temporary bridges. The reopened route constrained normal traffic use along one of the 3 year-round routes into Yosemite National Park and the only one without tunnels or subject to winter storm closure (Harp et al. 2008). Ongoing monitoring indicated continued movement on the rockslide for several years (De Graff et al. 2015).

The California Department of Transportation (Caltrans) served as the lead agency for the Ferguson Slide Permanent Restoration Project. The stated purpose was "…to reopen and restore full highway access between Mariposa and El Portal via State Route 140. Full Highway access for this portion of State Route 140 means a two-lane, all-weather highway that would accommodate all types of vehicles with some restrictions on vehicle length. The route would return to its previous status as a California Legal Advisory Truck Route with a 32-foot kingpin-to-rear-axle restriction (Caltrans 2014a)." Because this was a joint project between Caltrans and the Federal Highway Administration (FHWA), the environmental assessment had to satisfy the requirements of both the National Environmental Protection Act (NEPA) and California Environmental Quality Act (CEQA) (EPA 2016; AEP 2014).

The location of the project within a designated segment under the national Wild and Scenic Rivers Act (WSRA) and the scope of potential construction, any action alternative, would potentially have a significant effect on the environment. This recognition resulted in undertaking an environmental impact statement. Obviously, a project involving highway construction and restoration of a landslide-impacted area would have a variety of geologic issues requiring input from engineering geologists. Some input would be specific to foundation and excavation aspects influencing the design of alternatives. Other engineering geology data were related to the impact on and by the flow of the Merced River and the stability of the rockslide material blocking the existing roadway. How to dispose of excess rock during excavation was another important consideration. Less obvious was the need for geologic input for analysis of the project's impact under the Wild and Scenic Rivers Act. This need for input arose from the scenic designation along this segment of the Merced River being based partially on its geologic significance as documented in Appendix I (Caltrans 2014a). Based on the environmental impact statement, a decision was made to restore State Route 140 using the alternative which called for building a rock shed (cut and cover tunnel) through the talus deposited by the Ferguson Rockslide along the existing road alignment (Caltrans 2014b).

Summary

Environmental assessment is a process for considering the effect of a project on the environment on equal terms with engineering feasibility and economic considerations. The way the process is applied to a project and requirements for analysis and documentation will be dictated by the laws and regulations applicable to both the nature of the project and the jurisdiction where the project is proposed to be implemented. It is especially important to understand the laws and regulations that are applicable. While the preceding paragraphs describe the approach mandated in the United States, requirements can differ for other countries. As Scheuer (2005) explains, the European Union has overall environmental targets which serve as a cap to what individual members might mandate. In general, the physical environmental issues will involve surface and groundwater, soil and rock suitability and capability, air quality, landscape considerations, and public safety. Engineering geology should play a role in generating the data and making the analyses for these issues. This will commonly be a part of an interdisciplinary team to ensure completeness of the potential environmental impacts and collaboration with other technical specialists to produce mitigation and enhancements resulting in a project that best satisfies its purpose and need.

Cross-References

► Beach Replenishment
► Coastal Environments

▶ Desert Environments
▶ Fluvial Environments
▶ Glacier Environments
▶ Lacustrine Deposits
▶ Land Use
▶ Mountain Environments
▶ Nearshore Structures
▶ Reservoirs
▶ Tropical Environments
▶ Tunnels
▶ Volcanic Environments
▶ Waste Management

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Environments

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Synonyms

Aquatic; Arid; Coastal; Fluvial; Humid; Lacustrine; Marine; Mountainous; Terrestrial; Urban environments

Definition

Natural systems characterized by specific combinations of physical, chemical, and biological properties and interactions and reflecting impacts of human activities to varying extents (Warren 2001). In urban environments, human activities dominate over but usually, to some extent still reflect, the surrounding natural environment.

Environments are terrestrial or aquatic or reflect the interfaces between these types. Some are extensively modified by human activity (urban, suburban, or agricultural and mining land uses) but others are not.

Terrestrial environments (about 28% of the Earth's surface) are defined by a combination of:

- Temperature – polar, temperate, subtropical, or tropical;
- Humidity – arid, semi-arid or humid
- Geomorphology (e.g., mountains, foothills, plateaux, plains, and lowlands).

These influence the distribution of **ecosystems** which include polar (tundra), subarctic forests (taiga), temperate deciduous forest, temperate rain forest, tropical rain forest, grasslands, and deserts. Variations in climate affect the boundaries of these zones (Fig. 1) (Chapin et al. 2012; Gupta 2011).

Aquatic environments (about 72% of the Earth's surface) are:

- Nonmarine (fluvial, lacustrine)
- Coastal or
- Marine (with environments reflecting the dynamics, salinity of water)

Marine environments (Fig. 2) have saline water, whereas fluvial and lacustrine environments usually have less dissolved salts ("fresh water"), but confined bodies of water subjected to solar evaporation may become saline or hypersaline (Duursma 2002).

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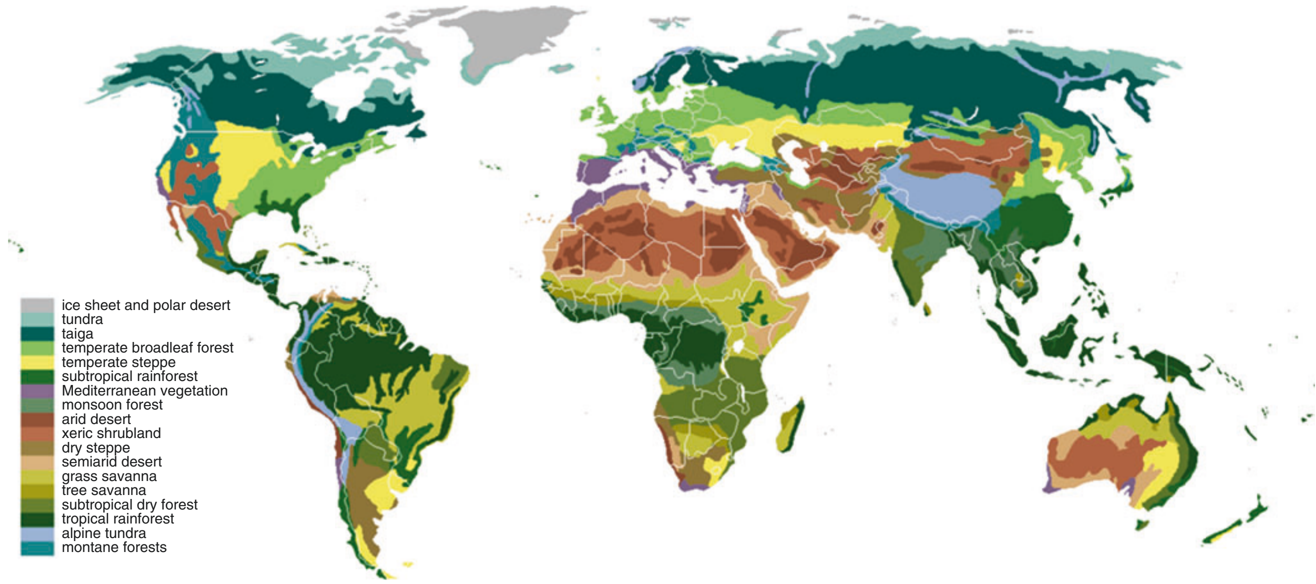
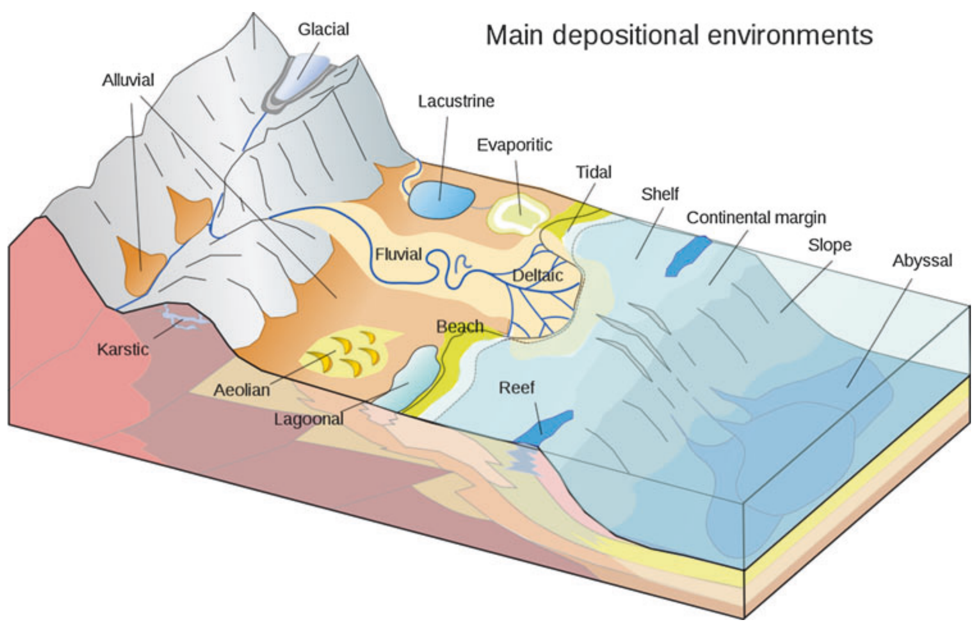


Environments, Fig. 1 Terrestrial environments based on a combination of physical, climatic and ecosystem characteristics (Source: https://commons.wikimedia.org/wiki/File:Vegetation.png)

Environments, Fig. 2 Some coastal and marine environments (Source: Wikimedia. Creative Commons Attribution Share Alike 3.0 Unported Licence. Principales medios sedimentales)



Coastal environments (e.g., beaches, estuaries, deltas, tidal flats with salt marshes, and mangrove swamps) commonly have intermediate salinity which fluctuates tidally. These are significantly affected by sea level change due to climate change, human intervention, and storms (Duursma 2002).

Urban environments are characterized by buildings, roads, parking areas, and playgrounds. Developed land is concentrated in central areas, whereas there tends to be more "greenspace" in suburban areas. Urban environments are influenced by the overall climate but generally have microclimates due to high energy use, associated emissions, and the sheltering effect of clusters of buildings (de Mulder et al. 1996; Bell 2004).

Categories usually have transitional boundaries.

Relevance to Engineering Geology

All environments exhibit a wide range of engineering geological issues (Table 1).

Environments, Table 1 Factors of significance to engineering geology in selected categories of environments

Category	Comments
Tundra	Seasonal freezing and thawing above permafrost; need to avoid warming the ground in winter and supporting structures in summer; methane generation from organic layers after thawing
Mountainous and hilly terrain	Slope instability (rockfall, avalanche and landslides); permafrost high altitudes and latitudes; flash flooding in narrow valleys
Lowlands	Fluvial flooding near major rivers. Poorly consolidated sediments sometimes with weak soils (e.g., peats) but also potential for sand and gravel for construction uses
Arid areas	Wind erosion and dust emissions; migratory dunes; saline ground and surface waters near ephemeral rivers, confined lakes, and coastal plains (sabkha)
Humid tropics	Deep weathering and deep engineering soils; original geology partly obscured; residual corestones leading to uneven ground conditions; high landslide potential on slopes; fluvial flooding during heavy rainfall
Urban environments	Complex anthropogenic deposits that are often an archaeological resource but have ground problems including: cavities (e.g., cellars, tunnels, wells, mines); old foundations; contaminated soils (harmful elements and compounds and gases); and polluted surface and groundwater. Sit investigation and interpretation may be difficult. Increased water run-off due to impermeable surfaces
Coastal environments	Effects of storms and tsunami; decisions on coastal defense works or managed retreat. Tidal flats and mangrove swamps with weak sediments and organic layers that can lead to methane generation
Marine environments	From the engineering geology standpoints mainly involving construction in shallow waters (e.g., jetties, piers) or in deeper continental shelf waters (e.g., oil exploration and production platforms). Principal issues relate to thick poorly consolidated sediments and hydrodynamics storms, currents, erosion)

Cross-References

- ▶ Coastal Environments
- ▶ Desert Environments
- ▶ Fluvial Environments
- ▶ Glacier Environments
- ▶ Marine Environments
- ▶ Mountain Environments
- ▶ Tropical Environments
- ▶ Vegetation Cover

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Equipotential Lines

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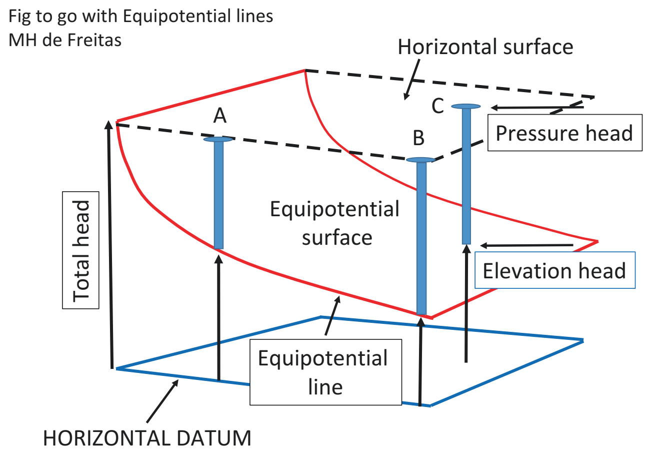
Definition

The trace of an equipotential surface on a 2D plane through a body of flowing groundwater, normally seen either in vertical section or in plan, on which total head at all points along it is the same.

Flow of groundwater can be described with reference to its "Head"; this is the height of the water as measured above a stated datum. This height is directly proportional to the work that has been done on the water to get it to this level and thus the water's ability to do work in return. Bernoulli recognized that in a field of flow three measurements of head, at any chosen point within it, were required to quantify and so explain the ability of the water at that point to do work, that is, to flow. They are (1) its *elevation head*, which is the height of the water at that point above a selected datum, such as sea level, because this height is proportional to the water's potential energy at that point, that is, its ability to do work by virtue of its position; (2) *pressure head* at that point, which is the height to which the pressure of water at this elevation (i.e., at its elevation head) will raise a column of the same water; this adds to its potential energy; and (3) *velocity head*, which is the height to which the momentum of water at that elevation and pressure, will raise a column of the same water and is proportional to its kinetic energy. Elevation and pressure head are measured in the ground using a piezometer and are usually many meters, possibly tens of meters and more in dimension; by contrast, the velocity head in ground water is usually so small (in the millimeter range) that it can be ignored unless dealing with the flow through screens around wells and sumps, where it can account for meters of head loss or with the erosion of fine particles from within pores and fractures.

The sum of these three heads, or most usually in ground water studies, the sum of the elevation and pressure head,



Equipotential Lines, Fig. 1 The relationship between an equipotential line and the equipotential surface to which it belongs, and to the heads that define it

equals the *total head* of the water relative to the datum from which its elevation head was measured; for groundwater studies this datum is usually sea level.

Water flowing through the ground will have at each point in the ground a value of total head particular to that point. Points in the ground where the total head has the same value can be thought of as defining a surface in the ground on which total heads are equal; such a surface is called an equipotential surface, that is, a surface on which water at each point on that surface has the same total head.

Figure 1 illustrates one such surface. Three boreholes are shown (*A*, *B*, *C*) in which the pressure head at any depth can be measured. The elevations at which the pressure heads in each hole produce an identical total head (shown by the *horizontal surface*) define an equipotential surface. An equipotential surface is described by its total head (so many meters above datum). Note that different combinations of elevation and pressure head can produce the same total head and this is why the elevation of a piezometer tip, which is the part of a piezometer used to measure pressure head in a borehole, must be recorded. An equipotential line is the trace such a surface makes when intersecting a 2D plane, such as a geological cross section or a map; it is described by the total head of the equipotential surface to which it belongs.

Bernoulli (1738) worked with water and so his findings relate to nonviscous and incompressible fluids. His principles remain the basis for describing and quantifying the flow of all fluids.

Cross-References

▶ Aquifer
▶ Artesian
▶ Groundwater
▶ Groundwater Rebound
▶ Hydrogeology
▶ Hydrology
▶ Piezometer
▶ Water

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Erosion

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Definition

Physical erosion is the removal of surficial and near-surface soil, sediment, and rock particles from their source and their relocation down slope by gravity and transporting agents, water, wind, and ice. Chemical erosion involves the dissolution and transport of soluble minerals. Erosion lies on the continuum between chemical and physical weathering and transport/mass wasting. Although erosion involves transport and mass wasting, these are usually considered to be separate processes.

Introduction

Erosion is effected by the action of geomorphic drivers, such as rainfall; bedrock wear in rivers by abrasion and scour; coastal erosion by the sea and waves; glacial plucking and areal flooding; wind abrasion; groundwater (internal) hydraulic pressure; and submarine currents and turbidity flow processes. Nonglacial, subareal slope erosion is most common and is most active on steep slopes composed of weak rocks or soils, in semiarid climates (less rainfall moves less material and more rainfall induces plant growth which inhibits erosion), where vegetation has been removed (Blanco and Lal 2010), in areas of active tectonic uplift, and in coastal sites where surf and/or tides are high and sand supply is diminishing.

Excessive water and wind erosion are responsible for about 84% of the global extent of degraded land, in extreme cases leading to desertification. Because accelerated soil erosion removes the nutrient-rich upper horizons, agricultural productivity and ecological function collapse, constituting one of the most significant current environmental problems worldwide (Toy et al. 2002) and one that has plagued civilization for millennia (Montgomery 2007).

Erosion causes engineering problems both on and off site due to soil loss, as well as by the transportation and deposition of the mobilized sediment. The most deleterious effects of erosion include: removal of agricultural topsoil; beach cliff retreat and collapse; stream piracy causing near-instant changes in channel flow; scour and damage to sub-aerial and submarine engineered structures; damage to or failure of dams and levees due to internal erosion (piping); excessive sedimentation of reservoirs reducing their capacity, and of harbors and bay causing hazards to navigation; destruction of aquatic habitat due to unstable substrate, excessive turbidity, and eutrophication; sediment-related damage to roads and human structures; and sinkhole collapse. Erosion also occurs on engineered slopes and fills: erosion caused by the overtopping of embankment dams accounts for nearly 75% of US dam failures (Association of State Dam Safety Officials 2017).

Although upland and coastal erosion rates will most likely increase due to climate change, there are many prevention and remediation practices that can curtail or limit erosion of vulnerable soils. Slope erosion, channel erosion, subsurface erosion (piping), wind erosion, and coastal (cliff and beach) erosion, the most problematic from an engineering perspective, are described below.

Slope Erosion

The most common and active agent of slope erosion is running water. The kinetic energy of rain fall splash lifts particles from their inertial position up to 0.6 m vertically and 1.5 m horizontally freeing them to enter overland flow (Toy et al. 2002). If water infiltration is slow due to soil saturation or low initial moisture content, runoff water will move quickly down slope by overland flow which progresses from sheet erosion, to rill erosion, and then gulley erosion.

During sheet erosion, noncohesive soil particles add to the shear force of the flowing water, abrading the land surface and freeing even more particles. Sheet erosion involves large surface areas, but flows quickly coalesce forming small rills up to a few centimeters deep that concentrate the energy of the sediment-laden water. The rills then deepen (incise) and progress upslope, usually forming a dendritic pattern. Continuing or ensuing rainfall enlarges the rills which then merge and further incise to form gullies which initiate the headward (upslope) migration of existing channels, form new channels, or capture existing channels (stream piracy).

Factors affecting terrestrial erosion include climate, substrate structure and composition, topography, vegetation, management practices, and antecedent moisture conditions. Soil erosion is maximum during short-duration, high-intensity rainfall on dry, silty-sandy soil; long-duration rainfall wets the ground thereby increasing its permeability and the cohesion of clay particles, whereas low-intensity rains lack the kinetic energy to initiate the splash effect (Whitford 2002). Erosion continues until the underlying bedrock is exposed, after which it declines precipitously and is controlled by the rate of rock weathering.

Erosional rates, especially long term, are difficult to measure because of erosion's episodic nature, often involving long-interval/short-duration flow events. The Universal Soil Loss Equation (USLE) has been used since the 1930s (it was developed by the USDA in response to the Dust Bowl) to plan, design, and implement methods to reduce soil erosion and control sediment. Its simplest form uses six measurable factors to predict the soil loss (or yield) from a given area by sheet and rill erosion (Wischmeier and Smith 1978):

$$\mathbf{A = RKLSCP}$$

where:

A = annual soil loss or yield in tons/acre
R = rainfall erosivity factor (kinetic energy)
K = soil erodibility factor
L = topographic factors, slope length, and roughness
C, P = cropping (or vegetation) factors

The USLE has undergone numerous iterations to improve its accuracy and usability. It is now termed "RUSLE2" and is available as a free download usable on conventional computers; a user's guide is included (USDA 2016a – RUSLE2).

Although erosion is a normal and natural process, human activities have increased the rate at which soil is being eroded globally by 10–40 times (Blanco and Lal 2010). Removal of vegetation is the most important factor increasing the rates; Orem and Pelletier (2016) found that the mean erosion rates increased tenfold, following a wildfire in New Mexico. Other factors that accelerate erosion include poor agricultural practices, construction of roads and grading that disrupt natural drainage patterns, anthropogenic climate change, and urban sprawl leading to increased runoff from impermeable surfaces.

Extensive work and progress has been made to reduce slope erosion including selecting and maintaining appropriate vegetation cover, plowing parallel to slope contours, creating roughness to slow water flow, reestablishing natural drainage systems, restoring wetlands, and infiltrating runoff on-site (Gray and Sotir 1996, Blanco and Lal 2010).

Channel Erosion

Stream (including river) channels deepen, widen, and migrate upslope through a combination of erosional processes depending on the substrate, climate, vegetative cover, and hydrologic conditions. In addition to the near-continuous erosion caused by abrasion of bedload sediment that elongates and deepens (incises) channels, they also grow when temporal vortices during high flows undercut relatively hard strata that form nickpoints or hold up banks. This process occurs mainly in channels that cut through bedded sedimentary rock or layered lava flows such as in the Grand Canyon, Hawaiian Islands, and at Niagara Falls.

Bank failure also occurs in channels where silt and fine sand having high transmissivity overlie bedrock or clay. During the falling limb of the hydrograph, the pieziometric surface in the bank is higher than the water level in the channel; the resultant seepage pressure can cause subsurface erosion (piping) of the high-transmissivity bed, leading to accelerated bank failure and headward erosion (Lindow et al. 2009). Headward erosion and incision are exacerbated by human activities that lower the stream's base level, straighten the channel, remove deeply rooted vegetation, or restrict bedload sediment supply.

Catastrophic channel erosion can occur during great floods. During the last deglaciation between 15,000 and 13,000 ybp in Washington and Oregon, periodic collapse of ice dams created over a dozen mega-floods that surged down the Columbia River at up to 130 km/h (Allen et al. 2009). The floods excavated over 210 cubic km of basalt bedrock and soil while carving out the canyon of the Columbia River, and transported sediment and rock to its mouth at the Pacific Ocean. Profound scour and deposition of braid bars formed the Channeled Scablands of eastern Washington.

Bank erosion can cause spontaneous or incremental realignment of a stream's channel, such as has been occurring along the Missouri River for the last 100 years, resulting in flooding and abandonment of the original channel. Channel scour erosion is a main cause of failure of structures within or encroaching on the channel, such as bridge abutments and piers.

Reducing both urban and rural channel erosion has become a major industry during the last few decades, changing from one of a purely agricultural and hardscaping approach to bioengineering which integrates native plants and hydraulic processes to restore the channel's natural form and function (Gray and Sotir 1996).

Internal Erosion-Piping and Karst

Internal erosion, or piping, is caused by subsurface water pressures acting against sediment having high hydraulic transmissivity and low cohesion, mainly silt and fine sand, allowing the water and sediment to escape through voids or fractures.

Piping has long been recognized as a major engineering problem that ranges in degree from a maintenance nuisance to catastrophic, depending on its extent and overlying structures. Piping occurs mainly in fine-grained, noncohesive soils when upward or outward pore water pressure exceeds the static soil load. Liquefaction then occurs and the water/soil mix (slurry) exits either preexisting channels, such as fractures, or carves new ones (pipes). Piping can occur in completely natural conditions or from leaking water pipes in either native or emplaced soils. The voids caused by piping can severely weaken overlying soil, with catastrophic results.

In 2010, piping of volcanic ash underlying Guatemala City created a collapse hole approximately 20 m wide and 30 m deep that swallowed a three-story building. Piping has led to nearly 15% of US dam failures (Association of State Dam Safety Officials 2017). The worst dam failure in US history was the 1889 collapse of the South Fork (embankment) dam in Pennsylvania creating the infamous Johnstown Flood (McCulloch 1968). Internal leaks due to piping weakened the dam's structure, and then overtopping water eroded through causing its total collapse. Over 18 million cubic meters of slurry and floodwater surged down the valley killing 2,209 persons and caused $16 M in property damage. The concrete arch St. Francis dam west of Los Angeles California tragically failed during the initial reservoir filling in 1928, when piping (combined with other factors) eroded unstable rock in the dam's abutment, causing the dam to catastrophically topple (Rogers 2013). Levees are also prone to failure by piping. The collapse of levees on the Mississippi River in 1927 that killed 246 people in seven states most likely started with piping. Initiation of the piping tubes are commonly initiated by decomposition of tree roots and animal burrows.

Karst occurs when acidic groundwater dissolves soluble, mainly carbonate, rock and the overburden collapses into the void. These are commonly called "sink holes" or doline. Karst holes may be as large as 600 m in diameter and depth. Karst is highly developed in parts of Australia, Slovenia, Mexico, southeast Asia, the Caribbean, Central America, and the south eastern United States. Spontaneous karstic collapse not only damages overlying infrastructure (see numerous examples in Parise and Gunn 2007) but exposes the underlying aquifer to contamination; approximately 25% of the world's population relies entirely or in part on carbonate aquifers.

Wind Erosion

Wind is a powerful erosional force in arid and semiarid lands and in many coastal areas where it can degrade the landscape, cause excessive evaporation, damage crops and structures, cover the landscape in migratory sand, and send harmful dust and pollutants into the atmosphere which may encircle the globe (Whitford 2002). Major sources of aeolian dust include the Saharan desert of Africa, eastern Mongolia, Australia, and the southern Great Plains of the United States. As much as 4,000 tons/h of dust can fall in the Arctic during severe dust storms originating in central China. The decline in vitality of coral reefs in the Caribbean has been partly attributed to fall out of aeolian dust originating from Africa (Shinn et al. 2000).

Wind erosion occurs by three processes similar to those in water transportation. (1) In surface, creep, larger, and heavier particles are pushed or rolled along the surface. (2) During saltation (from Latin "saltare" – to dance), noncohesive, fine to medium sand and silt in the grain size range of 0.15–0.3 mm are transported as aeolian bedload from a few centimeters to 0.75 m above the ground surface where they travel a short distance then drop back down, striking others and knocking them into the airstream. (3) During suspension, wind turbulence lifts smallest and lightest particles into the air and carries them possibly for long distances. In areas where fine soil is underlain by gravel, wind erosion finally self-arrests, forming a coarse lag deposit referred to as "desert pavement."

Most (50–70%) wind erosion occurs by saltation, followed by suspension (30–40%), and then surface creep (5–25%) (Blanco and Lal 2010). In a 60 km/h wind, the uplift force exerted on a particle can be up to 500 times the particle's weight. Although small particles are most likely to become air borne, larger clasts may be picked up as well. In 1977, winds exceeding 300 km/h roared through the town of Bakersfield, California, tearing off roofs, burying cattle alive, and denuding citrus orchards. As much as 60 cm of soil from natural slopes and 35 cm of weathered granite from outcrops was removed. Pebbles as large as 9.5 cm in diameter were mobilized by the wind, whereas others up to 2.5 cm in diameter were driven into wooden telephone poles 1.6 m above the ground (USGS 1980 p. 220).

Wind erosion is most aggressive in arid areas and during times of drought, for example, during the drought of the Dust Bowl in the American Great Plains. It is estimated that soil loss due to wind erosion was as much as 6,100 times greater than during wet years (Wiggs 2011). Wind erosion and consequent land deflation were particularly damaging during when drought and poor planting practices left the fine-grained soil exposed to fierce northerly winds. Over 400,000 sq km were denuded of top soil and/or buried in sand. Airborne dust storms known as "black blizzards" dropped their loads as far away as Washington D.C. Thousands of families lost their farms and livelihoods causing 3.5 million people to vacate the area – the largest migration in American history.

Although deflation is the main problem associated with wind erosion, sand deposition is also problematic in a number of arid coastal areas such as Libya and in northern Chile where mega dunes are actively burying the city of Antofagasta. Sand abrasion occurs when wind-borne particles strike structures, removing protective paint and coatings, pitting glass and metal, denuding vegetation, and weakening wooden structures.

Soil loss due to wind erosion is estimated by the Wind Erosion Equation (WEQ), which was originally developed in (1965) based on lab tests:

$$\mathbf{E} = \mathbf{f}\,(\mathbf{IKCLV})$$

where:

E = soil loss
I = soil erodibility
K = soil roughness
C = climate
L = field length
V = vegetation

But because it consistently underestimated field measurements, the WEQ continues to be revised. The current version is downloadable as a spreadsheet calculator from the USDA website (USDA 2016b – RWEQ).

Coastal (Beach and Cliff) Erosion

Beach and sea cliff erosion occurs most aggressively during storm surges, coinciding with high tides, and during infrequent tsunami surges (Fig. 1). The mobilized sediment is carried off the beaches by currents flowing nearly parallel to the shore (longshore drift) and sometimes deposited in submarine canyons.

Beaches are eroding worldwide because they are losing sediment more rapidly than it can be replaced due to rising sea levels; coastal structures that interrupt the normal, longshore drift of sand; and the trapping of sand behind dams. It is estimated that dams have reduced the annual sand supply to coastal beaches in California by 50% (Pipkin et al. 1992). In the eastern United States, seaboard beaches extending about 1,050 km from New York through the Carolinas have been steadily eroding over the last 150 years, averaging about 0.5 m per year (USACE 2002). The shoreline of the Beaufort Sea has been retreating at a rate of 5.6 m per year since the mid-1950s (Jones et al. 2008).

Reducing beach erosion is costly. Many designs have been implemented but none prevent it. Sand replenishment, also called "beach nourishment," uses sediment that is either dredged from offshore or hauled from the back beach to replace the lost sand. Common structural approaches include groins and jetties to trap the sand or breakwaters to reduce surf impact. These and other techniques are detailed in the bible of coastal engineering: the Shore Protection Manual (USACE 2002). However, long-term efforts to protect continuously eroding beach-side real estate may be unsustainable.

Cliff erosion and consequent landsliding may be incremental or spontaneous (Fig. 1) and occur by physical impact of waves, seepage pressure (piping), abrasion by water or wind-borne sand, and by chemical dissolution. Steep cliffs composed of soft, fractured rocks are most susceptible. The most spectacular coastal rock formations such as pillars, sea cliffs, and arches are all erosional features.

Cliff erosion also damages overlying infrastructure, especially in areas of expensive "ocean view" real estate where the most desirable properties are nearest the coast. The best

Erosion, Fig. 1 Extensive cliff erosion south of San Francisco, California, following storms of January 2016. Note various and ineffective engineered methods to protect these soft sandstone cliffs from the inevitable surf assault (Drone photo courtesy of Eric Cheng (echengphoto.com))

protection against cliff erosion is to maintain the presence of thick, wide beaches of sand, pebble, or cobble that can intercept and absorb the kinetic energy of breaking surf. Costly remediation to reduce erosion has been undertaken in numerous areas of the California coast including Pacifica (south of San Francisco), Santa Barbara, Oceanside, Malibu, and Torrey Pines/La Jolla near San Diego in southern California where cliff retreat during the last 100 years has averaged 4–87 m (Pipkin et al. 1992). Common structural solutions to protect the cliffs include seawalls, rip rap, and drains (USACE 2002), but these may be prohibited or restricted by coastal conservation regulations.

Summary

Soil erosion is a global problem, not only for the loss of agricultural production but also for its impacts on engineered structures. Although slope, channel, and coastal erosion by wind and water erosion cannot be prevented entirely (nor should it), by understanding the material properties and processes involved, damaging erosion can be reduced by engineered interventions and sound land management.

Cross-References

▶ Aeolian Processes
▶ Beach Replenishment
▶ Coast Defenses
▶ Coastal Environments
▶ Cohesive Soils
▶ Dams
▶ Desert Environments
▶ Dissolution
▶ Exposure Logging
▶ Failure Criteria
▶ Floods
▶ Fluvial Environments
▶ Gabions
▶ Geotextiles
▶ Ground Preparation
▶ Infiltration
▶ Karst
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▶ Liquefaction
▶ Loess
▶ Nearshore Structures
▶ Noncohesive soils
▶ Physical Weathering
▶ Vegetation Cover
▶ Sand
▶ Sediments
▶ Shear Stress
▶ Voids

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Ethics

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Definition

In line with the concept proposed by Aristotle (384–322 BC), ethics reflects on the conduct of humans and the criteria with which to evaluate behaviors and choices in order to identify "true good" including the means to achieve this goal. It also addresses the moral duties of humans towards themselves and others, and what is the right thing to do when facing a decision. Regarding the practice of a profession, ethics is the identification of duties and rights that regulate the professional activity (deontology) by members of a social group, who are characterized by the possession of specific technical-scientific knowledge, methods and tools for its application.

There are values that the human community accepts as universally representative of individual and social good, because of the universal character of the human species itself, such as honesty, justice, responsibility, respect for life, and the environment. However, depending on the cultural context, and considering time and place, the ways in which values are applied can change.

In the end, ethics concerns all humans, without distinction, and especially those who have major scientific, political, and social roles, and who certainly have to face issues of great ethical value.

Function and Practice of Ethics

Ethics is intended to clarify, for a given circumstance, what to do and how to do it, taking into account the consequences of that act. Its function is to guide humans when they need to make a choice by providing a framework of reference values, shared by the social group to which they belong, that can lead to good or to what is most useful to the individual or society.

Moral philosophy (i.e., the reflection on how to act rightly to achieve good) attributes the ability to distinguish good from evil, and therefore to make right decisions, to some human faculties, such as reason, conscience, and knowledge, provided these faculties are adequately cultivated. Therefore, university training that includes not only technical-scientific elements, but also ethical aspects of professional practice, becomes crucial for dealing responsibly with issues specific to each disciplinary field.

Individual freedom is the fundamental pre-requisite for ethically practicing one's own profession. It allows the intentional action (will) for the pursuit of good and/or profit. So, its absence prejudices the possibility of taking ethical decisions. The analytical tool for assessing and weighing situations and possible decisions in a detailed way is critical thinking, that is, the aptitude to question a problem in the complexity of its variables, assessing interactions, uncertainties, probabilities of occurrence, but also methods, models, and tools to solve the problem itself.

An ethical problem presupposes the existence of a choice between two alternatives, one of which is the best option, taking into account the reference system of social, scientific, economic, and cultural values in which one is acting, assuring an accurate knowledge of the problem to be faced and an adequate competence for its resolution. If one option is clearly better than another, then the decision to be taken could be relatively simple. But when the two options can both produce negative consequences (ethical dilemma), the best decision (from an ethical point of view) to be taken is problematic.

Ethics in Engineering Geology

Engineering Geology is a scientific discipline included in the wider group of geosciences. The IAEG – International Association of Engineering Geology and the Environment defines the discipline Engineering Geology as "*the science devoted to the investigation, study and solution of the engineering and environmental problems which may arise as the result of the interaction between geology and the works and activities of man as well as to the prediction and of the development of measures for prevention or remediation of geological hazards*" (IAEG Statutes – Article 2: http://iaeg.info/media/1008/iaeg-statutes.pdf, accessed 18 October 2017). One of the main goals of engineering geology is to assure safety, health, and welfare of the public (Cronin 1993).

The definition provided by IAEG encompasses both the micro-ethical domain of individual activities and the macro-ethical domain of engineering geology's relationship with society and the environment, providing the expertise needed to address local and global issues originated by the interaction between mankind and the Earth system (Bobrowsky et al. 2017).

Micro-ethics considers ethical choices and dilemmas faced by individual researchers/practitioners, especially as they relate to acting in accordance with professional codes and norms; macro-ethics deals with ethical issues of research and practice in larger social and institutional contexts, including broader social responsibilities of engineers and geologists, policy and political questions and debates, questions about what the rules and norms should be, and who is involved in debates (Vallero 2014).

In the problems of Engineering Geology often the decision can depend on the scale with which one is working (for example, a small village or a large inhabited center), the presence of infrastructures and/or economic activities, the possibilities of development and the vocation of the territory affected by the intervention. The more functional choice from a safety and economic point of view will include not only technical aspects but also aspects of land policy. Through a comprehensive analysis of the situation, it will be possible to propose to decision-makers the optimal solution to achieve the purpose.

An engineering geology intervention on the territory (such as the construction of an infrastructure) might seem ethically neutral. But since the construction of the infrastructure has a socioeconomic and environmental impact, the technical-scientific choices associated with the decision to carry out the work, the choice of the best typology and the modes of implementation also become subjects of ethical reflection. The greater the importance of the work, the greater the responsibilities associated with it, and the greater the need to adopt an ethical behavior.

Geoethics: Definition, Method and Values

In geosciences, the most specific "Geoethics" term now in use (Bobrowsky et al. 2017; Peppoloni and Di Capua 2015; Wyss and Peppoloni 2015) indicates the "*research and reflection on the values that underpin appropriate behaviours and practices, wherever human activities interact with the Earth system. Geoethics deals with the ethical, social and cultural implications of geoscience knowledge, education, research, practice, and communication, and with the social role and responsibility of geoscientists in conducting their activities.*" (Website of the IAPG – International Association for Promoting Geoethics: http://www.geoethics.org, accessed 18 October 2017).

The definition of geoethics includes aspects of general ethics, research integrity, professional ethics, environmental ethics, recalling the geoscientist to an individual conduct with awareness of being a social actor possessing scientific knowledge that can be put to the service of society and employed for a more functional interaction between man and the Earth system.

Geosciences have clear ethical and social implications (Peppoloni et al. 2017). On the basis of the abovementioned definitions, it is evident that Engineering Geology is a paradigmatic field of application of geoethics (Lollino et al. 2014).

Geoethics focuses on responsibility as the ethical criterion of geoscientists in decision-making, whatever their sector of work (academic, professional, research, teaching, industry, government, etc.). Taking the responsibility of a professional choice means to act rationally with respect to the purpose that one aims (Weber 1919), but also considering the impact of one's choices on future generations (Jonas 1979), assessing the trinomial "situation/decision/consequences" and submitting this to the technical and scientific judgment of colleagues and society. Therefore, responsibility means we answer for our actions, owing to our competence on the problem that need to be addressed and solved.

Certainly our decisions should consider scientific and technical aspects, as well as economic and temporal implications (for example lesser costs or shorter feasibility time). But at the same time, we should take into account the greater social benefit our choice can entail (for example, the protection of lives of citizens or the improvement of the economic activities of a certain area). Finally, we will take care of environmental aspects, by choosing interventions that respect as much as possible natural dynamics. The result of this analysis must lead to that point of equilibrium that optimizes the sum of the effects and allows us to take the most ethically sustainable decision for the human community and the environment that are involved.

Since Engineering Geology is essentially an applied science, it is based on experience, on continuous application of knowledge to physical reality, and on empirical testing of models, with the aim of identifying best practices. Following the scientific method, this discipline will analyze the problem, define elements that characterize it, apply a model that simulates it, verify and modify model variables, in order to refine the model itself and advance the knowledge. Likewise, the reference values of geoethics that accompany the practice of Engineering Geology must be constantly defined and verified in the light of the concreteness of the obtained results, of the cultural, social, and economic context in which one is operating and the temporal perspective of our solutions, so as to avoid being confined to a purely theoretical question.

Finally, a (geo)ethical approach to the profession also has great civil value, since it is an indispensable requirement for establishing a climate of trust between geoscientists, decision-makers, and citizens. If engineering geologists, as responsible scientists in the public interest, fail to act ethically, they will lose the trust of society. At the same time, following an ethical approach to the profession also means to mediate, on the basis of its own scientific expertise, among different positions and often to face pressures from politics, public opinion, economic powers, and industrial interests.

Individual and Social Dimensions of Geoethics

Engineering Geology can consist of small-scale interventions, such as the construction of a retaining wall or foundations of a new building, slope securement, or actions of great impact on the Earth system, such as the construction of a large dam or a tunnel, the exploitation of a mine, the modification of the groundwater regime, or the deployment of a nuclear waste disposal plant. Regardless of the scale, the intervention may have repercussions on ecosystems and local populations. For this reason, technical decisions should be accompanied by ethical considerations that take into account the social and environmental impact of that work.

But alongside this social dimension, there are also ethics intimately linked to the individual dimension of the geoscientist, including responsibility towards himself/herself, the dignity of their own work, the respect for colleagues and end users of that work, the value of one's own knowledge and skills, and the recognition of skills and positions of others.

The self, colleagues, society, and the Earth system are the four fundamental dimensions for geoethical analysis (Teaching GeoEthics Across the Geoscience Curriculum website: https://serc.carleton.edu/geoethics/index.html, accessed 18 October 2017; Bobrowsky et al. 2017). They include responsibility towards:

- Themselves as geoscientists, in doing the best work they can on the basis of their abilities and competences
- Colleagues, to cooperate with respectful attitude, in order to achieve the common goal to find solutions to geoscience problems by following a multidisciplinary approach
- Society, that geoscientists have the duty to serve for the very fact of possessing a scientific knowledge that has social repercussions
- The Earth system, that geoscientists have to manage, by protecting as much as possible its geodiversity and biodiversity and its aesthetic dimension.

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Deontology, Research Integrity and Codes of Ethics/Conduct

The IUGS – Task Group on Global Geoscience Professionalism (TGGGP) clearly states that "*Geoscientists in all areas of the geoscience profession are called upon to provide expert services and opinions. These services and opinions are relied upon by employers and the public to make key decisions; decisions which affect business, the general public good, and the environment. It is essential that those geoscientists providing the services and opinions are providing them at a professional level; incorporating sound geoscience knowledge and application of theory; exceptional ethics; and good judgment; providing services and opinions only in the areas of geoscience in which they are competent*" (Website of TGGGP: https://tg-ggp.org/, accessed 18 October 2017).

Engineering geologists work both as researchers and professionals after extensive academic training to acquire the special knowledge and skills to do their jobs effectively on behalf of society. In many countries, engineering geologists working as professionals must be licensed or certified.

Researchers have the duty to follow the scientific methods and rules of research integrity, intended as the set of ethical values, deontological duties and professional standards on which is based the responsible and correct conduct of those who carry out, fund or evaluate scientific research, as well as of institutions that promote and implement it. The Singapore and Montreal Statements on Research Integrity (Website of the World Conferences on Research Integrity: http://www.researchintegrity.org/, accessed 18 October 2017) are the two main reference documents to be considered in order to maintain high levels of one's own ethical profile while doing scientific activities.

Professionals, working as free-lance consultants or employees within engineering companies, mining, or industries, have to follow laws, codes, professional regulations, standards, and public policies by political/administrative authorities.

In order to ensure an ethical dimension in their job, in some countries geoscientists are called to adopt codes of ethics/conduct by their professional or scientific associations (see the list at http://www.geoethics.org/codes, accessed 18 October 2017), framing a set of general principles and listing best-practices (Andrews 2014). The observance of codes of ethics is generally mandatory: if such observance is disregarded, disciplinary measures may be taken by judging bodies.

Codes of conduct are certainly useful in establishing the ethical coordinates for a group of geoscientists. However, the observance of an ethical conduct is not obtained by the mere adoption of codes and rules but needs a sincere desire for individual adherence and a path of personal growth that must be encouraged from the first stages of academic education. In fact, ethics of responsibility must not be confused with ethics embodied by the code itself, as a tool that establishes principles and rules. The code of ethics can only certify an ethical decision, which may only arise from a responsible choice. The deep meaning of ethics for a professional is to be perfect and honest in his/her work, in relationships with colleagues and end users, so that his/her business also becomes an opportunity for personal inner growth.

Conclusion

In ethical issues, before the social dimension, one is always involved individually, and it is always necessary to question one self, one's own actions and decisions with objectivity and discipline, to understand the best way to act. Ethical behavior is a practice to be applied daily, in order to be honest and qualified in one's own work: this implies competence and awareness of the limits of personal knowledge and the uncertainties of models.

Ethics applied to Engineering Geology is primarily a scientific question that needs an accurate evaluation of variables at stake, aimed at a cost-benefit analysis that also includes social and environmental aspects, by taking into account the spatial and temporal impact of decisions to be taken. In any case, the cost-benefit assessment alone may not represent the solving criterion of a problem. Often, in the decision-making process that accompanies an Engineering Geology intervention on the territory, ethical, and social elements, that are beyond purely technical-scientific issues, also converge. The final decision on the feasibility and implementation of the intervention depends not only on science or technology but also on politics. However, science must not replace politics, but instead provide all the concrete and exhaustive elements to take a decision as sustainable as possible for that social and environmental system.

In this context, a modern and responsible engineering geologist is called to ensure:

- Excellence and continuous updating in technical-scientific preparation
- Rigorous application of the scientific method, providing models based on verified data, which takes into account uncertainties and probabilities
- Knowledge of the ethical and social implications of the profession
- Ability to listen to societal needs with the aim for public safety
- Promptness in identifying suitable technical solutions
- Compliance with laws and procedures
- Ability to relate and be mediator within multidisciplinary teams
- Ability to communicate data and results of public and or private interest

For all the above considerations, the current academic training of engineering geologists needs integrations aimed to increase the individual awareness of the ethical implications of the profession and the resulting responsibilities.

Cross-References

► Accreditation
► Engineering Geology
► Environmental Assessment
► Factor of Safety
► Geology
► Hazard Assessment
► Land Use
► Risk Assessment

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Evaporites

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Definition

An evaporite is a salt rock (in its broadest sense) originally precipitated from a saturated surface or near-surface brine in hydrological conditions driven by solar evaporation (Warren 2016). Most develop in hot arid coastal environments on evaporitic mudflats (sabkhas, e.g., in the United Arab Emirates) in shallow evaporitic seas or in inland salt lakes (salterns or salinas). Their ancient depositional environments were similar to the modern ones, but ancient deposits are considerably altered by complex diagenesis.

Seawater evaporation follows a depositional sequence becoming more saline with more exotic salts formed as the dissolved salt concentration increases. The first deposited minerals are alkaline Earth carbonates, followed by evaporites, gypsum/anhydrite, halite, sylvite, polyhalite, and finally complex evaporite minerals (Warren 2016). In enclosed continental evaporitic basins, gypsum and halite are common as are exotic salts; in very cold continental settings, cryogenic evaporites can also form (e.g., mirabilite, Table 1) when the brine is concentrated by ice formation.

Gypsum, anhydrite, and halite are commonly encountered in engineering geology both at the surface in hot environments and at the subsurface worldwide. Gypsum dehydrates to anhydrite on burial below depths of about 400–1000 m dependent on local geothermal gradient and adjacent lithologies. The dehydration involves a 39% volume reduction; on exhumation and rehydration, anhydrite swells by up to 63% (Zanbak and Arthur 1986) making it a geological hazard to tunneling, boreholes, and open-loop ground source heat pumps. Gypsum is highly soluble, and a gypsum rock face may dissolve at up to 1.5 m a year in a moderate river flow (1 ms^{-1}), less quickly in the subsurface if there is dissolved sulfate. Evaporites commonly produce a karst landscape with active sinkhole hazards (Gutiérrez et al. 2008), and gypsum is notable for hosting extensive maze cave systems.

Evaporites, Table 1 Common evaporite minerals

Common evaporite minerals	Common names	Chemical formula	Comments
Evaporitic alkaline Earth carbonates	Aragonite/calcite, high and low Mg calcite, dolomite magnesite	$CaCO_3$ through $MgCaCO_3$ to $MgCO_3$	The first to precipitate in a hypersaline evaporitic environment
Gypsum	Alabaster, fibrous gypsum, satin spar	$CaSO_4{\cdot}2H_2O$	Dehydrates to anhydrite and contracts
Anhydrite		$CaSO_4$	Hydrates to gypsum and expands
Halite	Common salt	NaCl	
Sylvite	Potash	KCl	
Polyhalite		$2CaSO_4{\cdot}MgSO_4{\cdot}K_2SO_4{\cdot}H_2O$	
Mirabilite		$NaSO_4{\cdot}10H_2O$	Common in cryogenic evaporites

Salt is highly soluble with a very high freshwater dissolution rate of up to 0.2 mms^{-1} (c.17 m a day) in flood conditions (0.5–3 ms^{-1}) (Frumkin and Ford 1995). Natural and anthropogenic salt dissolution can produce highly unstable land (see ► "Voids"). Because of its high dissolution rate, salt may be lost from borehole cores drilled for site investigation; if its presence is suspected, brine drilling fluid should be used. Salt is also highly mobile and can flow in unconfined conditions. Salt, gypsum, and anhydrite are highly problematic in hydraulic structures and should be avoided. Salt and gypsum are also prone to creep under loading (Bell 1987). Gypsum and anhydrite are associated with sulfate-rich groundwater and halite with brine; both can be problematic for engineering as they adversely affect concrete and can cause heave.

Cross-References

► Aeolian Processes
► Alteration
► Capillarity
► Characterization of Soils
► Clay
► Climate Change
► Desert Environments
► Desiccation
► Dewatering
► Dissolution
► Evaporites

E

▶ Expansive Soils
▶ Filtration
▶ Fluid Withdrawal
▶ Geohazards
▶ Groundwater
▶ Hydraulic Action
▶ Infiltration
▶ Limestone
▶ Noncohesive Soils
▶ Physical Weathering
▶ Quick Clay
▶ Risk Assessment
▶ Sabkha
▶ Saline Soils
▶ Sand
▶ Silt
▶ Sinkholes
▶ Soil Mechanics
▶ Subsidence
▶ Subsurface Exploration
▶ Tropical Environments
▶ Vegetation Cover
▶ Voids
▶ Water

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Excavation

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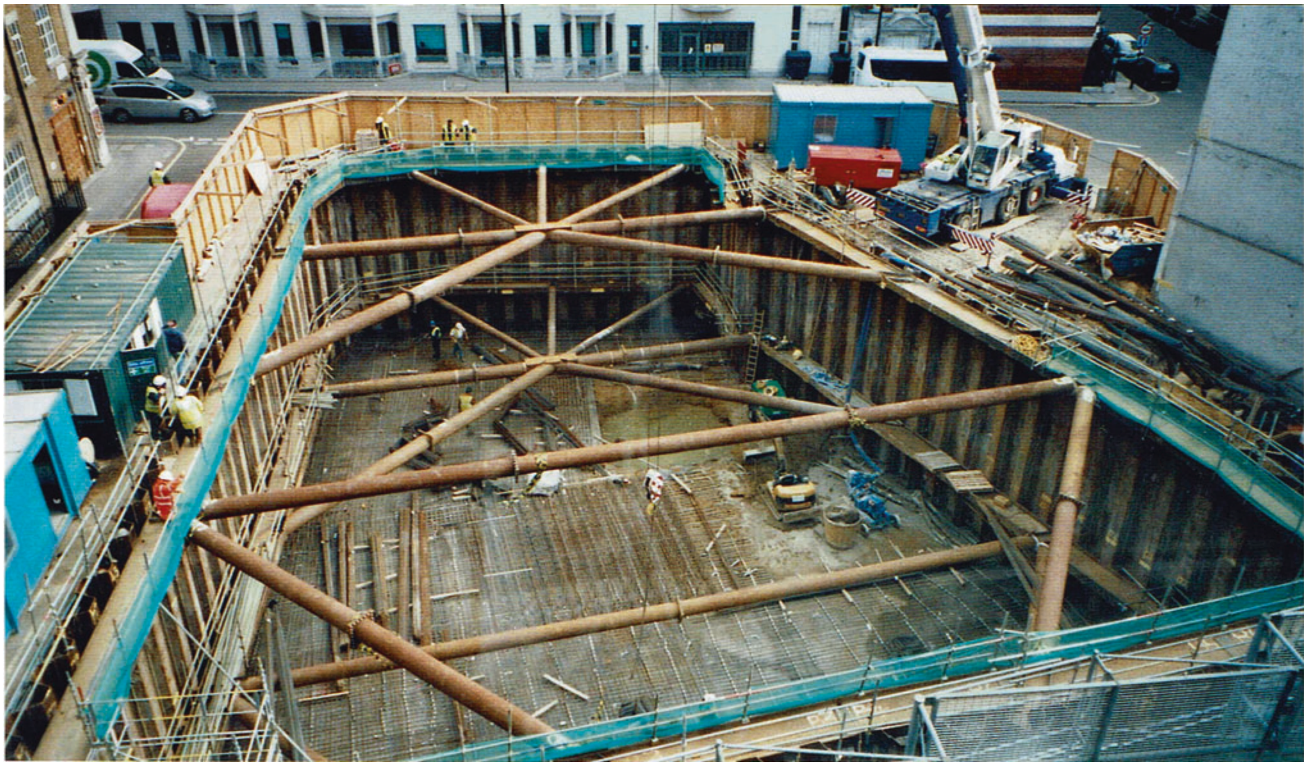
Definition

1. A general term used to describe a process of loosening in-place Earth material and moving it to create a subsurface hole or room for some purpose or to generate Earth material for use at a nearby location as compacted fill or erosion protection (e.g., riprap).
2. The subsurface hole or room created by the process (i.e., an excavation).

For contracting purposes, excavation tends to be designated as common, rock, or unclassified (NRCS 2014). Common excavation refers to Earth material that can be excavated, transported, and unloaded with conventional equipment that includes heavy ripping, self-loading wheeled scrapers, and scrapers assisted by tractors that provide supplemental power by pushing. Common excavation also includes material that can be dumped into a designated place or loaded onto hauling equipment by excavators having a capacity of approximately 3/4 m^3 or larger using attachments appropriate for the material and site conditions (e.g., shovel, bucket, dragline, or clam shell). Rock excavation refers to material that requires blasting or the use of equipment larger than that designated for common excavation. Unclassified excavation refers to material that needs to be excavated without regard to the equipment or methods. Special equipment and handling are required for underwater dredging excavation of uncemented sediments and muck (USACE 2015).

Guidance for surface excavations that require blasting is provided by USACE (1972), whereas guidance for underground excavations in tunnels and shafts is provided by USACE (1997). Many rock excavations for civil engineering purposes need to meet specifications for the positions of the final slopes. These types of projects typically have a penalty for over excavation beyond the final cut line, in part because it will require concrete that would not have been needed if the cut line had been attained. Rock excavations in mining and quarrying need to attain a maximum rock fragment size, in part because secondary blasting might be needed to attain the maximum size for processing with a crusher or simply hauling from the excavation site.

Special-purpose excavations are used during geotechnical investigations for obtaining samples for laboratory testing and engineering classification based on the spoil pile; these excavations are observed from the ground surface without the need for the geologist or engineer to enter. Other excavations may be made to expose geologic relationships for detailed logging (e.g., recency of faulting studies), in which case the excavation is braced with temporary shoring to enable the geologist to enter safely. Temporary excavations are backfilled after observation and sampling, or after placement of steel and concrete, as in the case of a foundation excavation. Some temporary excavations are needed in soft, saturated sediment that tend to flow when disturbed. Excavations in these materials are stabilized by the use of sheet piles and tie-back anchors to keep the soils adjacent to the excavation in place without undue deformations. Sheet piles also are used in sandy soils that may tend to “run” to create a slope at the angle



Excavation, Fig. 1 An excavation for underground parking and services, prior to construction of a high-rise building in London UK – note steel soldier piles and timber lagging around the periphery and bracing to prevent side collapse (Photo by Dr B R Marker)

of repose; the sheet piles in this application confine the sandy soils which keeps the width of the excavation from becoming very wide, which can encroach on the allowable limits of construction and create problems for equipment and personnel that need to access the excavation.

Some deep excavations are needed in densely urbanized areas (Fig. 1), such as for constructing underground parking or a subway station in a city center. In these cases, the stability of the excavated slope is stabilized with walls that may be constructed of soldier piles and heavy timber or concrete lagging elements. Other excavations might be braced to limit lateral deflection of the ground at the crest of the excavation. Careful monitoring is performed during excavation and construction of the station walls to ensure that deflections are minor and do not cause damage to adjacent buildings or roads.

Deep excavations in some settings may encounter groundwater, which might make excavation more expensive and possibly cause side-wall stability problems or heave of the bottom of the excavation. Construction dewatering using well points and pumping is a common method of mitigating the problems associated with shallow groundwater. However, construction dewatering causes ground surface settlement or subsidence that extends away from the dewatered excavation as much as tens of meters, if not 100 m. The subsidence needs to be predicted and monitored, or alternative construction methods may be required if the settlement may damage adjacent buildings.

Cross-References

▶ Blasting
▶ Building Stone
▶ Classification of Rocks
▶ Classification of Soils
▶ Cut and Cover
▶ Cut and Fill
▶ Designing Site Investigations
▶ Dewatering
▶ Drilling
▶ Erosion
▶ Factor of safety
▶ Failure Criteria
▶ Field Testing
▶ Geophysical Methods
▶ Geotechnical Engineering
▶ Ground Preparation
▶ Groundwater
▶ Inclinometer
▶ Instrumentation
▶ Mining

- ► Monitoring
- ► Retaining Structures
- ► Rock Bolts
- ► Rock Mass Classification
- ► Rock Mechanics
- ► Shear Strength
- ► Shotcrete
- ► Site Investigation
- ► Soil Mechanics
- ► Soil Nails
- ► Stabilization
- ► Subsurface Exploration
- ► Tension Cracks
- ► Vibrations

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Expansive Soils

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Definition

Expansive Soils are soils that have the ability to shrink and/or swell, and thus change in volume, in relation to changes in their moisture content. They usually contain some form of expansive clay mineral, such as smectite or vermiculite, that are able to absorb water and swell, increasing in volume, when they get wet and shrink when they dry. The more water they absorb, the more their volume increases. For the most expansive soils volume changes of 10% are common (Chen 1988; Nelson and Miller 1992).

Introduction

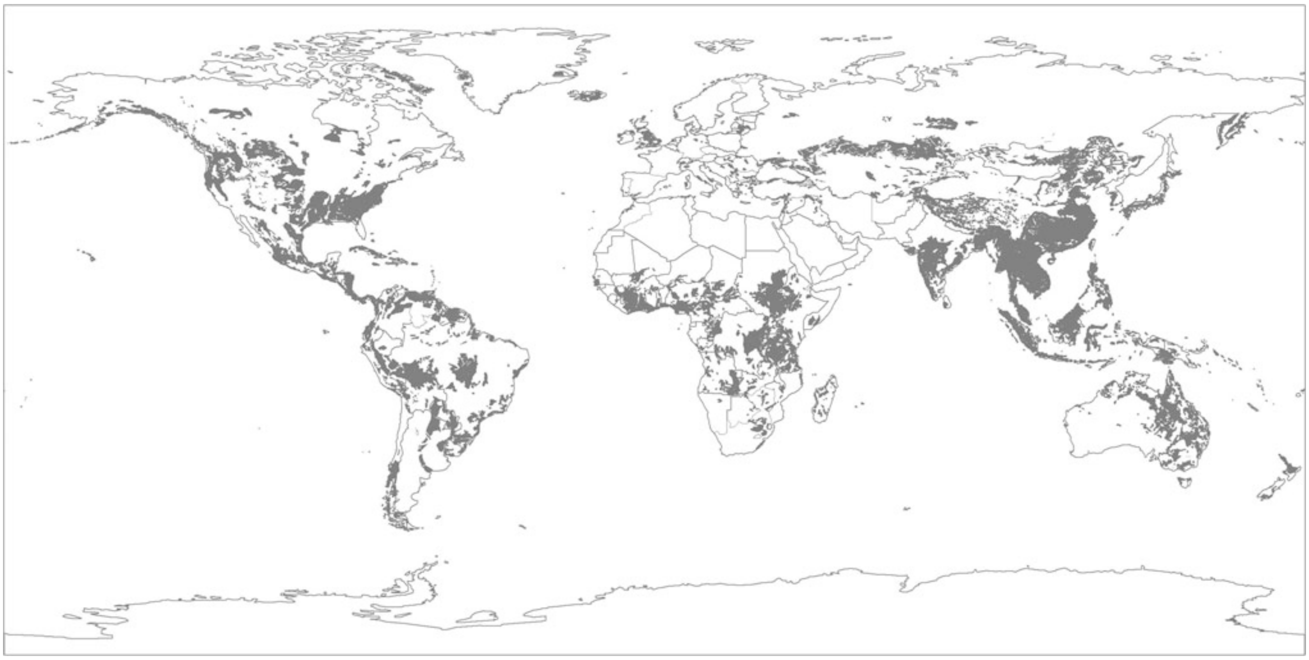
Many of the world's largest towns and cities, and therefore their arterial transport routes, services, and buildings, are founded on clay-rich soils and rocks. These expansive soils can prove to be a substantial hazard to engineering construction due to their ability to shrink or swell with seasonal changes in moisture content, local site changes such as leakage from water supply pipes or drains, changes to surface drainage and landscaping or following the planting, removal, or severe pruning of trees or hedges. Houses and other low-rise buildings, pavements, pylons, pipelines, and other shallow services are especially vulnerable to damage because they are less able to suppress differential movements than heavier multi-story structures. Pavements are also highly susceptible to damage because of their relative light-weight nature extended over a relatively large area.

The amount by which the ground can shrink or swell is determined by the water content in the near-surface (active) zone; significant activity usually occurs to about 3 m depth, unless this zone is extended by the presence of tree roots (Driscoll 1983; Biddle 1998, 2001). During rainfall these soils can absorb large quantities of water becoming sticky and heavy and causing heave, or lifting, of structures, and during prolonged periods of drought they can become very hard, causing shrinkage of the ground and differential settlement. This hardening and softening is known as "shrink–swell" behavior and presents a significant geotechnical and structural challenge to anyone wishing to build on, or in, them. The main factors controlling this behavior are the clay content and mineralogy, the *in-situ* effective stresses, and the stiffness of the material. Aspects such as original geological environment, climate, topography, land-use, and weathering affect these factors, and hence shrink–swell susceptibility.

Where Are They Found?

Expansive soils are found throughout many regions of the world, particularly in arid and semiarid regions, as well as where wet conditions occur after prolonged periods of drought. Their distribution is dependent on geology, climate, hydrology, geomorphology, and vegetation. Countries where expansive soils occur and give rise to major construction costs include Ethiopia, Ghana, Kenya, Morocco, South Africa, and Zimbabwe in Africa; Burma, China, India, Iran, Israel, Japan, and Oman in Asia; Argentina, Canada, Cuba, Mexico, Trinidad, the USA, and Venezuela in the Americas; Cyprus, Germany, Greece, Norway, Romania, Spain, Sweden, Turkey, and UK in Europe; and Australia (Fig. 1).

In large areas of these countries, the evaporation rate is higher than the annual rainfall so there is usually a moisture



Expansive Soils, Fig. 1 Global distribution of shrink–swell soil where major construction costs occur (by region)

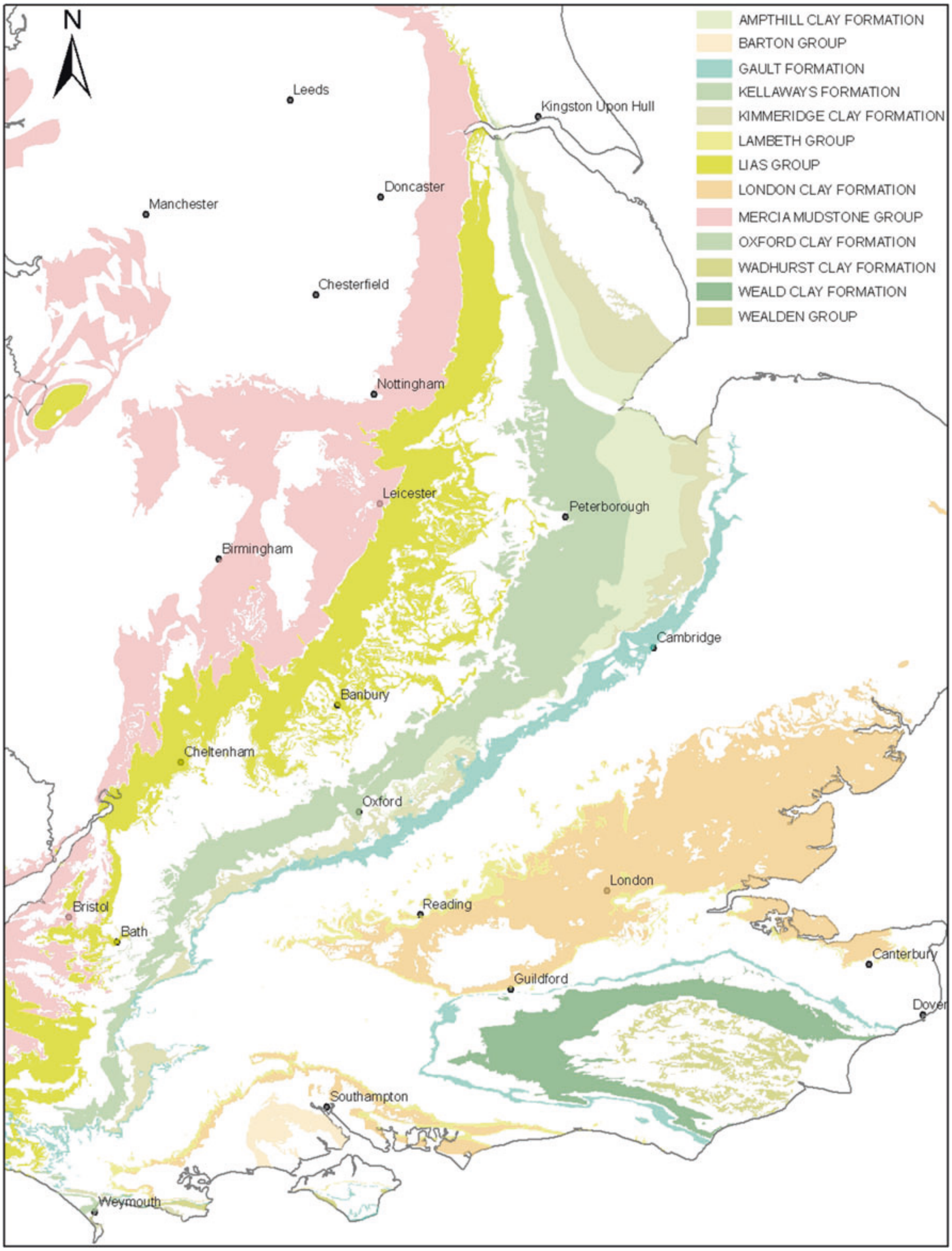
deficiency in the soil. When it rains, the ground swells and increases the potential for heave. In semiarid regions, a pattern of short periods of rainfall followed by periods of drought can develop, resulting in seasonal cycles of swelling and shrinkage; in humid climates, problems with expansive soils trend to be limited to those containing higher plasticity clays; and in arid climates, even moderately plastic soils can cause damage to residential property. The literature is full of studies, from all over the world, concerned with problems associated with expansive clays (Fredlund and Rahardjo 1993; Stavridakis 2006; Hyndman and Hyndman 2009).

In the UK, towns and cities built on clay-rich soils most susceptible to shrink–swell behavior are found mainly in the south-east of the country, south of a line from Dorset to the North Yorkshire coast (Fig. 2). Here many of the “clay” formations are too young (Jurassic or younger) to have been changed into stronger “mudstones,” leaving them still able to absorb and lose moisture. These deposits are normally firm to very stiff clay or very weak mudstones that weather to firm to stiff clay near the surface. Clay rocks elsewhere in the country are older and have been hardened by processes resulting from deep burial; they are less prone to shrink–swell behavior because they contain less active clay minerals and are less able to absorb water. Some areas (e.g., around The Wash, northwest of Peterborough, and under the Lancashire Plain) are deeply buried beneath other (surficial) soils that are not susceptible to shrink–swell behavior. However, other surficial deposits such as alluvium, peat, and laminated clay can also be susceptible to soil subsidence and heave (e.g., in the Vale of York, east of Leeds, and in the Cheshire Basin). In the UK, some Mesozoic and Tertiary clay soils and weak mudrocks are also susceptible to shrinkage and swelling as environmental conditions change (Harrison et al. 2012) (Based on section 3 of Jones and Jefferson 2012)

Whereas the distribution of UK clay soils is relatively well known in 2-D, for example, Loveland (1984), Jeans (Jeans 2006a, b), and Wilson et al. (1984), the 3-D distribution is less well known. A meaningful assessment of the shrink–swell potential of any soil requires a considerable amount of high-quality and well-distributed spatial data of a consistent standard (Jones and Jefferson 2012) and from this a Volume Change Potential (VCP) map can be constructed. However, looking at soils on a national scale (although giving a good indication of potential problem areas) does not tell the whole story; therefore it is better to look at them on a more regional scale. Jones and Terrington (2011) discuss a methodology for creating a 3D VCP interpolation of the London Clay, visualizing plasticity values at a variety of depths, relative to ground level, across the outcrop (Fig. 3).

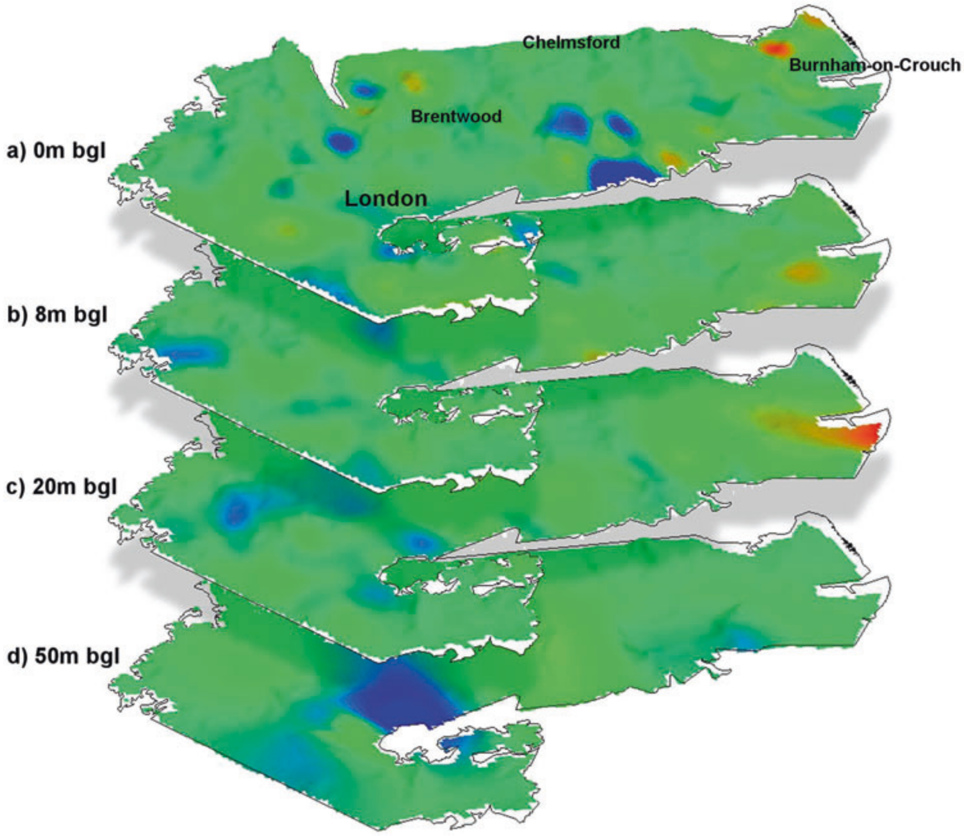
What Is the Damage?

Expansive soils were first acknowledged, in the UK, as a major cause of foundation damage following the drought of 1947, since then insurance claims have dramatically increased. In 1991, claims peaked at over £500 million, and over the past 20 years, the Association of British Insurers has



Expansive Soils, Fig. 2 Distribution of UK clay-rich soil formations (After Jones and Jefferson 2012)

estimated that damage caused by expansive soils has cost the insurance industry over £400 million a year (Driscoll and Crilly 2000), making it the most damaging geological hazard in the UK. In fact, one in five homes in England and Wales are at risk from ground that swells when it gets wet and shrinks as it dries out (Jones 2004), although susceptible ground conditions are perhaps less severe under a temperate UK climate than in some other countries. The American Society of Civil



Expansive Soils, Fig. 3 Interpolation of "Area 3" showing surfaces at 0 m, 8 m, 20 m, and 50 m below ground level. Note: blue: medium, green: high, red: very high (Jones and Terrington 2011)

Engineers has estimated that as many as one in four homes in the continental United States has some damage caused by expansive soils, with the annual cost of damage to buildings and infrastructure exceeding $15 billion. In a typical year they cause a greater financial loss to property owners than earthquakes, floods, hurricanes, and tornadoes combined (Nelson and Miller 1992).

Expansive soils can cause heaving of structures when they swell and differential settlement when they shrink. Damage to a structure is possible when as little as 3% volume expansion takes place (Jones 2002), especially where these changes are distributed unevenly beneath the foundations. If the water content of a clay soil around the edge of a building changes, the swelling pressure will also change, whereas the water content of the soil beneath the centre of the building remains constant, causing a failure known as end lift (Fig. 4). Where the swelling is concentrated beneath the centre of the structure (or where shrinkage takes place under the edges) a failure known as centre lift takes place.

Another major contributing factor to ground shrinkage is tree growth, more specifically tree roots. Roots will grow in the direction of least resistance and where they have the best access to water, air, and nutrients (Roberts 1976). The actual pattern of root growth depends upon the type of tree, depth to water table, and local ground conditions. Damage to foundations resulting from tree growth occurs in two principal ways:

- Physical disturbance of the ground – caused by root growth and often seen as damage to pavements and walls
- Shrinkage of the ground – caused by water removal and often leading to differential settlement of building foundations

Vegetation-induced changes to water profiles can also have a significant impact on other underground features, including utilities. Tree-induced movement has the potential to be a significant contributor to failure of old pipes located in clay soils near deciduous trees (Clayton et al. 2010).

Building, or paving, on previously open areas of land, such as the building of patios and driveways, can cause major disruption to the soil-water system. Sealing the ground in this way cuts off the infiltration of rain water and the trees that are dependent upon this water will have to send their roots deeper, or farther afield, in order to find water. The movement of these root systems will cause a major ground disturbance and will lead to the removal of water from a larger area around the tree (Jones and Jefferson 2012). Problems occur when structures are situated within the zone of influence of a tree

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Expansive Soils, Fig. 4 Structural damage to house caused by "end lift" (© Peter Kelsey & Partners)

(Fig. 5). Pavements are also highly susceptible to damage because of their relative light-weight nature extended over a relatively large area.

Shrink–Swell Behavior

The shrink–swell potential of expansive soils is determined by its initial water content; void ratio; internal structure and vertical stresses; as well as the type and amount of clay minerals in the soil (Bell and Culshaw 2001). These minerals determine the natural expansiveness of the soil, and include smectite, montmorillonite, nontronite, vermiculite, illite, and chlorite. Generally, the larger the amounts of these minerals present in the soil, the greater the expansive potential.

Clay particles are very small and their shape is determined by the arrangement of the thin crystal lattice layers that they form. Taylor and Cripps (1984), Taylor and Smith (1986), and Driscoll (1983) provide useful reviews of the controls that clay mineralogy has on the drained compressibility/expansibility of geological materials and hence their susceptibility to large deformations from effective stress changes which lead to shrinkage and/or swelling. In expansive clay, the molecular structure and arrangement of these crystal layers has an affinity to attract and hold water molecules between them (and on their surfaces) in a strongly bonded "sandwich," giving them a large shrink–swell potential. For further details of the mineralogy of clay minerals and their influence of engineering properties of soils see Mitchell and Soga (2005).

Potentially expansive soils are initially identified by undertaking particle size analyses to determine the percentage of fine particles in a sample. Clay sized particles are considered to be less than 2 μm (although this value varies slightly throughout the world) but the difference between clay and silt is more to do with origin and particle shape. Silt particles (generally comprising quartz particles) are products of mechanical erosion whereas clay particles are products of chemical weathering and are characterized by their sheet structure and composition.

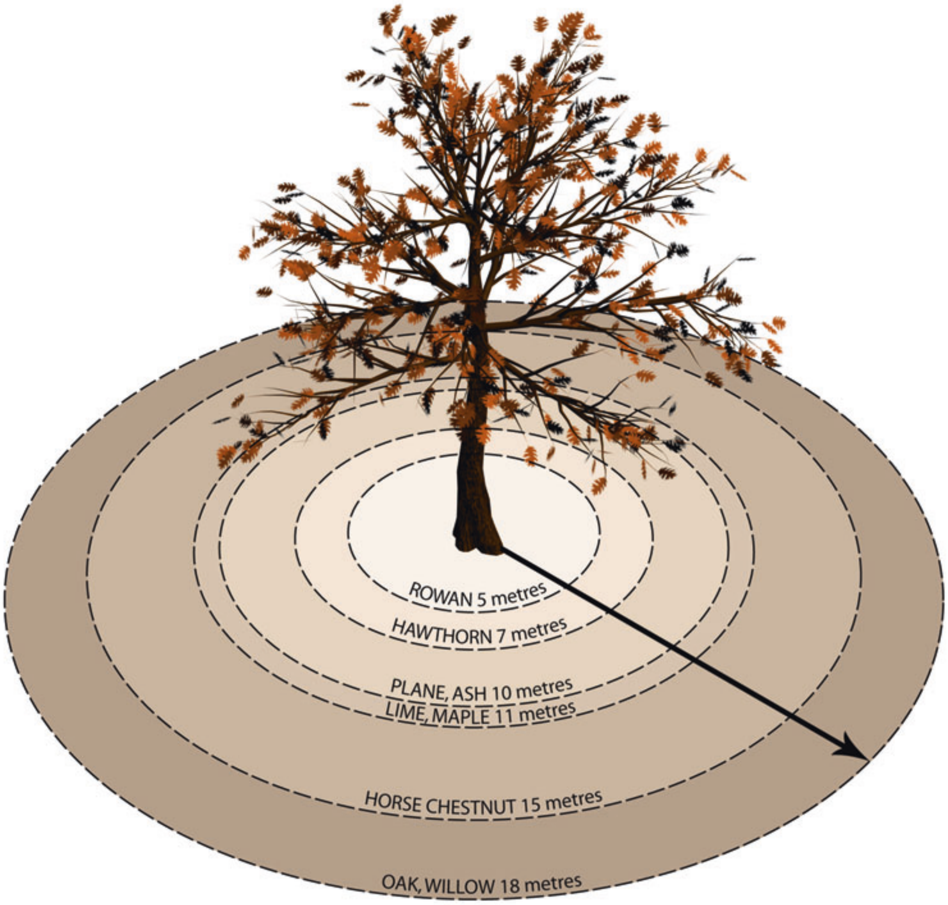
Soils with high shrink–swell potential will not usually cause problems as long as their water content remains relatively constant. This is controlled by the soil properties (mineralogy); suction and water conditions; water content variations; and geometry and stiffness of a structure founded on it (Houston et al. 2011). In a partially saturated soil, suction or water content changes increase the likelihood of damage occurring. In a fully saturated soil, the shrink–swell behavior is controlled by the clay mineralogy.

Expansive Soils in Construction

Potential shrinkage and/or swelling from these causes can usually be anticipated in most engineering circumstances. However, because of the differences between natural and tree-induced shrink–swell, and varying initial conditions, the relative susceptibility to volume change at any place may not necessarily always be the same for a given geological formation or soil type. Houses and other low-rise buildings, pavements, pylons, pipelines, and other shallow services are especially vulnerable to damage from shrink–swell clays because they are less able to suppress differential movements than heavier multistory structures.

Due to the global distribution of shrink–swell soils, many different ways to tackle the problem have been developed and these can vary considerably (Radevsky 2001). These methods depend not only on technical developments but the legal framework and regulations of a country, insurance policies, and the attitude of insurers, experience of the engineers, and other specialists dealing with the problem and, most importantly, the sensitivity of the owner of the property affected. A summary of these issues is provided by Radevsky (2001) in his review of how different countries deal with shrink–swell soil problems, and a detailed informative study from the United States has more recently been presented by Houston et al. (2011).

Shrink–swell soils require extensive site investigation in order to provide sufficient information. Normal investigations, relating to the structures most affected by shrink–swell soils, are often not adequate. These investigations may involve specialist test programs even for relatively light weight structures (Nelson and Miller 1992). Although there are a number of methods available to identify shrink–swell soils, each with their relative merits, there are no universally reliable methods available (Jones and Jefferson 2012), and they are rarely employed in the course of routine site investigations in the UK. This means that few data are available for data-basing the directly measured shrink–swell properties of the major clay



Expansive Soils, Fig. 5 The zone of influence of some common UK trees (Jones et al. 2006)

formations, and reliance has to be placed on estimates based on index parameters, such as liquid limit, plasticity index, and density (Reeve et al. 1980; Holtz and Kovacs 1981; Oloo et al. 1987). No consideration has been given to the saturation state of the soil and therefore to the effective stress or pore pressures within it. For further details on the strategies for dealing with the engineering issues and management of expansive soils see Jones and Jefferson (2012).

Summary

Expansive soils are found throughout many regions of the world and the subsidence and heave problems associated with them causes billions of pounds of damage annually, making them one of the most costly and widespread geological hazards to domestic properties and other low-rise structures. In arid/semiarid regions, their ability to take up large quantities of water can cause major damage to structures, whereas in more humid regions, such as the UK, problems mainly occur in the more highly plastic soils, especially after prolonged periods of drought. Either way, expansive soils have the potential to demonstrate significant volume change in direct response to changes in water content, induced through water ingress, modification to local water conditions, or via the action of external influence such as trees and shrubs.

The shrink–swell hazard is controlled by a number of factors, primarily, the geology and mineralogy and the climate. Shrinkage and swelling usually occurs in the near-surface to depths of about 3 m; water content in this upper layer is significantly influenced by climatic and environmental factors and is generally termed the active zone. The shrink–swell potential of expansive soils is determined by its initial water content; void ratio; internal structure and vertical stresses; as well as the type and amount of clay minerals in the soil.

To understand and hence engineer expansive soils in an effective way, it is necessary to understand soil properties, suction/water conditions, temporal and spatial water content variations, and the geometry/stiffness of foundations and associated structures.

Cross-References

► Casagrande Test
► Classification of Soils

▶ Clay
▶ Cohesive Soils
▶ Collapsible Soils
▶ Hydrocompaction
▶ Noncohesive Soils
▶ Organic Soils and Peats
▶ Residual Soils
▶ Saline Soils
▶ Soil Field Tests
▶ Soil Properties

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Exposure Logging

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Synonyms

Exposure mapping; Trench logging; Trench mapping

Definition

The making of a geological map of vertical (or near-vertical) face(s), whether natural or man-made.

The U.S. Bureau of Reclamation (USBR 1998) describes standards for the following types of exposure logging: Dozer [bulldozer] Trench Mapping, Backhoe Trench Mapping, Large Excavation Mapping, and Steep Slope Mapping (Fig. 1).

Prior to logging, the exposure must be cleaned well enough to expose the features of interest (structures, stratigraphy, and soil horizons). This requires removing vegetation and any thin regolith cover (on natural exposures) or material smeared on the excavated face by excavating machinery (on excavated exposures). The detail shown in the log is

Exposure Logging, Fig. 1 Multibench trench excavated during an active fault study, USA. Each vertical wall is ~1.2 m high and horizontal benches are the same width. Walls have been cleaned enough to differentiate the major deposits. All spoil from wall cleaning was removed from bench surfaces so contacts can be traced across benches, allowing 3D mapping

dependent on how well cleaning was done; insufficient cleaning will obscure subtle structures and contacts that may be critical for interpretation.

In logging, one first defines mappable units (depending on the purpose of logging), and then draws or transfer their boundaries to some type of scaled drawing or image of the face that faithfully reproduces them. The lithologic units on an excavated face are normally unconsolidated (Quaternary) sediments, and are differentiated as discrete deposits characterized by a consistent texture, sorting, bedding, fabric, or color (McCalpin 2009). Soil horizons, in contrast, are post-depositional weathering zones that may be developed on a single lithologic unit, or may be developed across multiple lithologic units. Defining units on trench walls is facilitated if visual contrast is enhanced. For example, contacts in dry sediments may appear sharper if walls are sprayed/misted with a portable water sprayer. Slight differences in deposit cohesion are accentuated if the trench wall is left to "weather" for several days or weeks. Similar relief can be created by brushing the face with brooms or paintbrushes.

Contacts identified visually are then marked on the face before logging, e.g., by scribing a line with a sharp tool (in finer sediments). In coarser sediments, one marks contacts with nails and attached colored flagging, or with spray paint, using unique colors for soil horizons, depositional contacts, erosional contacts, faults, etc. In the corresponding trench log, lines depicting target features of the highest importance for the particular study (e.g., faults, tension cracks, liquefaction features, landslide shear planes, sinkhole collapse zones, angular unconformities) are rendered by the thickest lines; lithologic contacts by thinner lines; and soil horizon boundaries or facies boundaries within major (genetic) depositional units by very thin or dashed lines.

If deducing the time history of an exposure is important to the project, *soil horizons* should be identified and logged separately from deposits, because they indicate the location of past ground surfaces in the stratigraphic sequence, and their degree of development may indicate the length of time that surface was stabilized. The interaction of soil profiles with lithologic units and structures is often critical to understanding the sequence of depositional events versus deformation events and their relative timing (Shlemon 1985). To accurately identify and map soil horizons separately from lithologic units on an exposed face requires some formal training in pedology, something that many engineering geologists lack. Techniques for recognizing and delineating soil horizon contacts are beyond the scope of this article; see Shlemon (1985), Birkeland (1999), and Borchardt (2010) for applications of pedology to fault trenching.

Exposure Logging Philosophies

There are many reasons to map an exposure in engineering geology, so map units should be defined in a way that best achieves the goal. Two end-member philosophies are subjective versus objective logging. In *subjective logging,* the logger first observes the trench wall and makes a visual/mental interpretation of the structural and stratigraphic relations exposed in the wall. The log is then made to illustrate the salient geologic features. The rock or soil matrix is added in secondary importance; small features that do not bear on the major interpreted structures or strata may not be logged at all. The log is thus planimetrically accurate but schematic. The subjective approach developed during nuclear power plant investigations in the 1960s when the log was meant to answer specific regulatory questions, such as "Is a fault present?" and, if so, "Is the age of faulting older than some predefined regulatory criterion?" Subjective logs can be made rapidly and are easy to interpret with respect to regulatory criteria, because all extraneous features that do not bear on the major interpretation have been omitted. The disadvantage of subjective logs is that it is difficult to advance alternative interpretations of the log, because the interpretation was integral to drafting the log, and thus many details (which might conflict with the interpretation) have been omitted.

In contrast, *objective logging* depicts all physical features on the trench face in an impartial manner without regard to perceived importance. The approach documents only what the trench wall looked like, so is similar to an unannotated photograph of the trench wall. The advantage of an objective trench log is that multiple interpretations can be proposed/tested against the relationships portrayed on the log. The log is also an unbiased archival record of how the trench wall appeared, which has archival value. The disadvantage of strictly objective logs is they may not be readily interpretable because the log is not annotated to support an interpretation. In practice, most trench logs combine subjective and objective aspects, with the former dominating the twentieth century and the latter dominating the twenty-first century.

Exposure Logging Techniques

Over the past 40 years, trench logging techniques have evolved from simple sketching on graph paper, to increasingly sophisticated digital techniques. Nevertheless, all engineering geologists should still be able to make a trench log using the manual method (McCalpin 2009). As of 2018, the 2D photomosaic logging method is arguably most widely used, especially in the consulting sector, but 3D digital methods will probably replace it within the next decade.

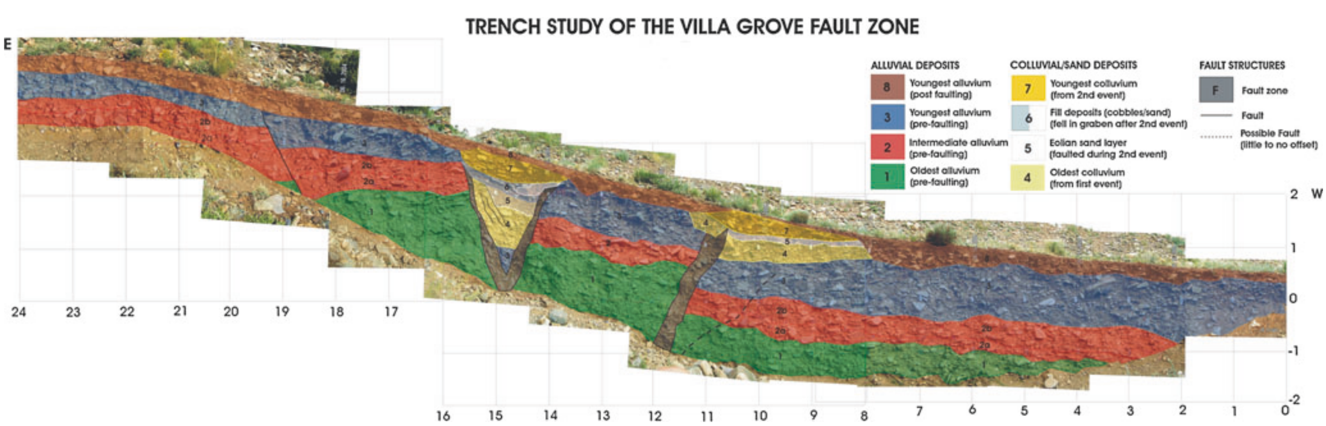
The two-dimensional photomosaic method became the standard for research-grade studies around the year 2000. Normally, the wall would be cleaned, horizontal and vertical reference marks attached to the face, and all contacts marked before taking the wall photographs. Each photograph would then be rotated, rescaled, stretched, contrast-enhanced, and trimmed as needed, before being added to the mosaic, with the assistance of the reference marks on the wall. After the mosaic was complete, the author would annotate the photomosaic with vector graphics software to illustrate the interpretation (Fig. 2).

In the mid2000s, computer software became available for creating three-dimensional images of man-made and natural exposures. This was done by terrestrial lidar surveying or by photogrammetry software. The earliest software used digital photographs aimed at pit-wall mapping in large mines where access to highwalls and benches was difficult (e.g., Sirovision; JointMetrix; see Haneberg et al. 2006). The software emphasized identifying faults and joint sets in bedrock in 3D by measuring their strikes-and-dips interactively from the 3D model. The data were then input into stereographic plots to define stability domains for slope stability calculations. Different rock types or structural domains could also be mapped as overlays on the 3D model. In 2010, Russian developers released a user-friendly software package (Agisoft PhotoScan) based on the Structure-from-Motion algorithms, which created a 3D model from numerous overlapping photographs. The photographs could be taken from the air looking down to the surface, or from the surface looking at cliffs, outcrops, or trench walls. The latest version of mapping natural outcrops and cliffs is termed Digital Outcrop Models (e.g., Wilkinson et al. 2016). Because the techniques are based on photographs, one should mark all possible contacts on the trench walls before taking the photos. Photogrammetric logging has advantages in the objective sense [it uses a high-resolution, georeferenced 3D model of the wall(s)] and in the subjective sense (by adding the third dimensions, strata and structures can be seen in their true 3D shape/orientation, rather than just in a 2D section, and this may change the interpretation). Within the next 10 years, 3D logging (e.g., Reitman et al. 2015) may replace 2D logging as the standard of practice.

Applications of Trenching in Engineering Geology

Trenching in engineering geology usually has one of two targets: (1) to expose and characterize structures (fault and joints, shear zones, landslide planes), in order to assess past movement history and/or future hazard [*structural targets*], or (2) to expose a Quaternary deposit in section, in order to assess its stratigraphy, sedimentology, geotechnical parameters, or to collect samples for dating the deposit [*stratigraphic targets*].

To date, most structural targets have been active faults studied as part of a seismic hazard assessment (McCalpin and Shlemon 1996). Since 1970, the field of paleoseismology



Exposure Logging, Fig. 2 Two-dimensional photomosaic of a trench wall in coarse-grained alluvial fan deposits, with overlaid semitransparent colored polygons depicting geologic units and structures. This combines the objective qualities of the photomosaic with the subjective qualities of interpreted trench contacts (lines and polygons) has grown considerably, due to government regulations requiring fault trenching studies for critical structures (dams, power plants, pipelines, etc.). Since 1990, trenching has been expanded to characterize deformation caused by strong ground shaking (e.g., liquefaction, sand blows, clastic dikes, earthquake-triggered landslides). Recently, trenching been applied to landslide studies, to supplement the more traditional methods of obtaining subsurface information (drilling and geophysics). As pointed out by Cotton (1999), the structures that define the head, flanks, and toe of a landslide are structural targets essentially identical to the normal, strike-slip, and reverse faults (respectively). In the past few years, landslide workers such as Gutiérrez et al. (2010) have demonstrated the advantages of trenching for answering questions about past landslide movement patterns, that were previously unanswerable. Similar results have come from trenching deep-seated gravitational slope deformations (DSGSD) and sinkholes.

Trenching for stratigraphic targets includes: (1) test pits to determine gross lithology, soil classification, and geotechnical parameters of shallow deposits; (2) trenching floodplain deposits to estimate flood depths and ages; (3) trenching alluvial fans to expose the number of debris flow deposits at various parts of the fan, their average thickness (proxy for flow depth), and to date their recurrence interval.

Summary

Logging of exposures in trenches, excavations, and steep slopes is undertaken to determine physical features, lithologies, soil classification, and geotechnical parameters. It involves cleaning the exposure and, sometimes, cutting benches to identify 3D relationships. The task is commonly undertaken using photomosaics but is becoming increasingly digital. Exposure logging is also undertaken to detect palaeoseismic features to identify potentially active faults, structures of large landslides, and relationships and recurrence intervals of debris flows in alluvial fans.

Cross-References

▶ Deformation
▶ Engineering Geological Maps
▶ Excavation
▶ Faults
▶ Geohazards
▶ Lidar
▶ Peels
▶ Photogrammetry
▶ Site Investigation

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Extensometer

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Synonyms

Extensometer

Definition

The extensometer is an instrument designed to measure the distance separating two fixed points by determining extension or contraction of a connecting element under stress which is temporarily or permanently attached to the fixed points.

E

Characteristics

The first such instrument was designed to measure deformation of iron rods during fatigue testing (Huston 1879). There are other instruments allowing determination of distance between fixed points by direct distance measurements (e.g., precision tape; laser distance meters; electronic distance meters) without using connecting element under tension.

Repeated readings are required to detect changes of the connecting element length which indicates relative displacement of the fixed points with respect to each other. Determination of their movement vector or total displacement requires additional information which cannot be provided by the extensometric measurements alone but is largely affected by the monitoring setting (e.g., placement of the fixed points with respect to geological and engineering structures, Corominas et al. 2000) which requires at least one point (i.e., reference point) to be stable or to move at much slower rates compared to the other fixed points.

Typical use of extensometers represents, but is not limited to, measurements of deformations across cracks on buildings and rocks, closure of underground constructions, convergence of building structures, slope deformations, and ground settlement. The specific application and monitoring setting determines the design of the extensometers among which number of types can be distinguished based on operational mode (portable/fixed; analogue/digital measurement readings; surface/borehole; single/series of interconnected extensometers), which often requires remote access and data downloading; type of connecting element (tape; cable; rod); and measurement technology (e.g., potentiometers measuring electric resistance; vibrating-wire transducers measuring frequency response; linear variable differential transformer measuring induction).

Accuracy of the measurements depends on the instrument design, in particular the deformation properties of the connecting element (e.g., steel tape; lead cable) and mechanism of conversion of the mechanical change (distance) into recordable readings. The latter may involve number of different electronic sensors, the performance of which may be adversely affected by harsh environmental conditions (e.g., temperature; humidity; corrosion; electric surge) under which extensometers often operate (Lin and Tang 2005). Temperature-induced deformations of the connecting element also have to be carefully considered during data processing. A possible source of errors, common to all types of extensometers, concerns the stability of the fixed points which may deteriorate through time disrupting the time series of the measurements.

Cross-References

► Deformation
► Dilatancy
► Instrumentation
► Landslide
► Mining Hazards
► Monitoring
► Site Investigation
► Strain
► Stress
► Surface Rupture
► Surveying
► Tension Cracks

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F

Facies

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Definition

Originally, an association of sedimentary rocks that is indicative of a specific depositional environment and/or set of depositional processes (Allaby and Allaby 1999) but later extended to:

- Bodies of metamorphic rocks with mineral assemblages characteristic of particular temperature/pressure regimes during progressive metamorphism
- Seismic facies which are units that have internal seismic reflection characteristics that contrast with those of adjacent reflection units

Synonyms

Metamorphic facies; Sedimentary facies; Seismic facies

Sedimentary Facies

The term was introduced in 1838 by Amanz Gressly for associations of sedimentary rocks that are characteristic of particular environments of formation. Johannes Walther later proposed a Law of the Correlation of Facies stating that a vertical sequence of facies is the product of a series of **depositional environments**, which initially lie laterally adjacent to each other, in situations where there is no break in the sedimentary sequence. Lateral changes in environments of deposition bring one facies to rest on another. Good examples are marine **transgressions**, which bring deeper water facies to rest on shallower water facies. Marine **regressions** show the reverse facies trend and often lead to emergence and erosion of the underlying facies. Gressly also recognized that some fossils are characteristic indicators of particular environments ("facies fossils") as opposed to those which are good indicators of levels in geological time (Cross and Homewood 1997) (Fig. 1). Sedimentary facies can be described in terms of:

(a) Petrological characteristics, such as grain size and mineralogy, and sedimentary structures (*lithofacies*).
(b) Assemblages of body and trace fossils (*biofacies*). The term *palynofacies* is used for associations of organic microfossils such a spores and pollen.

Metamorphic Facies

A metamorphic facies consist of associations of minerals characteristic of formation in particular pressure and temperature settings during **progressive metamorphism**. Pressure increases downward in the Earth's crust but also varies laterally during tectonic movements and close to magmatic/volcanic processes. Temperature also tends to increase with depth but is affected also by upward heat flow in parts of the crust. Therefore metamorphic facies reflect depth and dynamic, thermal and magmatic process during formation. Availability of water also influences which minerals form. Subsequent processes, such as **retrograde metamorphism** and **weathering**, may alter mineral assemblages requiring deductions about the former mineral assemblages and, therefore, the actual metamorphic facies (Turner 1948) (Fig. 2).

Seismic Facies

Seismic facies can be defined as crustal units that differ from adjacent units in **seismic characteristics** including: reflection amplitude, dominant reflection frequency, reflection polarity,

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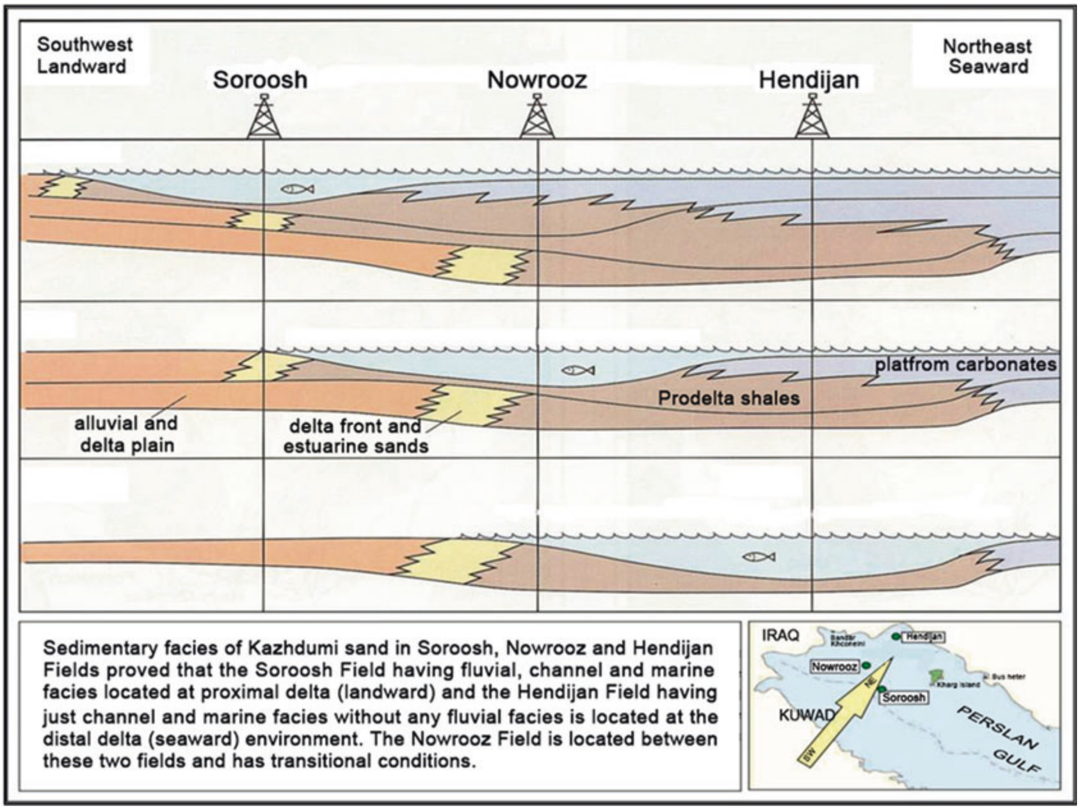
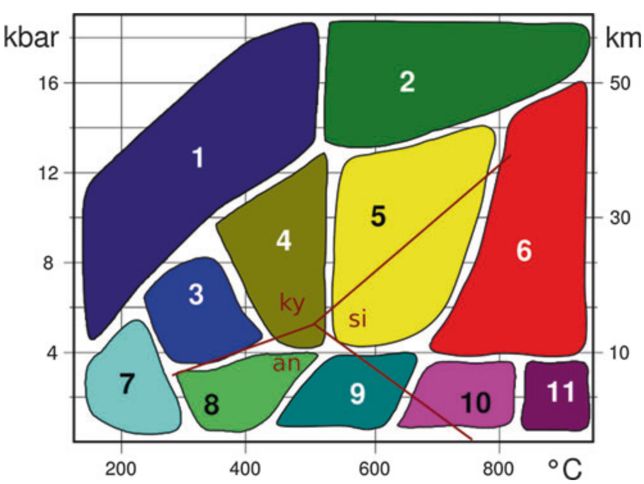
https://doi.org/10.1007/978-3-319-73568-9



Facies, Fig. 1 Example of sedimentary facies relationships in the northwestern Persian Gulf (Source: Noori et al. 2016: (copyright) the authors and Scientific Research Publishing Inc. Licensed under Creative Commons Attribution International Licence CC BY 4.0)

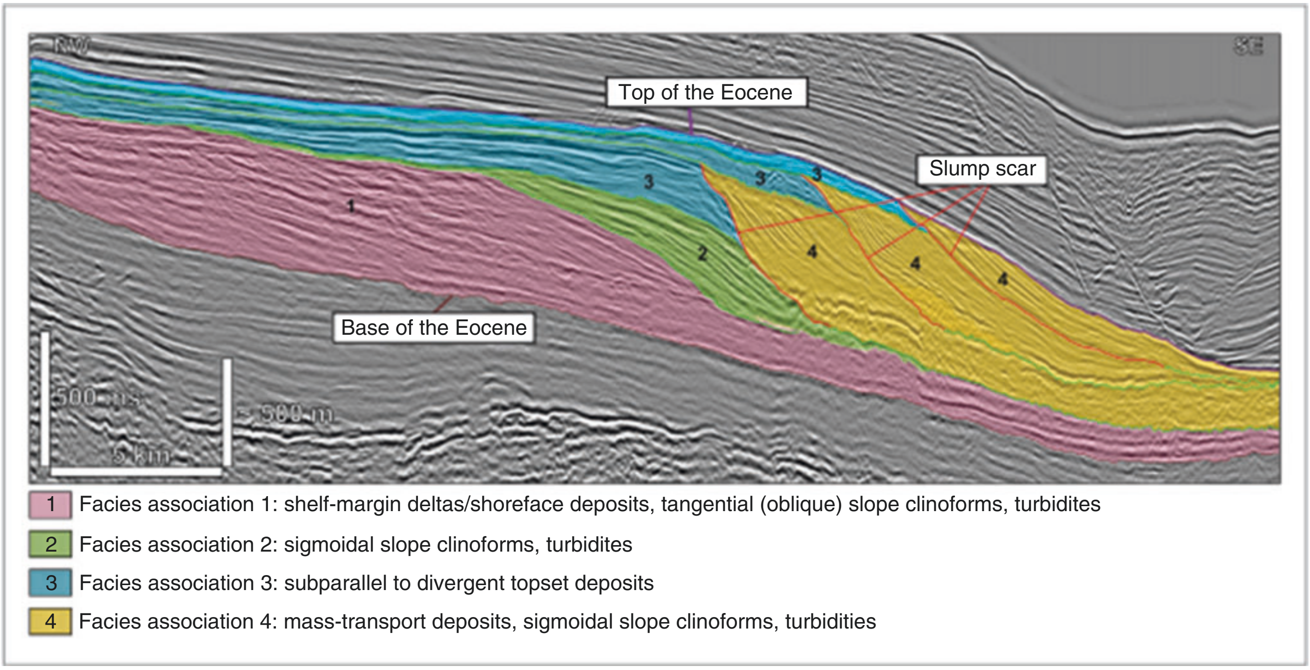


Facies, Fig. 2 Metamorphic facies (Source: Wikipedia Commons. Licensed under Creative Commons Attribution Share-Alike 3.0 Unported License). Key to facies: *1* – Blueschist; *2* – Ecologite; *3* – Prehenite-Pumpellyite; *4* – Greenschist; *5* – Amphibolite; *6* – Granulite; *7* – Zeolite; *8* – Albite-Epidote Hornfels; *9* – Hornblende Hornfels; *10* – Pyroxene Hornfels; *11* – Sanidinite

interval velocity, reflection continuity, reflection configuration, abundance of reflections, geometry of the unit, and relationship to those other units (Fig. 3). Characteristics are analyzed to deduce lithology, porosity, fluid content, relative age and types of stratification, and the depositional setting (e.g., processes, directions of transport, and events such as transgression, regression, subsidence, uplift, and erosion) as well as cross-cutting igneous phenomena (e.g., sills, dikes) and structural features (folds, faults, etc.). Results of analysis are shown on cross sections and facies maps (Roksandić 1978).

Relevance to Engineering Geology

The facies concept is fundamental to interpretation of sedimentary and metamorphic successions and processes and, therefore, to the interpretation of geological sequences from the results of direct and indirect investigation of the ground and interpolation between observation points in geological (and therefore engineering geological) mapping. It is also assists prediction of geological conditions that might be encountered during site investigation. In the past few decades, seismic sequence stratigraphy has become a key to understanding the deeper subsurface and is largely based on facies-related identification of relationships between seismic reflection units.



Facies, Fig. 3 Example of a seismic profile and facies analysis interpretation from the northern Santos Basin, southeastern Brazil (Source: Berton and Vaseley 2016. Distributed under Creative Commons Attribution License)

Cross-References

- ▶ Environments
- ▶ Metamorphic Rocks
- ▶ Sedimentary Rocks
- ▶ Sequence Stratigraphy

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Factor of Safety

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Definition

The Factor of Safety quantifies the capacity of a system beyond the applied loading or stress regime. For each failure mechanism, the Factor of Safety is calculated as the ratio of available forces preventing failure (including the maximum structural strength) to the forces driving failure (Ryder 1969; Wyllie 2018). It is sometimes referred to as Safety Factor.

Use

The Factor of Safety is a common approach to address the uncertainties associated with the available strength of structures, the applied loads, and future changes (Ryder 1969). The Factor of Safety evaluates the state of applied stresses to strength of a structure. Values above one indicate a capacity in excess of the applied stresses. A value of one is considered as the state of Limit Equilibrium. The Factor of Safety is also used to specify safety standards for design. In this context, the capacity of the structure is designed in excess of the expected loading as specified by the design Factor of Safety.

The Factor of Safety can be calculated directly from the ratio of resisting forces to driving forces (after Wyllie 2018):

$$Factor\ of\ Safety = \frac{\sum available\ forces\ preventing\ failure}{\sum forces\ driving\ failure}$$

where the available forces preventing failure include the available structure strength and the forces driving failure can include applied and future loads. Another common approach to calculate the Factor of Safety considers the ratio of the available structure strength to the mobilized strength required for a state of Limit Equilibrium (Ryder 1969):

$$Factor\ of\ Safety = \frac{available\ or\ ultimate\ strength}{mobilized\ or\ working\ strength}$$

In this approach, the forces acting on the structure (external and internal forces) are in equilibrium, and the mobilized strength (strength required for this state of equilibrium) is calculated. This approach can be applied to material friction, cohesion, tensile strength, bearing capacity, among others (see Shear Strength and Bearing Capacity).

Regardless of the approach, calculating the Factor of Safety requires consideration of the available maximum strength (peak strength, residual strength), foreseeable changes of strength with time (decreasing cohesion or friction), and future changes of the applied loading and stress regime.

Cross-References

- ▶ Bearing Capacity
- ▶ Failure Criteria
- ▶ Strength
- ▶ Stress

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Failure Criteria

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Definition

Modeling the behavior of soil, rock, or another material under general stress conditions requires the assumption of a mathematical model to describe the response. Many criteria have been formulated to model yielding and failure of a material. The "Mohr–Coulomb" and "Generalized Hoek–Brown" are the most frequently used failure criteria (Mortara 2008; Ulusay and Hudson 2012).

The Generalized Hoek–Brown criterion is an empirical failure criterion that provides the strength of rock based on major and minor principal stresses. The strength envelopes predicted with this criterion agree well with laboratory triaxial tests of intact rock and observed failures in jointed rock masses.

The Generalized Hoek–Brown criterion is non-linear, and the relationship between the major and minor effective principal stresses (σ_1' and σ_3') can be shown in the following equation:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \tag{1}$$

where:

- $\sigma_1{}'$ and $\sigma_3{}'$ are the axial (major) and confining (minor) effective principal stresses, respectively
- σ_{ci} is the uniaxial compressive strength (UCS) of the intact rock material
- m_b is a reduced value (for the rock mass) of the material constant m_i (for the intact rock)
- s and a are constants which depend upon the characteristics of the rock mass

Practical means of estimating the material constants m_b, s, and a are required for this criterion, as it is practically impossible to carry out triaxial tests on rock masses at the scale, which would be necessary to obtain direct values of the parameters. Hoek et al. (2002) provided the parameters of the Generalized Hoek–Brown criterion by the following equations:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{2}$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{3}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \quad (4)$$

where:

- *GSI* (the Geological Strength Index) relates the failure criterion to geological observations in the field.
- m_i is a material constant for the intact rock.
- *D* is a "disturbance factor" which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and/or stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses.

The Mohr-Coulomb criterion is the most common criterion encountered in geomaterials, in particular, soils. The Mohr-Coulomb criterion describes a linear relationship between normal and shear stresses (or maximum and minimum principal stresses) at failure.

The direct shear formulation of the criterion is given by Eq. 5:

$$\tau' = c' + \sigma_n{}' \tan\phi' \quad (5)$$

The Mohr-Coulomb criterion for triaxial data is given by Eq. 6:

$$\sigma_1' = \frac{2c'\cos\phi'}{1 - \sin\phi'} + \frac{1 + \sin\phi'}{1 - \sin\phi'}\sigma_3{}' \quad (6)$$

where c is the cohesive strength, and ϕ is the friction angle (Rocscience Inc 2004).

Cross-References

- ▶ Factor of Safety
- ▶ Hoek-Brown Criterion
- ▶ Modelling
- ▶ Mohr Circle
- ▶ Normal Stress
- ▶ Shear Stress
- ▶ Strength

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Faults

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Synonyms

Break; Break line; Fault line; Fault plane; Fault surface; Step

Definition

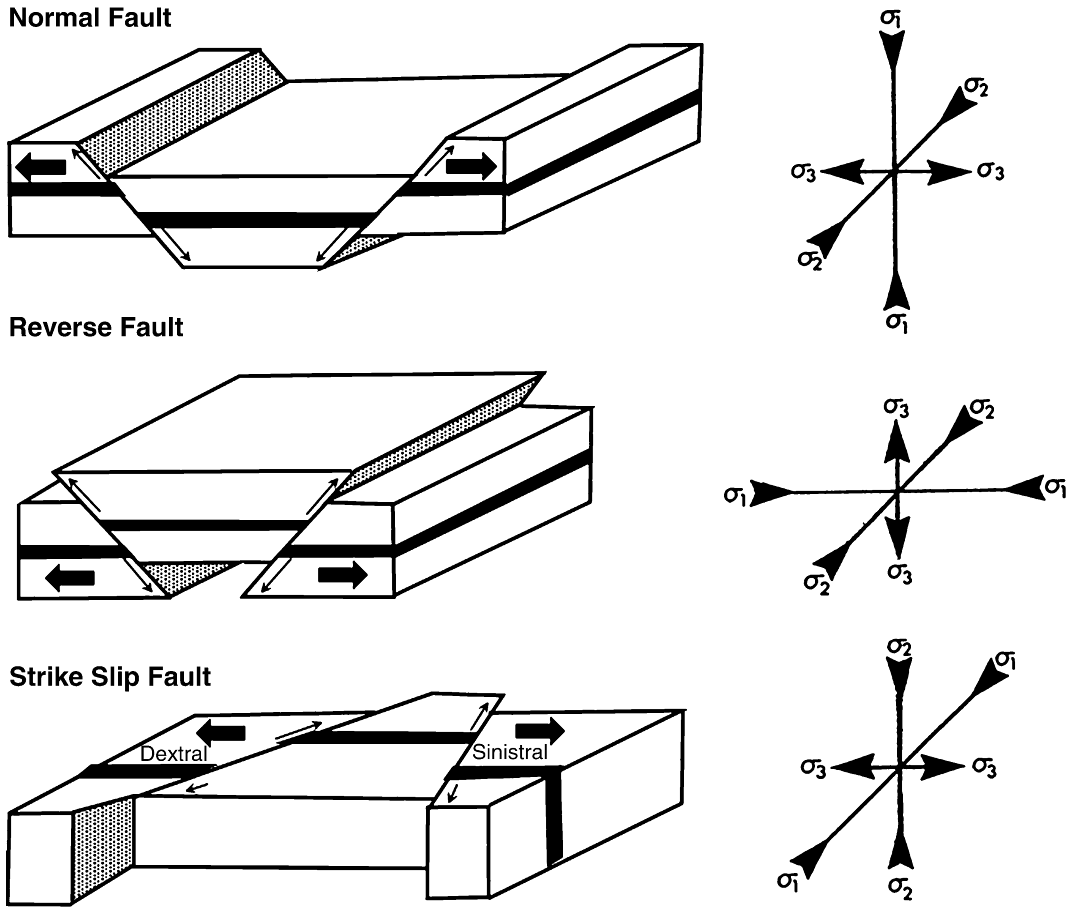
A fault is a discontinuity in soil, surficial deposits, or rock, where there is observable and measureable displacement from shearing.

Characteristics

The term fault was originally used by coal miners in the United Kingdom (UK) to describe dislocations in coal seams where the miners found themselves "at fault" with the strata. Localized mining terminology that may still be used to describe faults includes "troubles," "slips," and "shifts."

Faults are a common geological structure found in soils and rock types of all geological ages on the continental and oceanic crust. Faults vary considerably in size from submillimetric, visible in thin sections to major crustal dislocations up to thousands of kilometers long (Bryant 2013).

Brittle faults occur in the upper part of the Earth's crust. These are shear surfaces that originated during elastic deformation of a rock mass where the tectonic shear stress exceeded the shear strength. Initially, two conjugate and complementary shear surfaces develop, with their orientations being dependent on the principal, intermediate, and minimum directions of stress (Fig. 1). Subsequently, there is the preferential development of one shear surface. Brittle fracturing is the dominant mechanism generated by fracturing, grinding, and rigid-body deformation. By comparison, ductile faults are shear surfaces that develop when the strain exceeds the ductility of the rock mass undergoing deformation and is more common at deeper crustal levels. Ductile



Faults, Fig. 1 Structural classification of faults (Modified after Blés and Feuga 1986)

Faults, Fig. 2 Small fault associated with folding, Turkey (Photo: Dr Laurance Donnelly)

faults development depends on several inter-related factors such as temperature, pressure (depth), crystal plasticity, recrystallization creep, and grain boundary sliding. Faults may have both brittle and ductile characteristics (Fig. 2).

Faults can deflect, diffract, and reflect through strata; therefore the strike, throw, and dip of a fault may vary. Faults rarely occur as a single, discrete, planar discontinuity. More commonly, they are represented by a zone of deformation. Typically, this comprises complex, predominantly sub-parallel, interlocking discontinuities, with an infilling fault gouge (Wibberley et al. 2008; Jones et al. 1998). The shearing process leads to the disintegration of the original rock fabric, reduction in rock strength, generation of heterogeneity complex structures, alteration of the engineering properties, generation of shear and tensile discontinuities, and formation of a cohesion-less sediment or fault gouge, clay infill, fault breccia, cataclastic rock, pseudotachylite, or mylonite. Secondary mineralization may result in deposits of oxides or carbonates that may "flow" around angular fragments of stronger intact rock. These may show evidence of rotation and shear disruption. In voids, epigenetic minerals of potential economic value may become deposited.

Kinematic indicators provide evidence for the direction of displacement of a fault. Slickensides are nonpenetrative, smooth, or polished surfaces, which develop on planes of shear displacement between rock faces. The grooves and striae produced reflect the inscription of the direction of slip. Note that slickensides generally record the direction of the last relative displacement as earlier slickensides may be destroyed (Fig. 3). Some slickensides contain secondary fibrous minerals. The walls rock of faults may contain drag fold, distinct joint sets, or mineralized joints. Small scale sigmoidal, "S" and "Z" folds, en-echelon vein arrays, and reidal shears may also assist in determining the direction of net slip.

All faults terminate by the gradual disappearance of a fault as the displacement reduces to zero at the fault's tip line. Flexural folds may develop in the transition zone between the faulted and the unfaulted rock mass. Some faults may terminate abruptly against another (younger) fault. Alternatively, a fault may branch, splay, and divert into an array of small faults. Blind faults are faults that do not outcrop.

Synsedimentary faults are initiated during the deposition of sedimentary strata and are characterized by their listric form (concave) whereby the angle of fault dip reduces with increasing stratigraphic depth. There may be an increase in the thickness of sedimentary strata on the downthrown side of the fault, which is related to basin subsidence reflecting the sedimentary depositional environment.

Faults may be dry and impermeable groundwater barriers, aquicludes or aquifers containing significant quantities of groundwater that could be confined, under significant hydrostatic head pressure and with variable flow rates and directions.

Classification

Structural

The dip (angle between the horizontal and the fault plane) and strike (direction perpendicular to dip) describe a fault. However, "hade" (angle between the vertical and the fault plane) is sometimes used in coal mining to describe a fault. The displacement (slip or throw) describes the amount of movement across the fault. Faults are separated by fault walls, which are referred to as the upthrown side (or hanging wall) and the downthrown side (or footwall).

Several schemes have been presented to classify faults based on their structure. Commonly, faults are classified according to the displacement vectors between two points that were coincident prior to faulting (Fig. 4), as summarized below:

- Normal fault: The hanging wall moves down relative to the footwall and the translation is parallel to the fault plane.
- Reverse fault: The hanging wall moves upwards relative to the footwall. Low angled reverse faults may be termed thrusts, commonly associated with orogensis (mountain building due to converging tectonic plates), and these may develop into "overthrusts" accompanied by overfolds or recumbent folds.
- Strike slip fault: The direction of displacement is parallel and horizontal to the fault plane. Transcurrent (transform) faults are a type of strike slip fault that displace mid-ocean

Faults, Fig. 3 Multidirectional slickensides exposed on the hanging wall of a fault, Asia (Photo: Dr Laurance Donnelly)

Faults, Fig. 4 3.5 km long and 3 m high Tableland fault scarp and displaced stream channels, South Whales, United Kingdom. Fault reactivation was possibly initiated by valley deglaciation and exacerbated by coal mining subsidence (Photo: Dr Laurance Donnelly)

F

ridge constructive plate margins. Sinistral strike slip faults displace the rock mass to the left (anticlockwise), whereas for dextral strike slip faults the sense of movement is to the right (clockwise).

In rotational faults, the trajectory of displacement is arcuate and these may include scissor faults, pivot faults, and hinge faults. Antithetic faults describe a fault that is subordinate to a principal fault. Horst and graben (rift) structures develop in zones of tectonic uplift and horizontal extension (e.g., East Africa).

Engineering and Geotechnical

The structural and geometrical classification of faults does not provide information on the engineering properties of the fault. Various schemes have been presented for the engineering classification of faults. Whereby, the host rock, fractured and altered rock, matrix, and gouge are considered. For example, Riedmüller et al. (2001) differentiated cataclastic rocks into cohesive and noncohesive to determine if they were more aligned to behaving like a soil. Cemented cohesive fault rocks were further classified based on their type of cement and whether they produced a fault breccia or pseudotachylyte. The classification of cohesionless cataclatstic rocks was based on the relative proportion of intact blocks and the strength and stiffness of the matrix.

Geological Hazards and Geotechnical Constraints

Active Faults

A fault is considered to be active if there is evidence for its displacement in the Holocene (past ca. 11,500 years) or the past 8 million years in area of low seismicity for the siting of a sensitive engineered structure like a nuclear power station (Mallard and Woo 1991). Active faults occur at or in the vicinity of tectonic plate boundaries, although intra-plate faults are also capable of reactivation (e.g., new Madrid, Missouri, USA, in 1811 and 1812). Faults with a known history of recent reactivation and recurrence have a greater probability of future displacements. Fault recurrence varies from tens or hundreds of years to thousands of years. Some faults slip by gradual aseismic creep mechanisms. Active faults should be evaluated and avoided for the siting of infrastructure. The evaluation of faults is aimed at locating their precise position then determining the fault type, engineering characteristics, date of activity, offset, sense and magnitude of displacements, deformation, and probability for future surface displacement.

Ground Motion and Seismicity

Ground motion is the shaking of the ground caused by elastic waves generated from an active fault during an earthquake or volcanic hazard (e.g., eruption, explosion, lahar, landslide, or volcano-seismicity). The ground motion will depend on several factors such as the intensity, magnitude, fault geometry, attenuation, and ground conditions. Ground motions are incorporated into building codes. Ground motion may be artificially induced by hydraulic fracturing (e.g., shale gas exploration in the UK, 2011); coal bed methane, coal mine methane, and underground coal gasification; geothermal wells; mining (e.g., roof failure, pillar collapse, rock bursts, outburst, and open pit slope failures); nuclear test detonations (e.g., Nevada, USA, 1962); reservoir impoundments following the construction of dams (e.g., USA, Middle East and India); and the disposal of contaminated waste water into deep boreholes that penetrated faults (e.g., US Rocky Mountain Arsenal site, Denver, 1962–1965). Fault characteristics and parameters may be required as part of a Deterministic Seismic Hazards Assessment (DSHA), Probabilistic Seismic Hazard Assessment (PSHA), or Probabilistic Fault Displacement Hazards Analysis (PFDHA).

Liquefaction

When water saturated sediments are subjected to ground motion, their rigid strength may be lowered. Buildings and pipelines constructed on liquefiable ground can sink or tilt (e.g., Niigata, Japan, 1964). The potential for liquefaction can be evaluated by ground investigation, modelling, and an understanding of the engineering geology of specific location.

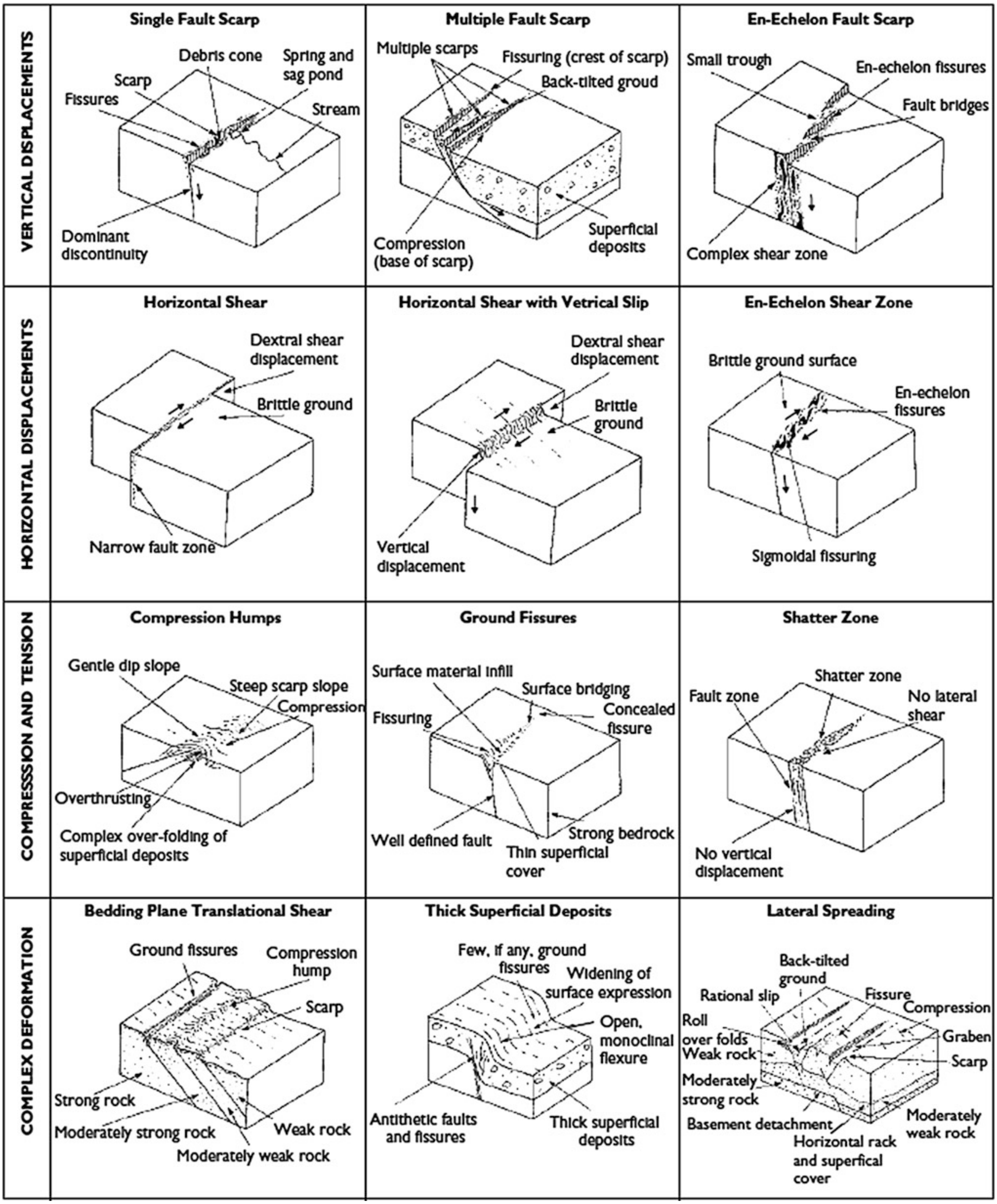
Ground Rupture

A ground rupture causes the offset or displacement of the ground surface and this typically generates a fault scarp or fissure. Primary ground ruptures of variable height and length may be observed following an earthquake, and smaller secondary ground ruptures occur on fault splays, subordinate faults, and sympathetic faults.

Deglaciation in parts of Canada, Scandinavia, and Northwest Europe has caused the reactivation of faults and ground ruptures. Scarps have deflected streams, disturbed and liquefied sediments. These scarps have been interpreted as originated during the Pleistocene (older than 10,000 years BP) and provide evidence for paleoseismicity (Davenport et al. 1989).

Ground ruptures may be artificially induced by the following:

- Coal mining subsidence-induced fault reactivation can cause the development of a fault scarp (step), graben, fissure, or compression humps along the ground surface (Donnelly 2006) (Fig. 5).
- The abstraction of groundwater, brine, oil, and gas can lead to fault reactivation and the generation of fault scarps and fissures up to several kilometers long (e.g., Houston and Galveston, Texas Gulf Coast, USA) (Holzer 1984).
- Groundwater decline due to over pumping and subsidence (e.g., California, Saudi Arabia, and China) (Holzer 1984).



Faults, Fig. 5 Mining subsidence-induced fault reactivation and variations in the style of ground deformation at fault outcrops (Modified after Donnelly and Rees 2001)

Tectonic Subsidence and Uplift

Fault reactivation may be associated with regional tectonic subsidence or tilting of coastlines by the order of several meters. For example, in Alaska following the 1964 earthquake approximately 285,000 km^2 of the sea bed and land was deformed by up to 3.5 m of vertical movement. Regional uplift and folding has also been observed as a consequence of fault movements. For instance, following the 1971 San Fernando, California earthquake, mountains in the region moved upwards and horizontally by approximately 2 m in each direction.

Gases

Gases that are potentially explosive, asphyxiant, or harmful to health, such as radon, methane, carbon dioxide, carbon monoxide, hydrogen sulfide, ammonia, and stythe (also known as "blackdamp" or oxygen depleted air, common in some coal mines) may discharge from mining-induced tectonic fault scarps and thermal springs (e.g., North Anatolian Fault, Turkey).

Landslides

Landslides and rock falls on slopes and in open pit mines can be induced by ground shaking. These range from isolated raveling and localized block failures less than 1 m^2 to landslides several kilometers long or the generation of thousands of small earthquakes up to many hundreds of kilometers from the causative fault. These may cause widespread loss of life and structural damage (e.g., Guatemala, 1976). In area of moderate to high relief, reactivated faults reduce substantially the strength of rock mass and provide permeable channel ways for large volumes of groundwater to drain onto upper slopes causing the initiation and subsequent reactivation of landslides (e.g., South Wales, UK).

Seiche, Tsunamis, and Flooding

Seiche (waves in a body of water induced by fault movements and earthquakes) can cause flooding of land around reservoirs or lakes, or the over flowing of dams (e.g., Nevada, USA, 1954). Tsunamis (large waves in an ocean caused by the vertical displacement of the sea floor) are highly destructive waves that travel at hundreds kilometers per hours and cause widespread damage to infrastructure and loss of life (e.g., Indian Ocean, 2004). Destructive waves can also be induced by landslides that fall into a sea or lake (e.g., Lituya Bay, Alaska, 1958). Groundwater may discharge from fault scarps and combined with the lowering of the ground surface as a result of ground rupture this may cause localized flooding.

Settlement

Engineered structures built across fault outcrops should consider the possibility for differential settlement caused by the contrasting bearing capacity and engineering properties of the intact displaced rocks or fault gouge.

Investigation

Desk Study

Geotechnical investigations should commence with a desk study to collate data and information on faults available from a variety of sources such as national geological surveys, professional bodies, universities, research centers, and consultants.

Earth Observation, Geological Mapping, and Geomorphology

Faults are susceptible to weathering and erosion. As such, Quaternary faults are rarely visible along the ground surface, unless they have "recently" ruptured. The analysis of satellite imagery (e.g., airborne geophysical surveys, infrared, LiDAR, airborne radar, digital elevation models, high resolution air photographs (taken at low sun angles), and stereo pairs) may enable the accentuation of geomorphological features associated with faults. This includes prominent high-angled fault scarps, infilled fissures, depressions, topographic lows, offset deflected stream channels, displacement of linear engineered structures (walls, roads, drains, building, utilities), saddles, sag ponds, rifts/graben, perched terraces, spring lines, thermal seeps and springs, gas emissions (noted in springs), shutter ridges, valley benches, notches, fault controlled drainage, and pressure ridges. In the field, the position of a fault may be inferred from small-scale geological structure in rock exposures including mineral veins, variations in joint trends, and density, drag folding, slickensides, or sudden changes in dip and strike of slopes.

Geophysics

Geophysics can be used to delineate faults; however, this must be preceded by a desk study and reconnaissance site visit to choose the most suitable suite of techniques. High resolution seismic reflection can be deployed over several kilometers to provide structural information on faults at intermediate (hundreds of meters) to deeper (several kilometers) stratigraphic levels in sedimentary basins. Conventional engineering geophysical surveys (e.g., seismic refraction, ground penetrating radar, magnetic, conductivity, and resistivity) can locate faults at shallower (up to several meters) depth. Gravity methods may be applicable if there is a strong density contrast across fault. It should be noted that geophysical surveys alone do not prove fault positions and any anomalies must be verified by trenching and/or drilling.

Trenching

Exploratory trenches may be excavated and inspected (following a detailed H&S assessment) to verify the presence or confirm the absence of a fault. Trench locations should be based on geomorphological mapping, interpolation, extrapolation, or geophysics result. Where faults are proven and exposed on trench walls, they should be accurately logged to determine the fault type, maximum displacements, maximum earthquake magnitude, annual slip rate, recurrence interval, timing of fault movement, displacement, and palaeoseismicity. Fault age dating methods include radiometric carbon-14, soil profile developments, weathering profiles, geomorphological processes, stratigraphic correlation of fossils, archaeological artifacts, historical records, tephrochronology, thermoluminescence, botany (lichens, mosses, fungi and plants colonization), paleomagnetism, and dendrochronology (Bonilla and Lienkaemper 1991).

Drilling

Faults can be explored by rotational drilling, which requires specialist and suitably experienced drilling contractors. Attention should be given to the borehole diameter and drilling flush to ensure that weaker fault gouge is not washed out and small scale shear structures are not destroyed. Where possible, double or triple tube core barrels and polymer or chemical flush additives help to stabilize the borehole resulting in greater recovery. Core logging should record the Rock Quality Designation (RQD), Fracture Index (FI), Solid Core Recovery (SCR), Fracture Frequency (FF), Geological Strength Index (GSI), and Rock Mass Quality (Q value). *In situ* testing and monitoring of faults boreholes may include the use of a high pressure dilatometer (stiffness), *in situ* hydraulic fracturing (rock stress), lugeon, slug and pulse tests, the installation of stand pipes, and piezometers to monitor the groundwater regime and cone penetrometer testing (CPT). Borehole geophysical methods might also be deployed to investigate faults (e.g., acoustic or optical televiewer, verticality, calliper, sonic, P-S suspension, natural gamma, gamma-gamma (density), neutron, fluid flow, and temperature logging).

Laboratory Testing

The mechanical properties of a fault zone may be determined by laboratory testing. Typical analysis includes quantitative clay mineralogy, triaxial, direct shear, and moisture. Due care must be taken to ensure the samples collected are not disturbed and are representative of the fault and the weaker fault gouge. The samples must be tested soon after core recovery as they are likely to deteriorate.

Mitigation

Fault outcrop positions should be avoided for the siting of engineered structures and infrastructure. If it is not possible for the known or extrapolated position of a fault to be avoided, for example, during the construction of a highway or pipeline, then engineering mitigation options should be considered to manage the expected deformation and displacements during the design life of the structure. This will require collaboration with design and structural engineers and the consideration of building design codes in seismically active regions. For example, pipelines may be placed in low angled wall trenches with a coarse granular fill trench to counteract the expected shear displacement throughout its design life.

Summary

Faults are discontinuities in rock or soil, where there has been shear displacements. Faults occur in rock types of all geological ages and are common throughout the Earth's crust. Faults are relevant in engineering geology as they generate natural and anthropogenic-induced geological hazards and geotechnical constraints. Active faults should be evaluated for the planning and development of new infrastructure and the building of sensitive or strategic engineered structures (e.g., nuclear power station, dams, bridges, hospitals, and pipelines) so that loss of life and damage or disruption to the engineered structure is minimized. Investigating and evaluating faults can be difficult as faults are rarely exposed. Typical investigative methods are geomorphological mapping, geophysics, observations in exploratory trenches, drilling, or indirect interpretive methods. Fault investigations must be pragmatic, proportionate, and cost effective.

Cross-References

▶ Deformation
▶ Earthquake
▶ Earthquake Intensity
▶ Earthquake Magnitude
▶ Effective Stress
▶ Engineering Geological Maps
▶ Engineering Geology
▶ Engineering Geomorphological Mapping
▶ Engineering Geomorphology
▶ Engineering Properties
▶ Fluid Withdrawal
▶ Foundations
▶ Gases
▶ Geohazards
▶ Geology
▶ Geophysical Methods
▶ Geotechnical Engineering
▶ Glacier Environments
▶ Ground Shaking
▶ Groundwater

► Groundwater Rebound
► Hazard
► Hazard Assessment
► Hydraulic Fracturing
► Hydrogeology
► Landslide
► Mechanical Properties
► Pore Pressure
► Reservoirs
► Risk Assessment
► Rock Mass Classification
► Rock Mechanics
► Rock Properties
► Shear Modulus
► Shear Strength
► Shear Stress
► Shear Zone
► Strain
► Strength
► Stress
► Subsidence
► Surface Rupture
► Volcanic Environments

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Field Testing

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Synonyms

In-situ testing

Definition

Field testing is a term used for all *in-situ* based methods and techniques that qualitatively and quantitatively determine physical, strength, deformability and hydromechanical properties of geomaterials in geotechnical site investigation. Geomaterials include rocks (igneous, metamorphic and sedimentary), soils and their complex, heterogeneous and anisotropic masses that consist of various components, fissures, flaws and other discontinuities at varying size scales.

Introduction

Complex engineering projects such as large infrastructure projects, electricity generating plants, underground openings for mining and nuclear waste repositories, tunneling, dam construction, open-pit mining and so on, require a comprehensive geotechnical site investigation (Schnaid 2009). A site investigation generally involves literature survey, field based measurements such as line surveying to identify geomechanical properties of discontinuities of rock masses and preparing index properties including soil profiles with representative cross-sections, taking suitable disturbed and undisturbed soil and rock samples for laboratory testing, and finally field testing. It is well known that field testing is an essential, expensive and time-consuming part of geotechnical investigation. Schnaid (2009) stated that a very good understanding of laboratory and *in situ* tests and the constitutive relationships that link geomaterials behaviour is very important for appropriate geotechnical design.

The ultimate objective of geotechnical investigation is to measure and then analyze engineering properties and behaviour of soils and rocks and to identify potential geotechnical hazards for estimating the performance and stability of any project. The most common potential geotechnical hazards related to the geomaterials and the construction of projects involve slope instability and landslides, induced stress depended instabilities in underground opening, foundation settlement, swelling and squeezing, liquefaction, managing underground water, and others. The strength, deformability, consolidation characteristics, compaction characteristics and hydraulic conductivity are well known engineering properties of geomaterials that are used in providing appropriate recommendations on controlling and mitigating geotechnical hazards. To date, many laboratory testing techniques have been recommended to measure some of these engineering properties of soils and rocks. However, as mentioned, these laboratory testing methods have some significant limitations and weaknesses to accurately assess the capacity of soils and rocks, particularly their strength and deformability properties. In addition to strength and deformability properties of geomaterials, the measurement of some engineering properties such as *in-situ* stresses, groundwater level and geomechanical properties of discontinuities that influence the strength and deformability characteristics of soil and rock masses can only be possible by field testing methods. Johnston (1983) recommended the geotechnical engineers to ask themselves the question of "*is the technique and are its results relevant?*" when they need to consider which test technique to use in geotechnical investigation to overcome any project related geotechnical problems.

Due to many different reasons, the upscaling strength and deformability properties of geomaterials to the field condition by utilizing the results obtained from laboratory tests or empirical approaches for stability evaluation of many above given complex projects is very difficult in some cases. These reasons are mainly disturbance during drilling, sampling, transportation, storage and preparation for laboratory testing, changes in stress conditions, scale effect of sample size, mechanical deformation, changes in water content and deficiencies arising from some assumptions (e.g., homogeneous and isotropic). Thus, field testing methods are considered to be more convenient methodology to overcome the limitations and weaknesses of laboratory tests and empirical models in understanding mechanical and physical behaviour of rock and soil materials and then to determine the strength and deformability characteristics of these geomaterials in geotechnical investigation. The procedures and other information related to the most commonly encountered field tests are presented in two other contributions entitled as "*Rock Field Tests*" and "*Soil Field Tests*" in this volume. Therefore, in this contribution, the main reasons of performing field testing for geotechnical investigations is discussed with brief introduction of major field testing methods utilized to obtain design engineering properties of soils and rocks.

Laboratory Testing Based Limitations

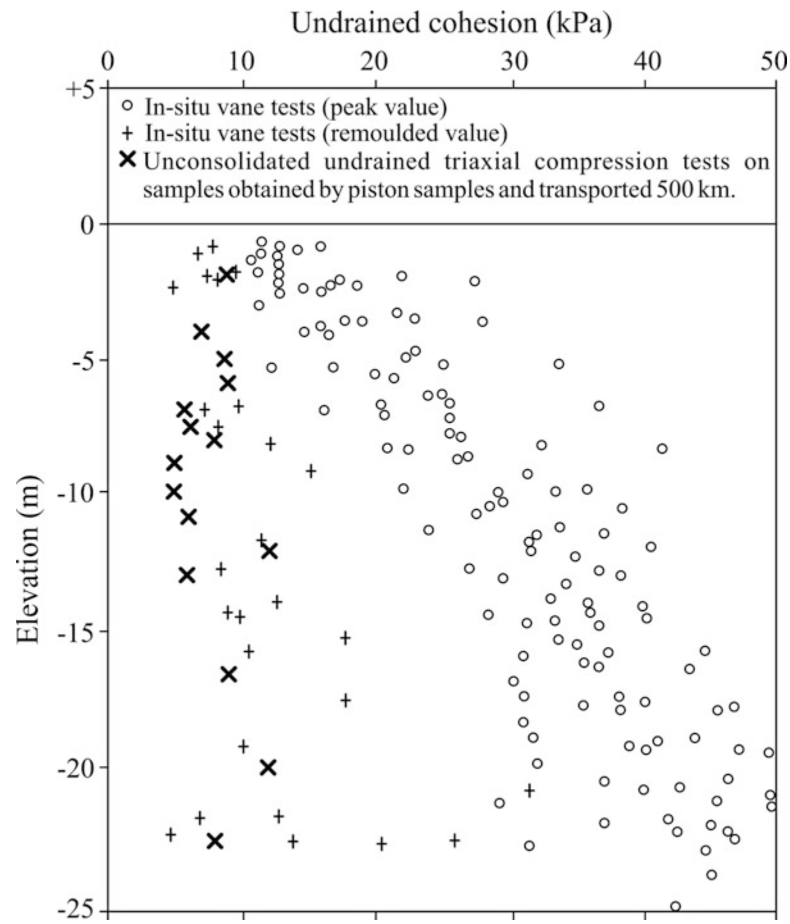
The conventional method to determine physical, index and mechanical (strength and deformability) properties of soil and rock materials consists of collecting samples from a project site and performing laboratory testing on specimens and then analyzing the obtained results. Although laboratory testing is the most preferred method in geotechnical investigations throughout the world, there are significant limitations and weaknesses with laboratory testing such as the disturbance of samples and scale effect of tested specimens. These uncertainties, which should not be neglected particularly in measuring design engineering parameters of geomaterials, are discussed below.

Whereas the disturbed samples are enough to be used for identifying some physical and index parameters of geomaterials, the undisturbed samples generally taken as soil or rock blocks or extracted by coring, pushing or driving techniques are required to measure the strength, permeability and deformation characteristics of soils and rocks. However, it is very difficult to collect undisturbed samples from various soils and rock materials. Although the disturbance problems may occur at different grades and forms during sampling of many soils and rocks, the very soft or sensitive soils, clay containing cobbles, cohesionless soils, soft clay-bearing rocks (claystone, mudstone, siltstone marl etc.), which are very sensitive to variations in water content, as well as fissile and highly fractured rocks are typical examples for such challenges of extracting undisturbed samples. According to Ameratunga et al. (2016), undisturbed sampling in cohesionless soils is still an unsolved problem in everyday practice. Erguler and Ulusay (2009) stated that many clay-bearing rocks break during coring, cutting, grinding and storing due to disintegration of such rocks when subject to change in the water content and an inherent weakness of bedding planes.

The disturbance issues are most likely to seen at the time of sampling and drilling. During drilling and sampling, some structural disturbance due to sampler, changes in water content and stress conditions influence the structural integrity of the in-situ geomaterials, particularly soil. Thus, it is not so easy to extract a perfectly undisturbed sample, especially when changes in stress conditions are considered. Further disturbance may be observed due to transportation, storage and then preparation for testing. Johnston (1983) emphasized such disturbance circumstances applicable to sensitive and cemented soils by investigating the undrained cohesion values a soft sensitive silty clay (Fig. 1).

The data given in Fig. 1 represent some laboratory and *in-situ* based testing results of a deep formation of soft sensitive silty clay. Unconsolidated undrained triaxial tests were performed on samples taken by piston sampler and carefully transported to the laboratory from a distance of 500 km (Johnston 1983). Considering the distribution of all undrained cohesion values in Fig. 1, it can be clearly seen that the *in-situ* peak cohesions increase with depth, whereas the residual *in-situ* cohesion values are in acceptable agreement with those obtained by performing triaxial tests on undisturbed samples. According to Johnston (1983), the evaluation

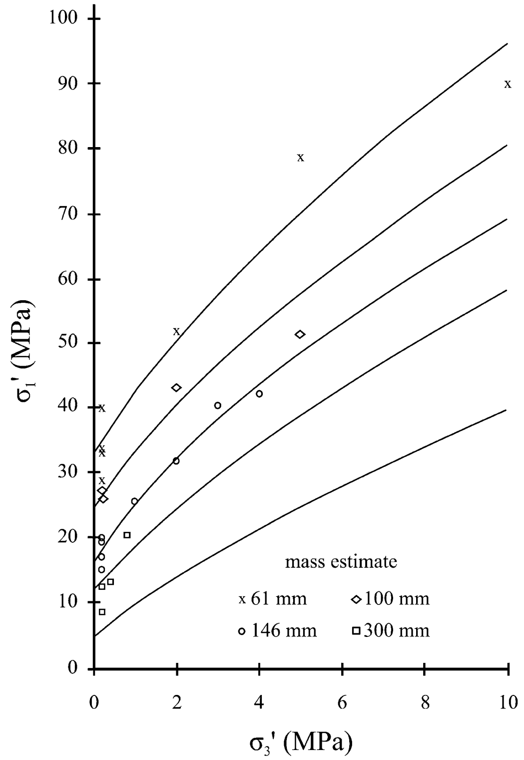
Field Testing, Fig. 1 The effect of disturbance on laboratory and field testing on a soft sensitive silty clay (Johnston 1983) (© 1983 Taylor & Francis Group, London, UK. Used with permission)



obtained from Fig. 1 indicates that the laboratory samples disturbance is far from satisfactory, and only laboratory testing based results should not be taken as design parameters.

The scale (or size) effect is another significant limitation of laboratory tests that should be certainly considered in selecting design parameters of soils and rocks. These geomaterials consist of quite different components on both the microscopic and macroscopic scale. Thus, the strength and deformability characteristics change depending on the corresponding sample size. Some laboratory testing such as uniaxial compressive strength (UCS) and triaxial tests require high quality specimens in accordance with the specifications suggested by international standards. These high-quality core samples having suitable length-to-diameter ratios recommended by standards and suggested methods for the laboratory tests cannot always be obtained, particularly from weak and clay-bearing rocks melanges, fault rocks, coarse pyroclastic rocks, breccias, ophiolitic rocks, as well as soft cohesionless soils. In addition, rocks show size dependent variations in their strength parameters (Fig. 2) and failure behaviour (Fig. 2). It is generally assumed that there is a significant reduction in strength with increasing sample size (Hoek and Brown 1980). Hoek (2007) suggested that the decrease in UCS with increasing sample size is as a result of failure through and around grains forming intact rock, and in the case of involving an adequately large number of grains in the sample, the strength of rock reaches a constant value in relation to the overall size of the structure being considered.

As seen in Fig. 2, the performance and stability of any engineering project is controlled by the strength and deformability of the whole mass of geomaterials. In such cases, it is difficult to define the overall structure strength and deformability features, since the failure mechanism of the composite structures appears to be different from that of the single component structures (Greco et al. 1991). Furthermore, the strength and deformability of rock masses are mainly a function of the strength of the intact rock and the geomechanical properties of discontinuities such as spacing, orientation with respect to the loading axis, joint surface condition (weathering, roughness, aperture, infilling, and persistence), and pore water pressure and the negative effect of water content on the strength of clay-bearing rocks. In order to see the effect of scale on the strength and deformation characteristics of intact rock and rock mass, Reik (1979) performed small-scale and large-scale triaxial tests on fractured and stratified limestone (upper Muschelkalk) and fractured limestone-mudstone sequence (Lias) and the results obtained from this study are given in Fig. 3. It is clearly



Field Testing, Fig. 2 Sample diameter depended changes in failure behaviour and peak strength for Moura coal, Australia (Reprinted from Medhurst and Brown 1998, Copyright (1998), with permission from Elsevier; © 1979 Taylor & Francis Group, London, UK. Used with permission)

seen from this figure that both strength and deformability parameters of these samples decrease with increasing core size.

Similar to rock materials, soil mass strength may not be accurately predicted from laboratory tests performed at a small scale for desiccated and fissured soils. The influence of the spacing of fissures on measured shear strength of London Clay can be clearly seen in Fig. 4. It can be inferred from this figure that the shear strength of London Clay would be over-estimated in the case of performing related tests on small scale specimens because of a very high possibility that these samples do not contain weaker fissures. Thus, for the design parameter, the diameter of London Clay samples should be at least two times the spacing of fissures due to significantly decreasing the shear strength with increasing sample size. In addition to strength and deformability parameters of soils, the permeability and thus the consolidation characteristics of soils also change upon the size of tested samples (Johnston 1983). Rowe (1972) studied the relationships between coefficient of consolidations (c_v) and sizes of samples by using data taken from piezometer readings close to highly fissured clay, full scale piezometer and settlement readings and finally small scale laboratory testing (Johnston 1983). When these relations are considered, it can be seen that the c_v values measured from laboratory tests are very low in comparison with other field based readings. Rosine and Sabbagh (2015) performed further experimental studies (one-dimensional consolidation tests on samples having diameter range of 100–250 mm and height range of 23–200 mm) to measure the scale effect on some compressibility parameters. They also found that the c_v value significantly varies with changing the diameter/height ratio. Johnston (1983) suggested to remove some of these important fabric-induced scale effects by testing as large a sample as possible.

To summarize, despite some advantages of laboratory testing such as well-defined boundary conditions, a possibility of multiple tests, uniform strain fields, controllable drainage conditions, following pre-selected and well defined stress paths during testing (Ameratunga et al. 2016), there are eventually a number significant limitations contained in laboratory testing. As described above in detail, these are mainly disturbance of samples and lack of success to extract sufficiently representative samples (scale effect) as well as changes in the *in-situ* stress condition. To overcome the limitation of laboratory testing, field testing methods are indispensable approaches to determine design parameters (such as strength and deformability properties) and engineering behaviour of rock and soil masses in geotechnical investigation.

The Limitations of Empirical Approaches

The testing methods, particularly those generally performed at field conditions, are expensive and time-consuming approaches in determining engineering design parameters of geotechnical projects. In the case of limited financial resources, empirical approaches derived based on laboratory or field data, past experience, and good engineering judgment (Ameratunga et al. 2016) are frequently utilized as preliminary feasibility studies to estimate engineering properties of soils and rocks or their masses. Ameratunga et al. (2016) emphasized that the shear strength and consolidation characteristics of a clay can be predicted by using simple index properties at little or no cost. The empirical approaches (e.g., empirical equations for predicting the deformation modulus of a rock mass and Hoek-Brown failure criterion) are also used for understanding engineering properties and behaviour of rocks and rock masses due to the fact that determination these properties by laboratory testing methods is very difficult when the scale effect (the presence of discontinuities) is considered. However, the predicted modulus of deformation can be nearly three times larger than the modulus of elasticity of intact rock in some cases. Furthermore, the empirical approaches assume the masses of soils and rocks as homogeneous and isotropic materials.

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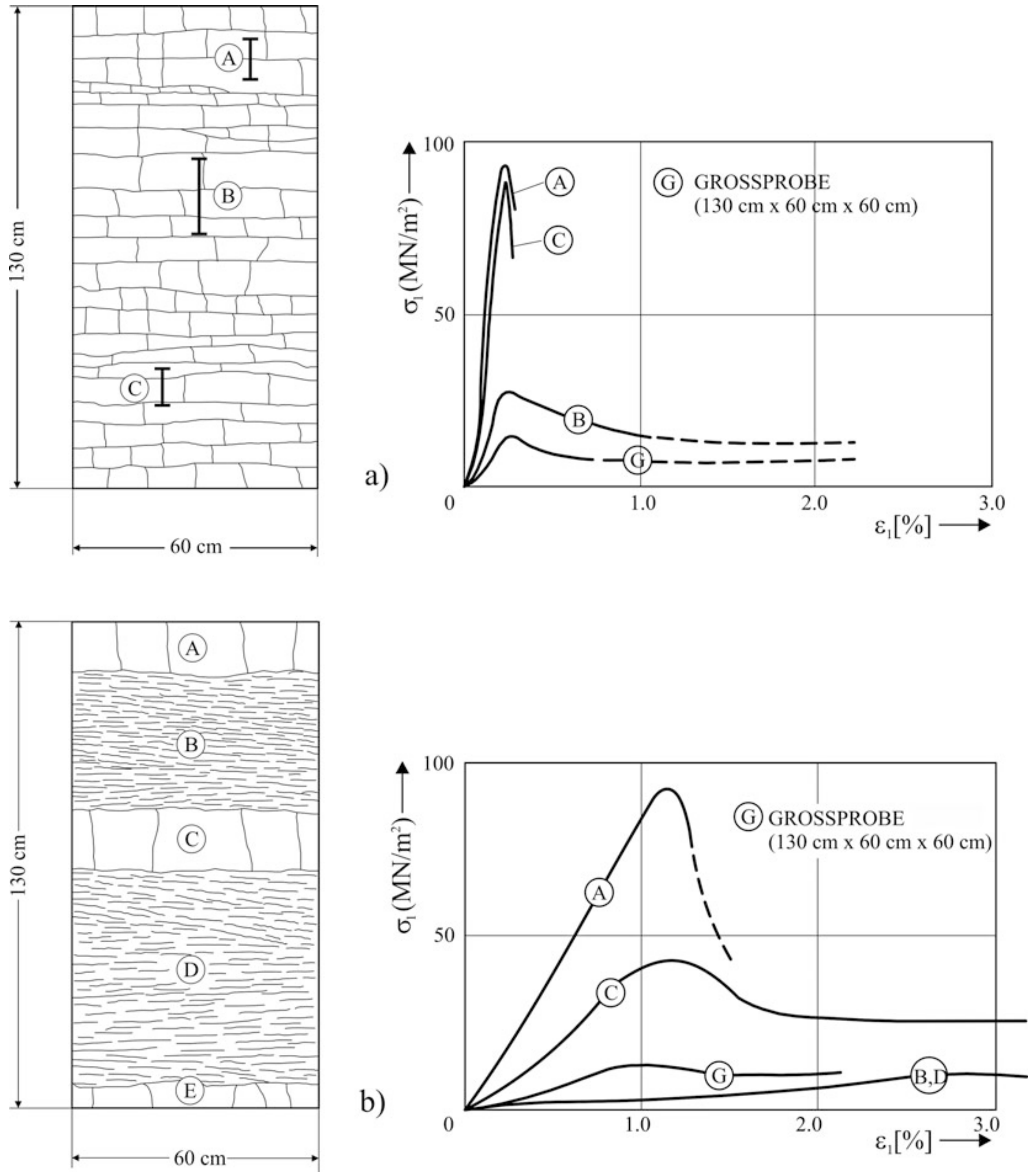
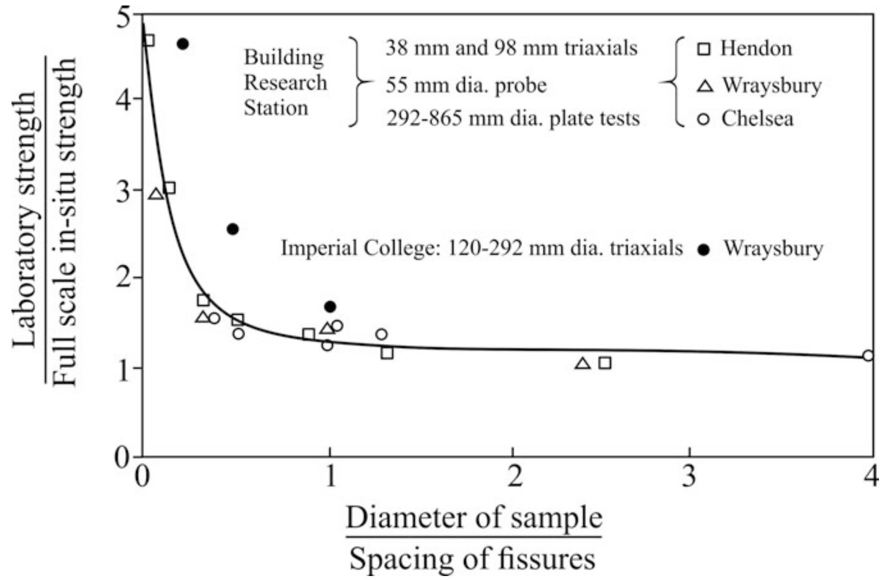


Field Testing, Fig. 3 Sample size depended changes in strength and deformation properties, (**a**) fractured and stratified limestone, upper Muschelkalk, (**b**) fractured limestone-mudstone sequence, Lias (Reik 1979)

Field Testing Methods

Over the years, considering the limitations and deficiencies of laboratory testing and empirical methods, geotechnical engineers have developed many *in-situ* based techniques to measure more accurately design engineering parameters of soils and rocks. Bell et al. (1990) specified that "*field testing and, to a lesser extent, geophysical surveying are major sources of both qualitative and quantitative data relating to the ground conditions.*" Although field testing methods are more expensive in comparison with laboratory testing and also it is not possible to control boundary conditions such as drainage conditions and confining pressure that can be easily provided in laboratory testing, there are several important advantages of field testing. These are (1) the effect of disturbance becomes insignificant or completely disappear in many *in-situ* tests, (2) the field based measurements provides more representative engineering parameters for design purposes (Bell et al. 1990), (3) test results can be obtained immediately, and (4) the performing tests on reasonably large size overcomes or reduces the scale effect related deficiencies. Considering other contributions such as "*Rock Field Tests*" and "*Soil Field Tests*" in this volume only the name and brief introduction of major field testing



Field Testing, Fig. 4 Spacing of fissures depended changes in measured shear strength of London clay (Marsland 1971)

methods utilized to obtain design engineering properties of soils and rocks are provided:

Penetration tests: Two different type of penetrations tests, called the standard penetration test (SPT) and cone penetration test (CPT), are widely used in geotechnical investigation of soft clay, silt and sand in which undisturbed sampling is very difficult (Bell et al. 1990). The determination of relative density, strength, bearing capacity and liquefaction potential of soils are generally provided by application of these *in-situ* tests.

Shear strength tests: The undrained shear strength of saturated cohesive soft soils with strength generally up to 200 kPa (Schnaid 2009) can be determined by a vane test. According to Schnaid (2009), this test can be also used for estimation of undrained shear strength of silt, organic peats, tailings and other geomaterials except for sand, gravel and other highly permeable soils. This test provides both peak and remoulded undrained shear strength values of soils. In addition to a vane test, a direct shear test is also employed for determining shear strength properties of geomaterials. This test can be used for measuring strength properties of both soils and rock masses, particularly those involving discontinuities.

Plate loading test: This *in-situ* test measures accurately the strength and deformation properties of soils and rocks on the ground. It may be concluded that the main purposes of this test are to determine bearing capacity of soils and rocks, the amount of settlement, the undrained shear strength of cohesive soils and the deformation modulus of weak rock masses. In addition to plate loading tests, there are other deformability tests such as "*plate test,*" "*large flat jack,*" "*radial jacking test,*" "*downhole plate test,*" "*downhole flexible dilatometer*" and "*downhole stiff dilatometer*" that are performed on the walls of underground openings (e.g, tunnel, gallery) and in the boreholes. These tests are mainly aimed to determine the *in-situ* stress-strain relationships of rock or rock masses.

Pressuremeter test: The pressuremeter is a cylindrical device developed by Louis Ménard in 1957 to measure the *in situ* strength and deformation properties of soils and weak rocks. To determine stress-strain relationships of these geomaterials, this cylindrical device applies uniform pressure to the wall of a borehole by means of a flexible membrane (Schnaid 2009). As specified by Bell et al. (1990), this *in-situ* test was successfully performed in identifying stress-strain relationships of soft geomaterials such as overconsolidated clay, mudrock and sand. Based on installation techniques, pressuremeters are generally classified as "*pre-bored (e.g., Ménard pressuremeter)*," "*self-boring*" and "*push-in*" pressuremeters (Schnaid 2009).

Dilatometer test: The dilatometer device is manufactured with a stainless steel blade with a circular, thin steel membrane mounted flat on one face (Schnaid 2009). This test can measure *in situ* stiffness, strength and stress history of the soil (Schnaid 2009). The engineering properties of rocks (Bell et al. 1990) and a wide variety of soils such as clay, sand, silt and hard formations (Schnaid 2009) can be determined by performing dilatometer test.

In-situ stress tests: The predetermination of both magnitude and directions of *in-situ* stresses is very important for stability evaluation of many engineering projects, particularly for underground openings. Therefore, in order to measure the required parameters of *in situ* stress, several methods have been developed. These techniques include "flat jack," "overcoring" and "hydraulic fracturing."

Permeability tests: The permeability characteristic of soils can be determined by changing the groundwater level in a borehole and then recording the flow rate at the steady state

(Bell et al. 1990). There are two types of permeability tests called a "constant head test" and "falling head test" based on this procedure. Whereas a constant head test is performed on soils having higher permeable features (e.g., coarse-grained soils), the falling head is accepted as the more appropriate test in fine-grained and low permeable soils. For hydraulic conductivity of fractured rock masses, the Lugeon test known also as "packer test" is used by recording flow rate after increasing and decreasing pressures in an isolated part of a borehole.

Geophysical methods: The geophysical methods involve measurements of some physical properties of soils, rocks or their masses. These properties are typically density, elasticity, electrical conductivity, magnetic susceptibility and gravitational attraction (Bell et al. 1990). The "*seismic,*" "*magnetic,*" "*resistivity,*" "*electromagnetic,*" "*gravity*" and "*downhole*" measurements are main geophysical techniques used in geological, hydrogeological and geotechnical studies.

Summary

The main purpose of site investigation in a complex project is to collect adequate geological and geotechnical data for determining engineering properties of geomaterials as well as *in-situ* stress conditions. The deformability, strength, settlement and hydromechanical characteristics are important engineering properties of soils and rocks that are used in the design phase of relevant project for mitigating geotechnical hazards. To determine engineering properties of soils and rocks, there are many laboratory testing methods in addition to empirical approaches. However, both of these naturally have some significant limitations in accurately measuring and predicting strength and deformability properties of soils and rocks. These limitations are mainly the effect of disturbance, changes in stress conditions, scale effect, changes in water content and assumptions if the sample is homogeneous and isotropic during modeling. Despite the fact that field testing methods are known as expensive and time-consuming techniques in geotechnical investigation, these methods are more appropriate approaches in obtaining accurately design engineering properties of soils and rocks. Considering the importance of *in-situ* tests in determination design engineering properties, brief introductions and aims of the most commonly used field testing methods are summarized above.

Cross-References

▶ Borehole Investigations
▶ Clay
▶ Cohesive Soils
▶ Collapsible Soils
▶ Compaction
▶ Compression
▶ Conductivity
▶ Cone Penetrometer
▶ Consolidation
▶ Deformation
▶ Desiccation
▶ Drilling
▶ Dynamic Compaction/Compression
▶ Effective Stress
▶ Engineering Properties
▶ Excavation
▶ Expansive Soils
▶ Exposure Logging
▶ Extensometer
▶ Facies
▶ Geophysical Methods
▶ Groundwater
▶ Hydraulic Fracturing
▶ Hydrology
▶ Inclinometer
▶ Instrumentation
▶ Jacking Test
▶ Mechanical Properties
▶ Modulus of Deformation
▶ Noncohesive Soils
▶ Piezometer
▶ Pore Pressure
▶ Rock Field Tests
▶ Rock Mechanics
▶ Shear Strength
▶ Shear Stress
▶ Site Investigation
▶ Soil Field Tests
▶ Soil Mechanics
▶ Soil Properties
▶ Strength
▶ Stress
▶ Tiltmeter
▶ Water Testing

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Filtration

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Definition

The use of filters to reduce the flow of fine grained soil particles into coarser soils.

Seepage erosion is reported in many different geological settings and in a variety of materials. In many construction activities, there are problems with the migration of fine soil particles into coarser soils. Particle migration is directly related to the groundwater flow conditions and, in particular, to high flow velocity. As a consequence, coarser materials can become clogged due to groundwater flow and particle migration and retention. In most problematic situations, the use of granular filters or packs or geosynthetic filters helps to decrease these problems, and improve the factor of safety in constructions, as well as reducing internal erosion, formation of underground cavities and clogging.

Many examples of ground surface, slope, and structural instabilities as a consequence of internal erosion by seeping water are known. The variety and complexity of the hydraulic processes involved in filtration problems are evidenced by the diversity of descriptive terms (Crosta and di Prisco 1999). Commonly adopted terms are piping or soil piping (Terzaghi and Peck 1967; Skempton and Brogan 1994; Jones 1990; Dunne 1990; Hagerty 1991; Koenders and Selimeyer 1992; Worman 1993), sapping (Higgins 1984), and subsurface or seepage erosion (Hutchinson 1982). Seepage erosion seems the most appropriate to describe internal erosion.

Internal erosion can originate by two main processes. In granular materials, it can result from the dragging induced by seeping water; in cohesive materials and unsaturated granular soils, it can derive from the enlargement of macropores because of the shear stresses applied to their perimeters. Seepage erosion involves the displacement of soil or rock particles resulting from water flowing through and emerging from a porous medium. In this case, the mobilization may occur by flow, if the fluid strongly affects the interparticle frictional strength, or by the shear failure of the continuum. Seepage erosion can advance easily in soils with macropores along which water flow occurs and where a conspicuous water supply (intermittent or continuous) and well-concentrated water infiltration takes place. Tunnel scour occurs when water enters a hole or crack (e.g., by biogenic factors or tensile cracking) within or at the surface of a soil deposit.

To limit filtration problems granular filters or granular pack are frequently used. Filters allow water to flow while protecting the solid particles from being transported. In groundwater wells, gravel packs are required to facilitate water seepage from the aquifer through the well screen without displacing soil particles. In these cases, forced water flow conditions are used to generate a well-performing protective granular filter. Filters are fundamental in Earth dams and embankments construction, both to prevent particles migration and internal erosion and to avoid subsidence or collapse. In geotechnical structures, buildings and pavements, groundwater, and infiltration water are required to be drained to avoid destabilizing seepage pressures, wetting and flooding of surfaces (e.g., pavement, road surface). Different permeability and retention criteria have been proposed in the literature and should be adopted for the correct design of granular or geosynthetic (geotextiles) filters. Geotextile designing (Koerner 2013) for filtration problems shares much with graded granular filter design (i.e., retention, permeability criteria) and considers also clogging resistance and survivability criteria. Geotextiles are frequently favored because of the initial cost savings.

Cross-References

► Aquifer
► Clay
► Conductivity
► Contamination
► Dewatering

F

► Dissolution
► Erosion
► Factor of safety
► Fluid Withdrawal
► Geotextiles
► Groundwater
► Hydraulic Action
► Infiltration
► Liners
► Percolation
► Voids

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Floods

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Definition

Flooding is a natural process that occurs when the level of a body of water rises until it overflows its natural banks or artificial levees and submerges areas that were usually dry. Along a watercourse, a flood can manifest itself annually. Usually high water flow is contained between the natural banks or artificial levees, but when the volume of the flood waters can no longer be contained within those natural or artificial confines, waters expand into the surrounding areas. The flood extent follows a dynamic propagation that depends essentially on the amount of water that overflows, the speed of the water flow, and the morphology of the surrounding areas (Fig. 1).

Introduction

Precipitation events have a fundamental role in the formation of a great number of exogenous natural processes. Their interaction can promote the formation of landslides, mud-debris flows, avalanches, and floods. Undoubtedly, floods impact the largest number of people, as a consequence of involving large areas that are often densely populated.

Water has always played a vital role in the life of people. From the beginning of civilization, people have tended to live near the water, along creeks, streams, and rivers or along lakes and sea coasts. Land close to water has usually offered many advantages to settlers, initially for basic survival and then facilitating societies, development, and industrialization. The water's presence was important not only for having a fertile soil to grow food but also for transportation. But now flooding produces damaging events which affect approximately 21 million people worldwide on an annual basis (World Resources Institute 2016).

Cause of Flooding

The Role of Rainfall

Flood events are usually preceded by heavy rain: they can have different developing time and intensity. Rainfall of short duration and high intensity can cause easier flooding in small mountainous streams/creeks whereas rainfall of prolonged time and low intensity can provoke large floods mainly in larger basins on the plains. In fact, a precipitation widely distributed over an ample basin can create problems along the entire hydrographic network. All streams become swollen and when flowing into the main river they contribute to the formation of an extraordinary flood.

Other Causes

Flood events are not limited only to rainfall from storm events. They can happen due to:

1. Rapid melting of snow and/or ice masses by an abrupt rise in temperature. The eruptions of the volcano Eyjafjallajökull (Iceland) on March 2010 caused melting of its glacier. A flow meter device in the Krossá glacial river recorded a sudden rise in water level and in water

Floods, Fig. 1 Large area flooded through a break produced by collapsing of the levee embankment: Kinugawa River in Joso, Ibaraki Prefecture on 10 September 2015 (From: http://mashable.com/2015/09/10/japan-flooding-photos/#fKrjhULU08qi – The Yomiuri Shimbun/Associated Press)

Floods, Fig. 2 (**a**, **b**) Ceva, small town in Piedmont (Northwestern Italy). Bridge before and after the peak of the Tanaro flood occurred on November 1994. During the process the river swept everything that was on its way: not only trees and shrubs from the banks but also cars, dustbins, and tons of rubbish (Photos of the author)

temperature. About 1000 people from the zones of Fljótshlíð, Eyjafjöll, and Landeyjar were quickly evacuated (Smith 2013). However, heavy snowfall followed by melting due to a sudden rise in temperature sometimes causes extensive flooding in temperate areas.

2. Sudden emptying of a glacial cavity, like the case of the outburst flood from Glacier de Tête Rousse that occurred in 1892. The rupture of an subglacial cavity in Glacier de Tête Rousse released 200,000 m^3 of water and ice: the village of Saint-Gervais-Le Fayet (French Alps) suffered 175 fatalities (Vincent et al. 2010).
3. Accidental blockage or the flow along the bed of a watercourse or at its mouth. The obstruction can happen for a landslide fall (very common), for a bridge collapse, for floating materials jammed against a transversal infrastructure (Fig. 2), for sediment bed load, etc. The water

Floods, Fig. 3 Unbelievable frame depicting the wave of the tsunami that struck on 26 December 2004 in the village of Ao Nang (Thailand) (Courtesy of D. Rydevik, email: david.rydevik@gmail.com, Stockholm (Sweden))

usually overflows in the lateral areas and upstream of the blockage. In 1961, for example, the riverbed of the Wei River (the Yellow River's largest tributary) was blocked by 1.5 billion tons of sand, and its bed was lifted by 40 m. A large area was inundated, and almost half a million local people were forced to move.

4. Sudden release of water from natural or artificial reservoirs due to natural (*a*, *b*) or anthropic causes (*c*).
   (a) On October 1963, in Northeastern Italy, the Vajont landslide (>230 million cubic meters) caused a man-made tsunami in an artificial basin. Fifty million cubic meters of water overtopped the dam with a 250-m wave: several little towns were completely destroyed with 1917 casualties (Semenza and Ghirotti 2000).
   (b) Earthquake-induced movement of the subsoil such as like the disaster of Baldwin Hills Dam (Los Angeles). On December 1963, the collapse of the dam released 950,000 m^3, resulting in five deaths and the destruction of 277 homes (Anderson 1964).
   (c) In December 1959, the Malpasset Dam failed due to mistakes in the planning stage. The huge water outburst caused 423 deaths with 83 injured, 155 buildings destroyed, 796 damaged, and 1350 hectares wrecked. The worst effects were felt in the valley downstream, in particular in the town of Fréjus (French Riviera), located eight kilometers from the dam (Habib 1987).
5. Water surges at the seashore as a result of (a) storms, (b) earthquake, and (c) submarine landslide.
   (a) Flooding of the North Sea hit the Netherlands, Belgium, England, and Scotland on the night of 31 January–1 February 1953. The flooding was caused by the combination of a high spring tide with a severe cyclone over the North Sea. In some areas the sea level rose by more than 5.50 m above the mean value, overwhelming the sea defenses and causing extensive flooding (more than 2300 victims) (Baxter 2005).
   (b) In Southeastern Asia, in 26 December 2004, an earthquake occurred with an epicenter off the west coast of Sumatra (Indonesia). The earthquake, with a magnitude of 9.1–9.3 on the Richter scale, triggered a series of devastating tsunamis along the coasts with waves up to 30 m (Fig. 3): 230–280,000 people in 14 countries died (Kelman et al. 2008).
   (c) A mega-tsunami occurred on 9 July 1958 at Lituya Bay (Alaska): it was caused by a gigantic landslide of ice, debris and rock; about 30 million cubic meters of rock fell into the sea generating the highest wave ever recorded, with a height of more than 500 m. The wave swept 11 km to the mouth of the bay at a speed between 150 and 210 km/h. The surge and wave of water destroyed the forest on the shores over an area of 10 km^2 (Miller 1960).
6. Military attack. Floods can also occur caused by humans. In 1943 the British bombed three artificial dams in Germany to weaken the Ruhr, the largest industrial region of Nazi Germany. The disaster cost 1200 human lives and led to the destruction of the downstream settlement. In 1944 the Germans tried to slow down the Allied troops by flooding large areas using a tactic of war frequently utilized by the military in the past (Rettemeier et al. 2001).

Flooding Frequency

Not all watercourses experience inundation with the same frequency. This is influenced by the climate of the area and by the condition of the basin (bank stability, riverbed cleaning, presence of infrastructures, stability of the slopes). To assess their frequency, hydrologists use the term "return period," which is the time in which an intensity value assigned is equivalent to, or is exceeded on average at least once. For convenient representation, the return time is often used in place of the concept of probability of not exceeding associated with a certain natural event. In other words, the probability that a flood discharge can occur with an intensity is greater than or equal to a pre-determined one. It is important to emphasize that, when a severe flood is defined as a "100-year flood," it refers to an event of a magnitude corresponding to an average annual probability of 1%. This statement does not mean that there will be a flood of that magnitude every 100 years. While the flood-frequency approach does not provide a deterministic assessment of the risk, it is useful for the purposes of flood risk management or the likelihood of the occurrence of any given damage in a given time interval.

Flood Measurements

People who have suffered the terrible experience of a flood are usually astonished at what a river can cause. Geologists and engineers rather tend to see the phenomenon as a cyclic event of natural instability correlating the causes of initiation and studying the most important effects and consequences.

Hydrologists compare the flood with those that occurred in the past, whose measurements have been gathered and can constitute an important database. Generally, the flood is classified depending on its "flow," that is, the liquid volume that passes through a unit of time a section of a waterway or channel. It is measured in m^3/sec or in ft^3/sec. The presence on a bridge or along a bank of a measuring instrument (hydrometer/hydrometrograph) may allow measurement and recording of the quantity of water discharged in real time. On most occasions, such sophisticated devices along the river are absent, or, if present, they were removed by the high flow. As a result, scholars must rely on indirect methods that enable one to estimate the extent of the flow in the aftermath. Where high-water marks of the flood are still present, the width of a peak flow water surface can be measured knowing certain factors such as the slope of the riverbed, its geometry and roughness. Similarly, in the sections of waterway where the outflow scales are known, it is possible to infer the rate of the flow from the hydrometric levels measured or estimated. All indirect methods must be calibrated and updated over time.

Observations and Controls

Before the Flood

Since flood events are generated predominantly by precipitation, initial data to be quickly obtained is the amount and the duration of rainfall. Measurements can be made through automatic devices permitting continuous recording or simply using special containers of standardized capacity. In some localized situations, such as streams fed by natural lakes, it helps to know in advance the size of the hydrometric increases within the reservoirs.

During the Flood

The best way to follow the evolution of a flood along a watercourse is based on the constant control of the water levels, in order to identify a threshold of height limit (warning level) above which overflows and flooding may occur. These observations can be made using automatic transceiver equipment or in faster way, collecting during the event periodic readings of the level reached by the flood, corresponding to a grade rod or other reference points.

Especially in the rising phase of the flood, it is essential to record data concerning the ascent rate of the water levels (cm/hour) and the degree of turbidity of the water. The latter can be evaluated by measuring the concentration of suspended material in water samples collected at regular time intervals with bottles of appropriate capacity containing an amount not less than one liter.

Repeated visits to the more vulnerable streambanks permit identification of the intensity of erosion of the banks by the amount of land progressively removed. Along embanked rivers it is necessary to check the levee embankments both from the inside (the river) and from the outside (to the country) to recognize the early clues of embankment instability.

Along the floodplains of the secondary river system that flows into the main stream, one must follow the trend of outflows to detect the possible slowdown of water flow or end of the flooding.

After the Flood

At the end phases of withdrawal and lowering of the level of floodwaters, it is very useful to record all the consequences resulting from the dynamics of the phenomenon. Within the riverbed, it is particularly useful to verify:

- The major aggradation of alluvial deposits
- The points with greater concentration of erosive processes with particular attention to those located in proximity to structures with potential for exposing their foundation

Finally, with regard to the outside of the river-bed, the time period of water remaining in any submerged area should be recorded, data collected on the nature and thickness of the deposited material, and markers placed or any stable structure reference points noted to indicate the maximum level reached by flood waters.

Floods, Fig. 4 Aerial view of Passau (about 200 km northeast of Munich, Germany), an important town flooded by the Danube River on 3 June 2013. Following heavy rain and thaw, the Inn and Donau rivers are expected to rise to over 11 m (REUTERS/photo Michaela Rehle)

Flood-Prone Areas

Floodplains are the areas most prone to flooding although alluvial fans and all coastlines are also susceptible to a lesser degree. The identification of areas that are potentially subject to flooding has been one of the most frequent debates among the scientific community in recent decades. The delimitation of these critical areas has been requested by local citizens, industries, and organizations as well as state and regional bodies responsible for disaster prevention (Luino et al. 2012). In the last 25 years, insurance companies have shown interest in this field, and various papers and reports have been published. It is important to be able to estimate the likelihood and the social, economic, and environmental consequences of a flood disaster (Fig. 4).

The identification of flood-prone areas can be approached by different methods. Some authors have used specific criteria, including historical, geomorphological, hydrologic-hydraulic, and remote-sensing methods. Other authors have combined methods, which can yield better results because such an approach can compensate for the limitations of individual methods. Notable results have been achieved, for example, by combining historical and geomorphological methods or geomorphological and hydrological methods or by an approach based on historical-hydro-geomorphological reconstitution and hydrological-hydraulic modeling.

In recent decades, GIS (geographical information systems) and LiDAR (laser imaging detection and ranging) have been important tools in spatial processing. GIS uses a series of software tools to capture, store, extract, transform, and display real-world spatial data, whereas LiDAR is an optical remote-sensing technology for creating high-resolution digital models of the earth's surface. LiDAR is particularly useful because the ongoing construction of levees, dikes, roads, railway embankments, and buildings constantly changes the land's appearance. Regardless of the method, the quality of the results obtained is always dependent on the assessment of the natural world. Only after a flood can a model be recalibrated and inadequacies improved.

Conclusions

Floods are one of the more significant natural processes affecting populations. In the twentieth century, eight great floods in China (the most harmful in history) killed more than seven million inhabitants. Every year, in fact, there are numerous floods in populated areas all over the world: the waters flood the land and destroy crops, facilities, and infrastructure, often causing the death of thousands of people. More people are affected by floods than by any other type of natural disaster. More than 20 million people worldwide are affected by river floods each year on average and that number could increase to 54 million in 2030 due to climate change and socioeconomic development (World Resources Institute 2016).

Better land-use planning and flood risk reduction especially in heavily populated areas will have to take into consideration most of the following aspects:

1. Accurate definition of river corridors based on the basin's historical, geomorphological, and hydraulic characteristics.
2. Redesign of the area's principal man-made structures, after evaluation of their interactions with the river dynamics.
3. Review of local planning procedures based on current knowledge with recognition of zones at different degrees of risk where different rules will have to be applied.
4. Relocation of urbanized areas. Rather than spend millions to "secure" high-risk areas, an economically wiser choice would be to build new houses, industrial sheds, schools, and other constructions in low-risk areas where inhabitants and business could safely relocate. This eliminates the need to compensate a certain percentage of damage and rebuilding within an area that is likely to be destroyed or flooded again 5–20 years in the future.
5. Introduction of compulsory insurance coverage. Taking the example from countries where insurance coverage has long been a regular procedure, this may be an effective tool to favorably influence urban development of already heavily populated areas. When combined with a flow of information to citizens and local communities, this measure would spare the government the relief expenses usually spent on helping communities in the wake of natural disasters.
6. Only with a farsighted view that embraces the watercourse as a whole system, including large works upstream with flood control reservoirs, or local works like floodway channels, bridges without piers in the riverbed, and regular removal of natural vegetation from the riverbed can the damage of future floods be limited.

Cross-References

▶ Aerial Photography
▶ Armor Stone
▶ Beach Replenishment
▶ Catchment
▶ Climate Change
▶ Coast Defenses
▶ Coastal Environments
▶ Cofferdam
▶ Current Action
▶ Dams
▶ Earthquake
▶ Engineering Geomorphological Mapping
▶ Engineering Geomorphology
▶ Environments
▶ Erosion
▶ Factor of Safety
▶ Fluvial Environments
▶ Gabions
▶ Geohazards
▶ Glacier Environments
▶ Hazard Assessment
▶ Hazard Mapping
▶ Hydraulic Action
▶ Hydrogeology
▶ Hydrology
▶ Infrastructure
▶ Instrumentation
▶ International Association of Hydrogeologists (IAH)
▶ Landslide
▶ Levees
▶ Lidar
▶ Marine Environments
▶ Monitoring
▶ Nearshore Structures
▶ Probabilistic Hazard Assessment
▶ Reservoirs
▶ Risk Assessment
▶ Risk Mapping
▶ Run-Off
▶ Saturation
▶ Sea Level
▶ Tsunamis
▶ Water

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Fluid Withdrawal

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Synonyms

Gas extraction; Groundwater extraction; Oil extraction

Definition

Fluid withdrawal involves the extraction of groundwater, thermal water, oil, or gas from the subsurface. Depth of extraction may range from few meters (e.g., water table aquifers) to several kilometers (oil/gas reservoirs). Time period of extraction is generally on the order of several years or even decades.

Introduction

One major environmental, social, and economic consequence of groundwater pumping and gas/oil/geothermal water production is anthropogenic land subsidence. To be of major concern, subsurface fluid withdrawal must occur in densely populated or highly developed areas located close to the sea, or a lagoon, or a delta and take place within unconsolidated geological basins of alluvial, lacustrine, or shallow marine origin, formed typically, although not exclusively, during the Quaternary period. The first observation concerning land subsidence due to fluid removal was made almost one century ago by the American geologists Pratt and Johnson (1926) who wrote that *"the cause of the subsidence is to be found in the extensive extraction of oil, water, gas, and sand from beneath the affected area"* which was located above the oil field of Goose Creek, S. Jacinto Bay, Texas, USA.

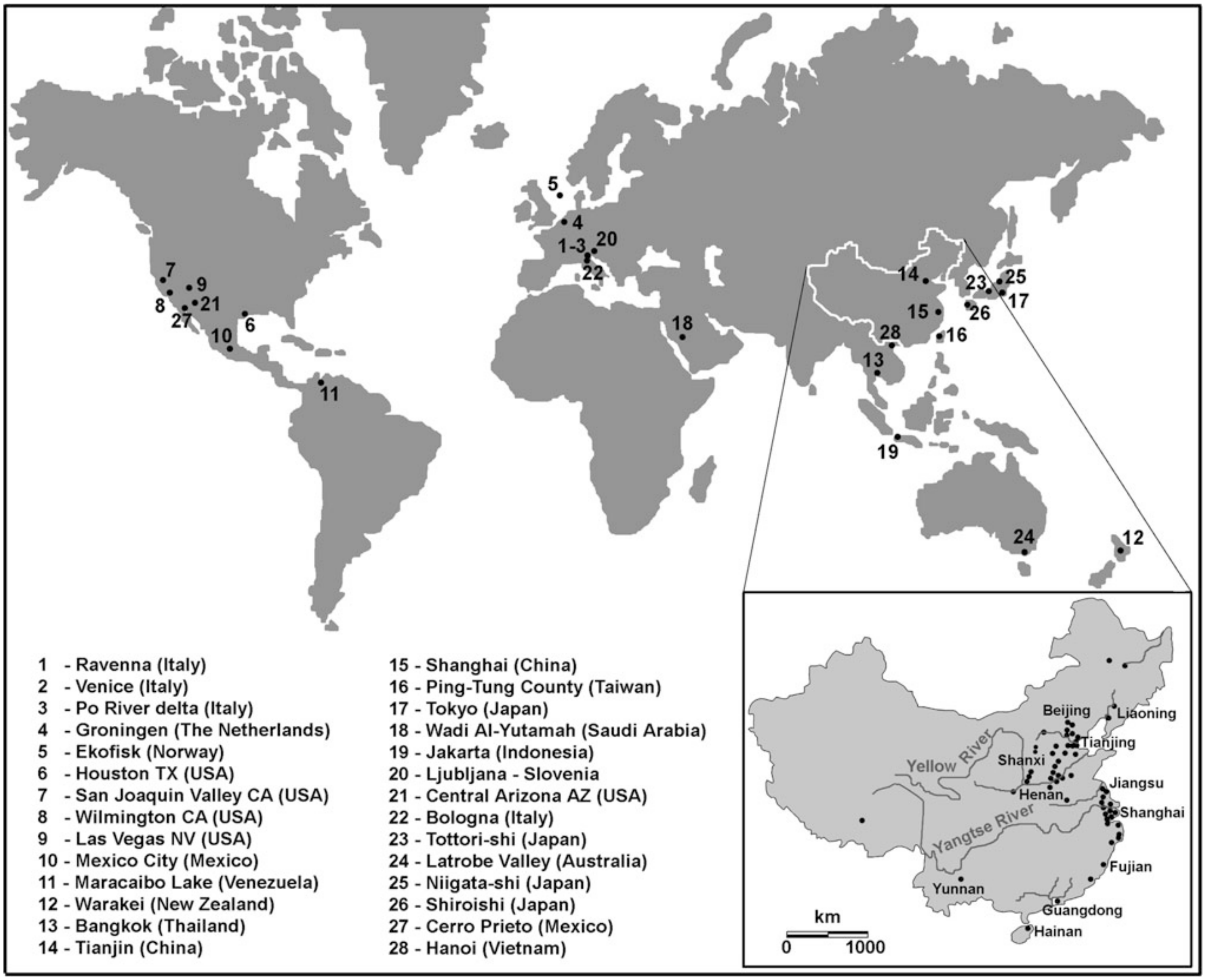
Occurrence

The early conjecture of Pratt and Johnson (1926) was later confirmed and reconfirmed by countless examples of anthropogenic land subsidence, and especially so in the second half of the last century. Figure 1 shows the areas of major anthropogenic land subsidence worldwide. The maximum recorded settlement amounts to as much as 14 m and 13 m due to geothermal water production at Wairakei, New Zealand, and groundwater pumping in Mexico City, respectively. However, settlement over the range 5–10 m is no exception (e.g., S. Joaquin Valley, CA, USA, 10 m; Wilmington oil field, CA, USA, 9 m; Ekofisk oil field, North Sea, 6.7 m). The depth of pumping wells may range from those tapping very shallow water table aquifers just close to the ground surface to those tapping very deep (4000–5000 m) gas/oil reservoirs. In the case of complex aquifer systems, the overall extent of the sinking area can be on the order of several thousands of km^2. China is perhaps the country where anthropogenic land subsidence due primarily to subsurface water overdraft has occurred in the largest number of settling areas with a cumulative size close to 80,000 km^2 (see the inset of Fig. 1 for the major sinking areas and cities in China).

Mechanism

The mechanism by which rocks deform and compact under the influence of fluid extraction, and hence a fluid head change, is well understood. The total geostatic or overburden load σ_t, which is the weight of the overlying material column acting upon the aquifer or reservoir and confining beds, is balanced by the fluid pore pressure p and the effective vertical and horizontal inter-granular stresses σ_{ev} and σ_{eh}, which are the stresses exchanged between the rock grains in contact (Fig. 2). The introduction of a pumping well into a natural fluid flow system produces a disturbance that yields an abatement of the fluid pore pressure with a stress transfer from the fluid phase to the solid phase, and a consequent increase in effective stress under whose influence the pumped and neighboring formations compact, with the amount of compaction primarily related to the compressibility of the drained layers and the fluid pore pressure decline. The resulting cumulative compaction of subsurface layers extends its effect to the ground surface, which therefore subsides.

Most freshwater aquifer systems are normally consolidated and pressurized, or only slightly over-pressurized, and may lack important faults given the typical formation mechanism involving a depositional alluvial/marine environment that takes place without significant tectonic interference. However, their geomechanical simplicity may be partly offset by a litho-stratigraphic complexity related to the actual distribution of clay, silt, and sand-rich soils within the compacting



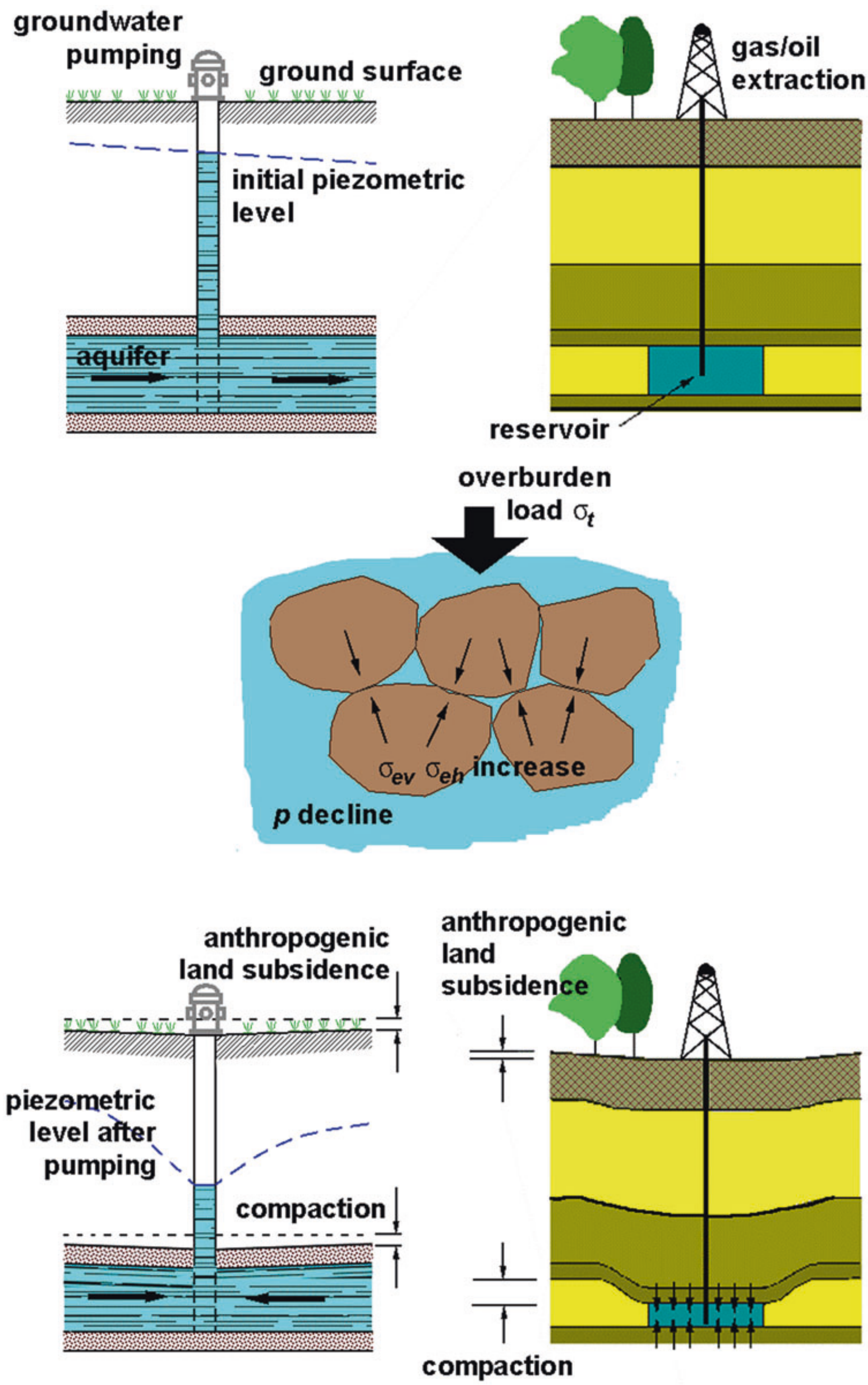
Fluid Withdrawal, Fig. 1 Major worldwide areas of anthropogenic land subsidence due to subsurface fluid removal. The inset shows the major sinking areas and cities in China (after Gambolati 2014)

systems. As a result, land subsidence of a freshwater aquifer system depends to a large extent on the clay and silt fraction within the system, formed by confining beds, intervening aquitards, and interbedded clay/silt lenses. Moreover, drainage from these beds can lag behind drainage from the producing sand, thus causing a delayed land subsidence which may partly manifest itself after wells shut down.

Controlling Factors

Four factors may partially combine to produce measurable settlement records: (1) shallow burial depth of the depleted layers; (2) highly compressible deposits in alluvial or shallow marine environments; (3) large pore pressure decline; and (4) large thickness of the depressurized fluid bearing sediments. Unless the aquifers or gas/oil fields are overpressurized, factors 1 and 3 are mutually exclusive, whereas they can be both associated with factors 2 and 4. For a large subsidence to occur, however, a soft compacting stratum is needed and/or a large pressure drawdown.

Some reservoirs and aquifers may be overconsolidated. Overconsolidation tends to reduce the early subsidence rate with a sudden unexpected growth at some stage of production when the *in situ* stress exceeds the preconsolidation stress. A preconsolidation effect may be caused in the geological past by uplift followed by erosion of the sediments overlying the fluid bearing layers or by fluid overpressure or both. The aforementioned processes may have led to a reservoir/aquifer system expansion which was much smaller than the original mostly unrecoverable compaction. When pore pressure drops due to fluid production, a reloading of the depleted formations



Fluid Withdrawal, Fig. 2 Mechanism of anthropogenic land subsidence due to groundwater and gas/oil withdrawal (after Gambolati et al. 2005)

takes place. Initially compaction, and hence land settlement, are small. However, as soon as the maximum experienced load is overcome, rock compression occurs on the virgin loading curve with a sudden increase of compressibility, and the resulting subsidence rate.

Environmental Impact, Ground Ruptures, and Induced Seismicity

Some major impacts of anthropogenic land subsidence include: (1) increased flood risk (frequency, depth and duration of flooding events) and more frequent inundation induced by rainfall because of the reduced effectiveness of the drainage systems; (2) damage to buildings, foundations, infrastructures (roads, bridges, dikes), and underground structures (drainage, sewerage, pipes); and (3) disruption of water management and related effects (change of gradient of streams, canals, drains, increase salt water intrusion, increased pump power).

Anthropogenic land subsidence causes direct and indirect damage. Direct damage includes the loss of functionality and/or integrity of structures such as buildings, roads, subways, and underground utility networks (infrastructures). The most common indirect effects are related to changes in both surface and subsurface relative water levels. The estimation of the associated cost is quite complex. In practice operational and maintenance costs are addressed in several short and long-term policies and budgeting.

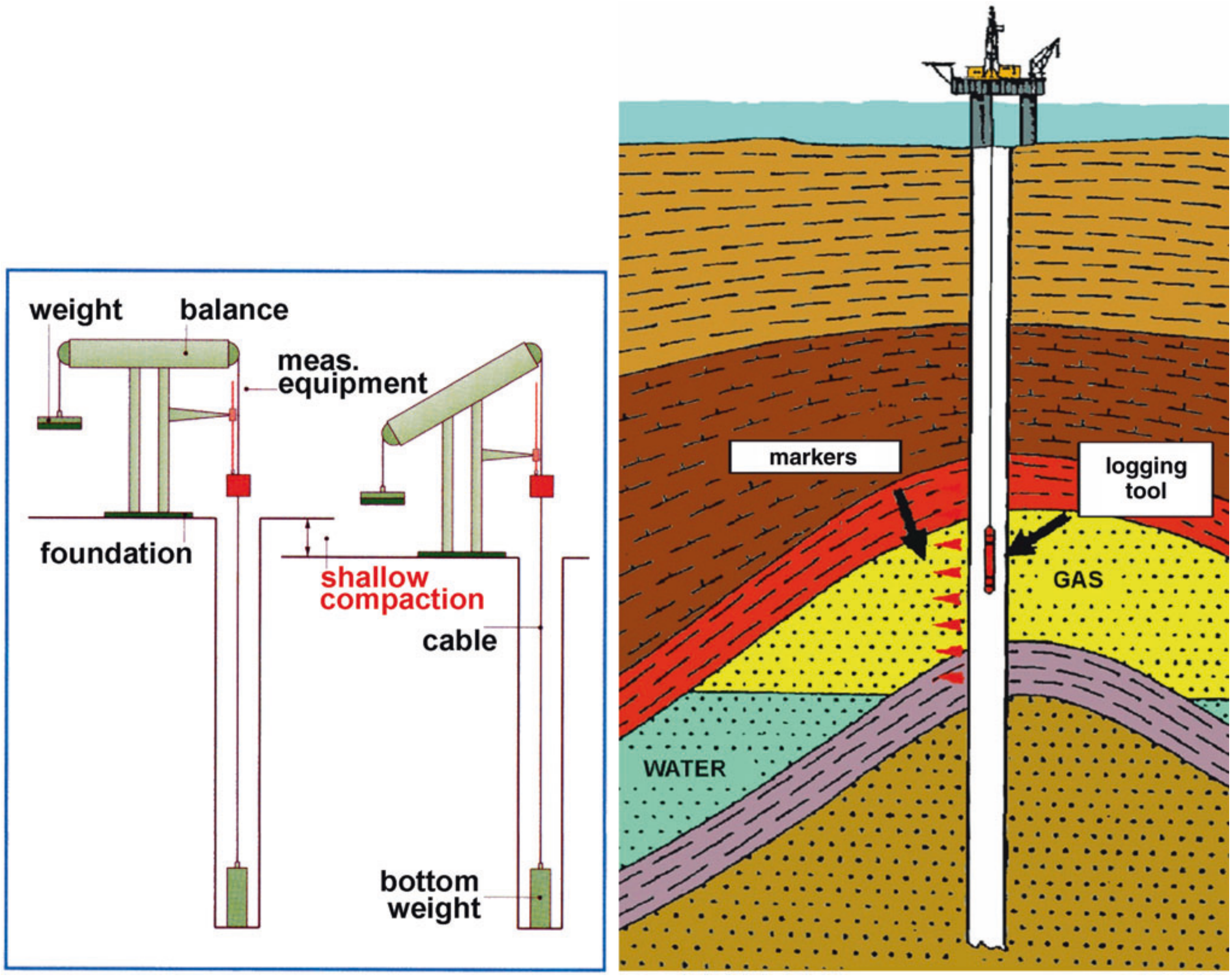
For instance, in China the average total economic loss due to anthropogenic land subsidence is estimated at 1.5 billion dollars per year, 80–90% of which are indirect costs. In Shanghai, over the decade 2001–2010, the total cumulative loss approached two billion dollars. In Bangkok many private and public buildings, roads, pavements, levees, and subsurface structures (sewage, drains) have been severely damaged by land subsidence although reliable estimates of costs are not available. The total cost of damage related to subsidence in The Netherlands was estimated at over 3.5 billion euro per year.

Ground ruptures associated with land subsidence caused by groundwater withdrawal have been reported from many alluvial basins in semiarid and arid regions including southwest USA, central Mexico, Iran, Saudi Arabia, and China. Density, shape, length, aperture, depth, and dislocation of fissures vary greatly from site to site and are mainly related to subsurface litho-stratigraphic variations. Vertically dislocated fissures >2 m have been recorded, up to 15 km long, 1–2 m wide and 15–20 m deep. Considerable economic, social, and environmental damage is reported: in particular rupture of borehole casings, pipes, and canals used for conveying water, oil, and gas both in rural and urban areas. Other consequences include reduction of potable water supply, cost increase in groundwater extraction, and damage to surface structures and infrastructures, thus aggravating the damage caused by anthropogenic land subsidence.

A factor that may influence the occurrence and impact on the environment is the presence of faults within the developed system and the overburden, as in the case, for instance, of Las Vegas, NE, USA. The activation of thrusts/faults caused by fluid withdrawal (as well as by fluid injection) may pose a very serious risk of anthropogenic seismicity. According to Ellsworth (2013), the mechanism responsible for inducing seismicity "*appears to be the well-understood process of weakening a pre-existing fault*" by changing the fault loading conditions. In essence "*increasing the shear stress, reducing the normal stress and/or elevating the pore pressure can bring the fault to failure triggering the nucleation of an earthquake.*" The number of earthquakes with magnitude $M \geq 3$ recorded yearly in the USA mid-continent has grown significantly since 2001 (Ellsworth 2013) with anthropogenic earthquakes suspected to be partially responsible for the increase. For a review of the above issues see Gambolati and Teatini (2015).

Measuring and Monitoring Rock Compaction and Land Subsidence

The analysis and the prediction of the expected anthropogenic land subsidence due to fluid pumping requires a careful reconnaissance study of the area of interest with the detailed recognition of the basin geology and geometry and the reconstruction of its past history. Geomechanical and hydraulic properties are of the utmost importance. Preconsolidation stress, zones of overpressure, and faults/thrusts with their extent and orientation and geomechanical properties (i.e., friction angle and cohesion) must all be reliably identified. Advanced technology (2-D and 3-D seismic surveys, airborne-electromagnetic investigations, well-logs, exploration boreholes, *in situ* geophysical measurements, pumping tests, laboratory analyses) can be of great help. Much progress has also been made from the traditional spirit levelling in accurately recording and monitoring the ground surface movements. New techniques include DGPS (Differential Global Positioning System) and InSAR (Interferometric Synthetic Aperture Radar) by which land subsidence is measured from space with a very high precision. Advances have also been accomplished in measuring aquifer system and reservoir compaction. The main techniques currently available to measure the *in situ* deformation of depleted formations are borehole extensometers for shallow aquifer-aquitard systems (Fig. 3, left) and radioactive markers for deep gas/oil reservoirs (Fig. 3, right). A typical extensometer tool consists of a balance beam carrying a cable or a pipe which is fastened at one end to an anchor weight at the bottom of the compacting system, and at the other end to a counterweight so as to keep the cable at a constant tension. A computer-controlled system records the compaction data vs time. The instrumental precision is very much dependent on the actual extensometer implementation, but a nominal strain resolution of 0.01–0.1 mm can be achieved over depths of 200–1000 m. The radioactive marker technique is based on the regular monitoring of the distances between a number of low-emission isotopes (usually ^{137}Cs or ^{60}Co), incorporated into bullet-shaped leak-proof steel containers and shot at fixed intervals along the wall of a generally nonproductive vertical well. The location of each bullet after the porous medium has deformed is obtained by drawing a specific tool provided with gamma-ray detectors up through the well from the bottom. Thin gas/oil reservoirs mainly compact in the



Fluid Withdrawal, Fig. 3 Schematic representation of a borehole extensometer (left) and radioactive markers (right) recording shallow and deep compaction, respectively (after Gambolati et al. 2005)

vertical direction with a negligible lateral deformation, so that the uniaxial compressibility c_M of the medium comprised between two consecutive markers can be readily derived as:

$$c_M = \frac{\Delta h}{h_0 \Delta p}$$

where Δh is the measured deformation, h_0 the initial spacing of two neighboring markers, and Δp the pore pressure variation provided by the available logging tools. InSAR measurements and the related processing methodologies allow for the detection of subcentimeter scale ground movement in both vertical and horizontal dimension with high spatial detail and measurement resolution. SAR processing chains can provide millions of data points over a large region (10^4–10^5 km^2) and are often less expensive than sparse point measurements from “traditional” labor intensive spirit leveling and costly GPS-stations. For these reasons, leveling and GPS are being less and less used to measure land subsidence.

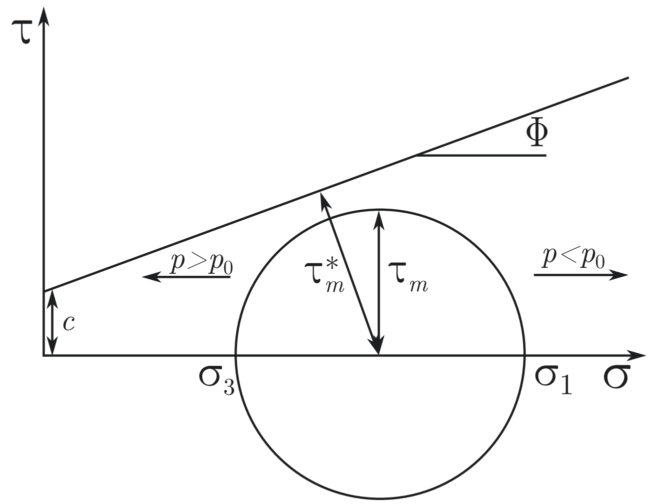
Modeling and Predicting Anthropogenic Land Subsidence

Anthropogenic land subsidence modeling and forecasting tools are continuously being improved (Gambolati and Teatini 2015). They take advantage of both the enhanced computer facilities (e.g., parallel hardware) and the advanced measurements technology of horizontal and vertical ground movements (e.g., DGPS and InSAR technologies). Once the models (that might include the evaluation of a possible seismic risk) have been calibrated with the observed history of the aquifer/hydrocarbon field, they can be used in a predictive capacity under different scenarios of groundwater use or field development thus helping develop integrated resource management that hopefully includes environmental and socioeconomic impacts and provides a “*sustainable*” development.

The latter depends on the vulnerability of the area where the aquifer/field is located, and particularly so in coastal land.

Three basic major steps can be envisaged in a control program to be set up in advance of the withdrawal inception:

1. Prediction of the expected land settlement in the area to be accomplished with the aid of state of the art models and all the available geological, geomechanical, and fluidynamic information.
2. Continuous monitoring and measuring of the occurrence, with the aid of spirit levelling, DGPS, InSAR, and markers technique for land movement and the compaction of the depleted gas/oil formations. The behavior of the shallow fresh water multi-aquifer system should be checked with the aid of extensometers. A network for measuring the triggered micro-seismicity should also be installed.
3. Prevention of the expected anthropogenic land subsidence or mitigation of the settlement experienced during aquifer/field development by sensitive spots in the area involved (if larger than the sustainable value). The last goal can be fulfilled by contrasting the pore pressure decline directly within the depleted gas/oil formations and the pumped aquifer system by a pressure maintenance program and an artificial recharge program, respectively.



Fluid Withdrawal, Fig. 4 Mohr–Coulomb's circle. When the pore pressure p increases ($p > p_0$) because of fluid injection, the circles move left and may achieve the limiting yield surface or friction line $\tau = c + \sigma_n \tan\Phi$ where σ and τ are the normal and shear stress, respectively, c is the cohesion and Φ is the friction angle. τ_m and τ_m* are the current largest and maximum allowable shear stress, respectively, σ_1 and σ_3 are the maximum and minimum principal stress, respectively

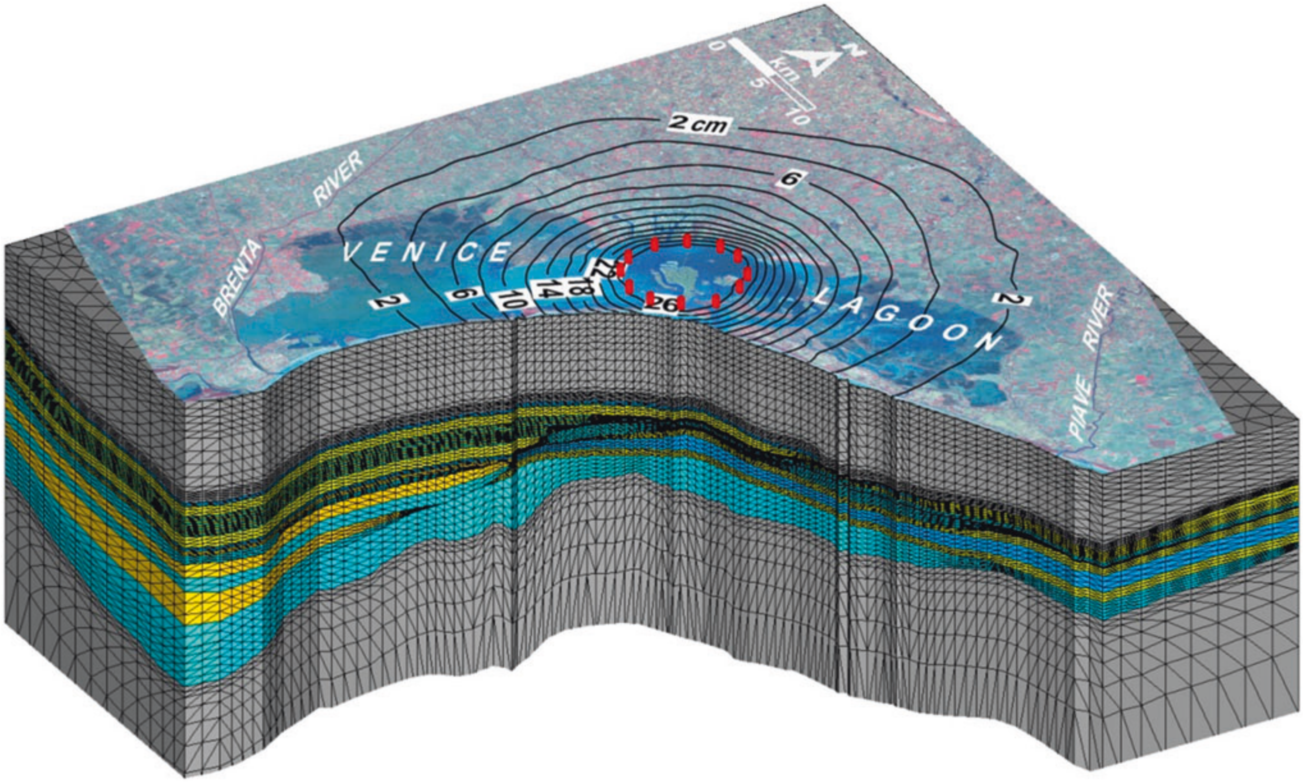
For a discussion of the possible procedure leading to a decision that integrates the technical, social, economic, legal, and political conflicts arising from land subsidence see Freeze (2000).

Land Uplift Due to Fluid Injection

Whereas anthropogenic land subsidence is a well-known process, the reverse, namely, land uplift, due to underground fluid injection, is a much less observed and recognized event although the practice of injecting fluids underground is more than a half century old. Land motion related to subsurface fluid injection went unnoticed for a long time in the vast majority of cases. Only in recent times has satellite technology offered a relatively inexpensive, spatially distributed, and accurate methodology to detect ground movement practically worldwide, revealing anthropogenic uplift of some interest in terms of magnitude, size of the area involved, and time of occurrence. An example of anthropogenic land uplift is offered by the injection of treated seawater at a depth of 50′ to mitigate land settlement over the oil field of Long Beach, CA, which resulted in a ground rebound of about 30 cm (Rintoul 1981).

The pore pressure increase in the injected formation may induce new geomechanical processes which are not encountered when fluid is withdrawn. These mechanisms can be more easily understood with the aid of the schematic Mohr-Coulomb representation of the stress state as shown in Fig. 4 where compressive stresses are taken with the positive sign. When fluid is removed, the pore pressure p decreases with respect to the original value ($p < p_0$) and the effective stress σ increases. Hence, Mohr-Coulomb's circle of Fig. 4 moves to the right, or farther from the failure line bounding the envelope of the allowable stress states. By contrast, when fluid is injected p rises and possibly exceeds p_0. In this case, the effective stress decreases under the original *in situ* value with Mohr-Coulomb's circle moving left, toward the failure line. If the Mohr-Coulomb's circle touches the envelope line a shear failure may occur and a fault/thrust may activate. Moreover, a dilation (or dilatancy) phenomenon, that is an increase of volumetric strain due to shear, could be induced, thus contributing to the magnitude of the injected formation expansion.

Venice (Italy) is a special case of predicted land uplift by a finite element model (Teatini et al 2011). Using an updated 3-D reconstruction of the Quaternary deposits, finite-element (FE) predictions are performed with injected water volumes into 650–1000 m deep Pleistocene formations through 12 boreholes located along a 10 km diameter circle centered over the city (Fig. 5). Careful adjustment of the injection pressure in the FE model allows for a prediction of a quite uniform 25–30 cm uplift over 10 years after the inception of injection (Fig. 5). The gradient of the vertical displacement ξ_z does not exceed 5×10^{-5}, that is well below the most conservative rebound recommended for the safety of the masonry structures. A pilot experiment has been designed to test the feasibility of the project for uplifting Venice. The pilot experiment plan foresees three boreholes located at the vertices of a triangle with respective sides 1 km long, in a lagoon area yet to be selected in the vicinity of the Venice historical center. The primary aim would be monitoring continuously and in real time the actual land uplift in the area, with the aid of high precision leveling, DGPS, and InSAR. The ultimate goal is the use of the local scale



Fluid Withdrawal, Fig. 5 Axonometric view of the hydro-geomechanical Finite Element model (FEM) of the Venice subsurface with the 12 injection wells (in red) and the predicted uplift (cm) of the city 10 years after inception of seawater injection that occurs in the blue and yellow formations (after Gambolat and Teatini 2014)

experiment to plan the full scale project for uplifting Venice. For a detailed description of the project of anthropogenic uplift of Venice, its expected cost and major environmental impacts, see Gambolati and Teatini (2014).

Summary and Conclusions

Anthropogenic land subsidence due to fluid withdrawal has been known for almost a century. Groundwater pumping is the primary cause of the occurrence worldwide although other examples include thermal water and hydrocarbon production. Overall damage is estimated today at billions of dollars a year (e.g., Borchers and Carpenters 2014), with an increasing trend due to population and economic growth; land settlement is still a problem largely under evaluated by both government and public opinion, especially in developing countries. Mitigation of land subsidence can be achieved by injecting treated surface water into the depleted formations, as is shown by the example of Wilmington oil field (Rintoul 1981) and the recharged aquifer systems of Santa Clara valley, California (Schmidt and Burgmann 2003) and Las Vegas Valley, Nevada (Bell et al. 2008).

The mechanism underlying the basic process is well understood and accepted. Mathematical modeling of past events and future evolution is well established. However, important research advancements must still be accomplished in predicting Earth fissuring, fault activation, and induced seismicity. These processes require approaches developed in the field of discontinuous mechanics, approaches only partially assimilated in engineering geology and geosciences so far. Monitoring their occurrence and developing representative, numerical models still pose major challenges for research in the near future.

Cross-References

▶ Consolidation
▶ Extensometer
▶ Groundwater
▶ Mechanical Properties
▶ Mohr-Coulomb Failure Envelope
▶ Soil Mechanics
▶ Soil Properties
▶ Subsidence

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Fluidization

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Definition

A process in which saturated or partly saturated unconsolidated sediments significantly and suddenly lose strength and stiffness because of an applied cyclic stress such as vibration during an earthquake (Seed and Lee 1966).

From the viewpoint of field survey, Lowe (1975) proposed three modes of deformation of sediment-water mixtures: hydroplastic, liquefied, and fluidized which, from the sedimentological viewpoint, give rise to water escape structures in natural strata. At the liquefaction stage, grains become temporarily suspended in pore fluid and settle rapidly through the fluid until a grain-supported structure is re-established. Fluidization is an allied process where the granular material is converted from a static solid state to a dynamic fluid state by the passage of water. This occurs when the drag exerted by moving pore fluids exceeds the effective weight of the grains, the particles are lifted, the grain framework is destroyed, and the sediment strength reduced to nearly zero. These processes are triggered by strong vibrations, often from earthquakes.

From the viewpoint of Geo-Engineering and Civil Engineering, these processes have been called "liquefaction."

However, advanced field analysis methods for liquefaction phenomena from anthropogenic strata, for example, trench survey, large lacquer peels made directly from trench faces and X-ray photogram analysis of borehole cores, and so on (Nirei et al. 2016), it became clear that the these phenomena consist of combined liquefaction, fluidization, and earthquake ground wave effects (Fig. 1; Hiyama et al. 2017).

These three phenomena jointly give rise to serious problems including:

- Major damage to services such as cables, drains, and pipelines, especially in hydraulic fill within anthropogenic deposits (Fig. 1)
- Subsidence of individual houses mainly on reclaimed land
- Interruptions to transport especially in reclaimed coastal land, for example, movement and tilting of bridge piers and quays (Fig. 2), and undulations in roads, railways, and airport runways
- Damage to basements of manufacturing facilities and eruptions of sand boils (Hiyama et al. 2017)

Fluidization, Fig. 1 Subsurface water storage tank for emergency use uplifted due to fluidization in hydraulic fill, Urayasu City, Tokyo Bay, Japan

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Fluidization, Fig. 2 Shifting and tilting of a quay structure due to fluidization during the 1995 Southern Hyogo Earthquake, Port Island, Kobe, Japan (*arrow* shows direction of movement)

Cross-References

- ► Artificial Ground
- ► Earthquake
- ► Cut and Fill
- ► Liquefaction
- ► Subsidence

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Fluvial Environments

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Definition

Sedimentary environments are places on the Earth's surface characterized by distinctive physical, chemical, and biological processes. *Fluvial environments* are a type of sedimentary environment, describing where fluvial landforms (geomorphology) and fluvial deposits (facies) are created, modified, destroyed, and/or preserved through the erosion, transport, and deposition of sediment.

Modes of fluvial sediment transport include bedload, suspended load, and dissolved load, and rivers are typically classified as bedload, mixed-load, or suspended load rivers based on the predominance of these modes. Dissolved load transport will not be discussed further in this section because it has a greater importance for water quality than for fluvial geomorphology and facies, with the exception of the importance of saline dissolved constituents in creating features and deposits in dryland environments. Most rivers also transport particulate and dissolved organic matter, and large woody debris (LWD) can be a major factor creating features and deposits in rivers, such as fluvial bars downstream of logjams.

Introduction

Studies of fluvial environments are sometimes split between *fluvial geomorphology* and *fluvial sedimentology*, but this distinction is artificial and should be avoided. Most observable features in streams (except small features such as ripple marks) formed under one set of flow conditions and were subsequently modified under different flow conditions; in other words, a large feature such as a bar has a history of multiple erosional and depositional events. Thus, the only way to correctly interpret fluvial geomorphic features is through sedimentological analysis. Similarly, the deposits (sedimentary facies) can only be understood by reference to features they form, for example, cross-bedded sand form from the downstream propagation of dunes. The trend today is to regard fluvial environments as entities constructed from a number of 3-D elements, where each *architectural element* (or morpho-stratigraphic unit) consists of a suite of related morphological features and sedimentary facies, separated from adjacent architectural elements by *bounding surfaces* (Miall 1996).

Fluvial environments are strongly affected by neighboring sedimentary environments, particularly *colluvial* (hillslope) *environments*, which introduce sediment into fluvial environments by various processes including rock fall, debris avalanches, slumps, debris flows, and sheet (unconfined) flows. In mountain environments, fluvial features such as rapids and bars are typically located proximal to sediment source areas, which are debris fans fed by colluvial processes. In dryland areas, ephemeral stream features are typically sourced by debris flows and sheet flows. Other important adjacent environments could include *volcanic environments*, *glacial environments*, *aeolian environments*, *lacustrine environments*, and *deltaic environments*. Each of these could serve as major sediment sources or sediment sinks for fluvial environments. In some cases, such as natural lakes or dam-reservoir systems, lacustrine and deltaic environments might interrupt the continuity of a through-going fluvial system. The processes governing these sedimentary environments could have a major impact on the fluvial system, for example, wave re-suspension of sediment deposited in reservoirs could significantly augment downstream suspended sediment loads.

Human impacts on fluvial environments are complex, and few fluvial environments can be understood without reference to historical changes in rivers due to human activity such as land clearance for agriculture, mining, or urbanization. A useful approach is to consider human impacts on *sediment budgets*, such that:

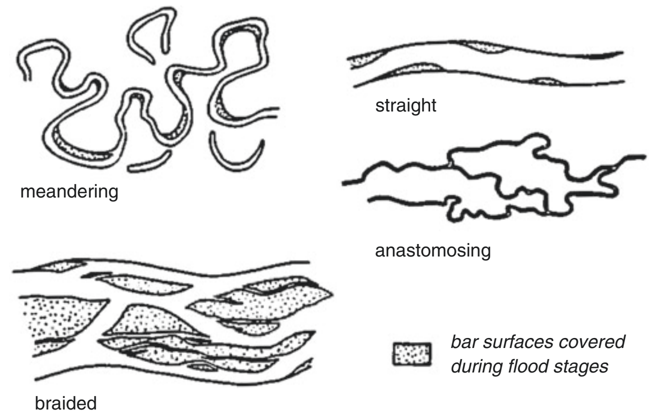
$$\text{Sediment Inputs} = \text{Sediment Outputs} + \Delta\,\text{Sediment Storage}$$

For example, there is widespread agreement that agricultural land clearance increases sediment inputs due to soil erosion from farm fields. Typically this increases both sediment outputs (bedload and suspended load) and sediment storage (aggradation of the fluvial system after exceeding conveyance capacity). The latter deposits are often referred to as *anthropogenic* or *legacy sediment*s (James 2013). For any river, reconstructing the causes of legacy sediment accumulation could provide key insights for river management and restoration (e.g., Webb-Sullivan and Evans 2015).

Morphologic Features

Fluvial environments are typically divided into *channels* (the location for both bedload and suspended load transport) and *floodplains* (typically dominated by suspended load transport). Each of these can be subsequently divided into proximal and distal sub-environments. Proximal channel environments include main stem and tributary channels, pools, riffles, channel bedforms (ripples, dunes, and bars), and features on channel banks. Distal channel environments include chute channels, scroll bars, levees, crevasse splays, and oxbows and outwash plains (sandurs) in glacio-fluvial environments. Proximal floodplain environments include floodplains, floodplain channels, flood-basin lakes, and wetlands. Distal floodplain environments are transitional to non-floodplain environments or may include infrequently inundated terrace surfaces.

Channels are commonly subdivided into length segments called *reaches* defined by changes in discharge (such as tributary inflows), bed and bank materials, or channel pattern. The four recognized channel patterns are shown in Fig. 1. *Straight* channels are relatively rare and more typical of high-energy, gravel-rich rivers or bedrock-confined rivers. *Anastomosed* channels may represent initial stages in avulsions, as described below.

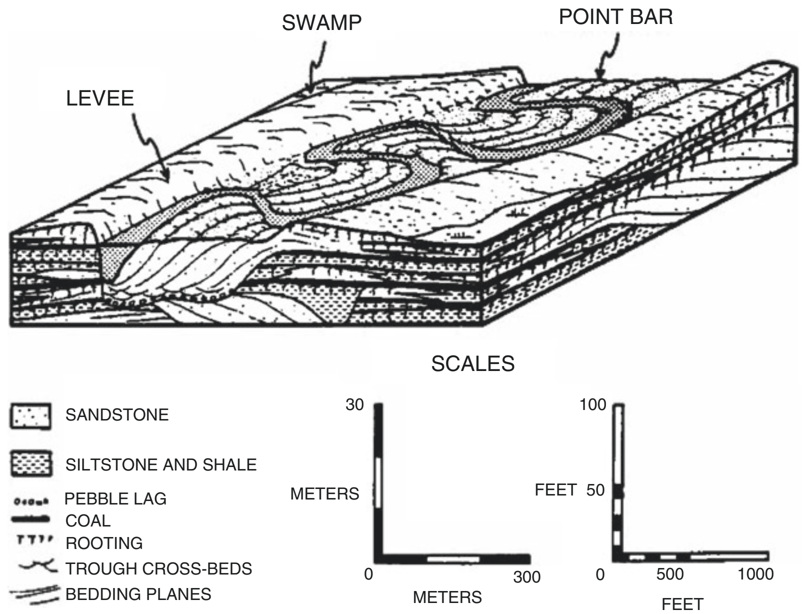


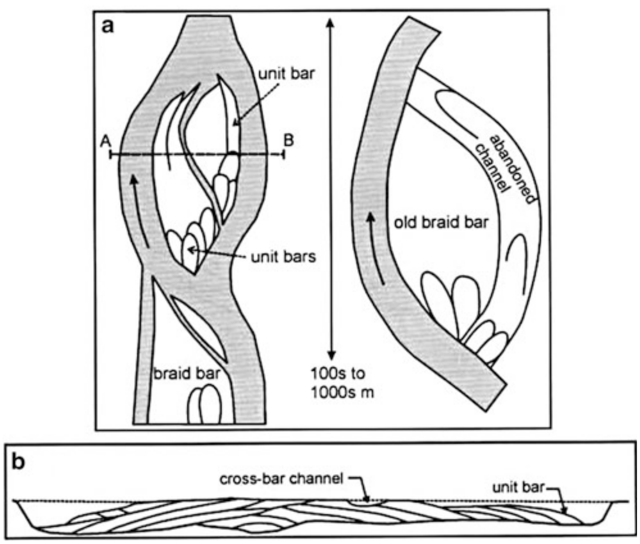
Fluvial Environments, Fig. 1 Types of channels based on platform geometry and sinuosity (Miall 1977)

Meandering channels have a sinuous pathway with cutbanks and pools at the outer part of bends, point bars on the inner part of bends, and riffles across the channel between sequential bends (Fig. 2). Lateral channel migration (erosion on the outer bend and deposition on the point bar) occurs episodically due to cutbank failure, typically on the falling stage. In the geologic record, these shifts in channel position produce *lateral accretion surfaces* (low-angle surfaces indicating sequential position of the *point bar*) in cross section and *scroll bar* topography in plain view (Fig. 2). At any location, point bar migration produces an overall fining-upward sequence as coarse-grained pool deposits are sequentially overlain by medium-grained sandy dune deposits in the lower point bar, fine-grained sandy ripples in the upper point bar, and finally silty-clay deposits from the floodplain. Channels might also shift position by *chute cut-offs* (reoccupation of swales in the scroll bar), by *neck cut-offs* (where loops of adjacent channels intersect), or by *channel avulsions* (where levee breach and sequential growth of a crevasse splay result in relocation of the channel). *Oxbow lakes* are abandoned portions of the channel resulting from neck cut-offs and display an infilling history where channel substrates are overlain by suspended-load sediment from introduced flood waters, interspersed with (and eventually replaced by) lacustrine gyttjas and peat.

Braided streams are often divided into sandy braided streams (primarily sand dunes) and gravel braided streams (primarily gravel bars with some sand dunes). Classification of fluvial dunes and bars is mostly based upon long-axis orientation of the feature with respect to flow direction, for example, *longitudinal bars* are oriented long-axis parallel to flow, whereas *transverse bars* are oriented long-axis perpendicular to flow (Ashley et al. 1990). However, large fluvial features commonly have complex histories where they formed in one hydrologic event and were subsequently modified. A useful approach (Fig. 3) is recognizing *unit bars* which formed under certain flow conditions versus *compound*

Fluvial Environments, Fig. 2 Sub-environments of a meandering stream showing morphostratigraphic units (Walker and Cant 1979; Horne et al. 1978)





Fluvial Environments, Fig. 3 Unit bars and compound bars in multiple-channel streams (Bridge 2003)

bars where one or several unit bars amalgamated within the channel or attached to the channel banks (Bridge 2003). Internally, sand dunes consist of cross-bedded sand reflecting downstream migration of the avalanche face of the dune. Gravel bars can be organized into bar-head, bar-platform, bar-margin, bar-tail, and supra-bar platform settings. Typically, bar-head deposits often contain imbricated gravel, bar platform deposits consist of crudely stratified gravel, and avalanche-face deposits at the bar margin or bar tail produce cross-bedded gravel (Bluck 1979).

Facies Analysis

Facies are the basic building blocks of any sedimentary deposit and are both descriptive and genetic, for example, trough cross-bedded sand are interpreted as the deposits of 3-D sand dunes. For fluvial environments, the most common form of facies analysis designates *lithofacies* (based on the physical characteristics of the geologic material) as shown in Table 1 (e.g., Miall 1977). However, there are alternative approaches, such as designating *radar facies* analysis using a ground-penetrating radar (e.g., Hickin et al. 2009), *seismic facies* analysis using environmental seismic methods (e.g., Grimm et al. 2013), and *pedofacies* analysis using properties of soils (e.g., Wright and Marriott 1996). Although not fully developed, there is also the potential of *biofacies* analysis, using properties of both living organisms (ecosystem structure) and dead organic materials (such as wood loads).

Lithofacies analysis of a particular river system would start with establishing a lithofacies classification system similar to Table 1. This classification system is then used to describe vertical and lateral trends observed in surficial deposits, trenches, or cores. As shown in Fig. 4, use of lithofacies codes helps organize observations and appreciably speeds up the description process. Surfaces, which are transitions between adjacent lithofacies, are particularly important because these might represent time gaps (unconformities) due to erosion (such as scours) or due to nondeposition (such as weathered surfaces, soils, or paleosols).

The next steps in facies analysis look for which facies are commonly found adjacent to one another vertically or laterally. Such *facies associations* represent key components in the

Fluvial Environments, Table 1 Common fluvial lithofacies

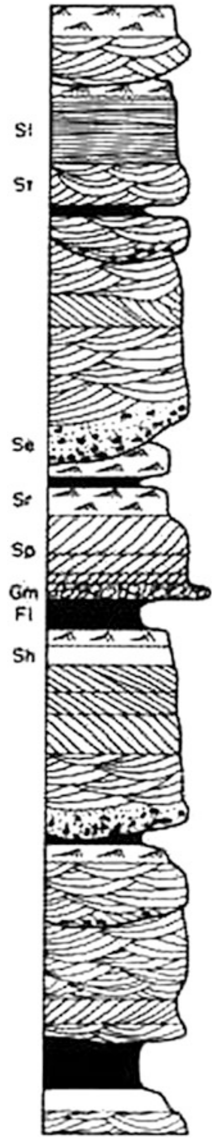
Code	Lithology	Textures	Sedimentary structures	Interpretation
Gms	Gravel	Coarse to fine grained, poorly sorted	Massive	Debris flow deposit
Gm	Gravel	Coarse to fine grained, moderately sorted	Massive	Bar platform deposit
Gh	Gravel	Coarse to fine grained, moderately sorted	Planar bedded	Bar platform deposit
Gt	Gravel	Coarse to fine grained, moderately sorted	Trough cross-bedded	Supra-bar platform minor channel fills
Gp	Gravel	Coarse to fine grained, moderately sorted	Planar-tabular cross-bedded	Linguoid bars or bar-margin avalanche face (small bar-pool deltas)
Sm	Sand	Coarse to fine grained, moderately sorted	Massive, destratified	Rapid deposition, or homogenized by roots
Sh	Sand	v.cos. to med. grained, moderately sorted	Planar bedded	Upper/lower flow regime plane bed
Sl	Sand	Coarse to fine grained, moderately sorted	Low-angle ($<10^0$) cross-bedded	Scour fills, crevasse splays, antidunes
St	Sand	v.cos. to med. grained, moderately sorted	Trough cross-bedded	3-D dunes (lower flow regime)
Sp	Sand	v.cos. to med. grained, moderately sorted	Planar-tabular cross-bedded	2-D dunes (lower flow regime)
Sr	Sand	cos. to v. fine grained, moderately sorted	Ripple marks or ripple laminated	Ripples (lower flow regime)
Se	Sand	v.cos. to fine grained, moderately sorted	Erosional scours with mud intraclasts	Scours and scour fills
Ss	Sand	v.cos. to fine grained, moderately sorted	Shallow scours	Scours and scour fills
Fl	Sand, silt, mud	Range of fine sizes, typically well sorted	Planar lamination, flood couplets	Overbank or waning flow deposits
Fsc	Silt, mud	Range of fine sizes, typically well sorted	Laminated to massive	Backswamp deposits
Fcf	Mud	Range of fine sizes, typically well sorted	Massive with freshwater molluscs	Backswamp pond deposits
Fm	Silt, mud	Range of fine sizes, typically well sorted	Massive, destratified, desiccation cracks	Overbank or drape deposits, soils
Fr	Silt, mud	Range of fine sizes, typically well sorted	Massive, with rootlets	Mineral soils (various types)
C	Carbonaceous mud, peat/coal	Mixture of fine-grained sediment/organic matter	Peats, leaf litter layers	Organic soils (incl. histosols)
P	Pedogenic carbonate	Soil hosted in sand/mud	Carbonate nodules or rhizoliths	Calcisols

Source: Modified from Miall (1977)

depositional environment, for example, certain lithofacies would be commonly found in the downstream migration of a gravel bar. Statistical techniques can be used to improve the robustness of these binning efforts. Repetitive vertical successions, or *facies sequences*, can be interpreted as the evolution (through time) of certain features and deposits, such as the fining-upward point bar sequence (Fig. 4). Statistical methods, such as Markov chains, can improve these interpretations. Facies associations do not cross major unconformity surfaces because those time gaps interrupt the continuity of the related fluvial processes that produced any specific association of lithofacies.

Architectural Element Analysis

Architectural element analysis is reviewed by Miall (1985). Each *architectural element* is a three-dimensional facies association separated from adjacent architectural elements by *bounding surfaces* (surfaces of erosion or nondeposition). The most common architectural elements are shown in Table 2. In scale, each architectural element can be up to meters thick and hundreds of meters in lateral dimensions, and understanding their full extent and contact relations requires exceptional exposures or correlating numerous trenches and cores (Fig. 5). Other important aspects of the internal fabric of an architectural element include the



Fluvial Environments, Fig. 4 Example of facies analysis of braided stream deposits (Miall 1977)

Fluvial Environments, Table 2 Lithofacies composition and geometry of architectural elements

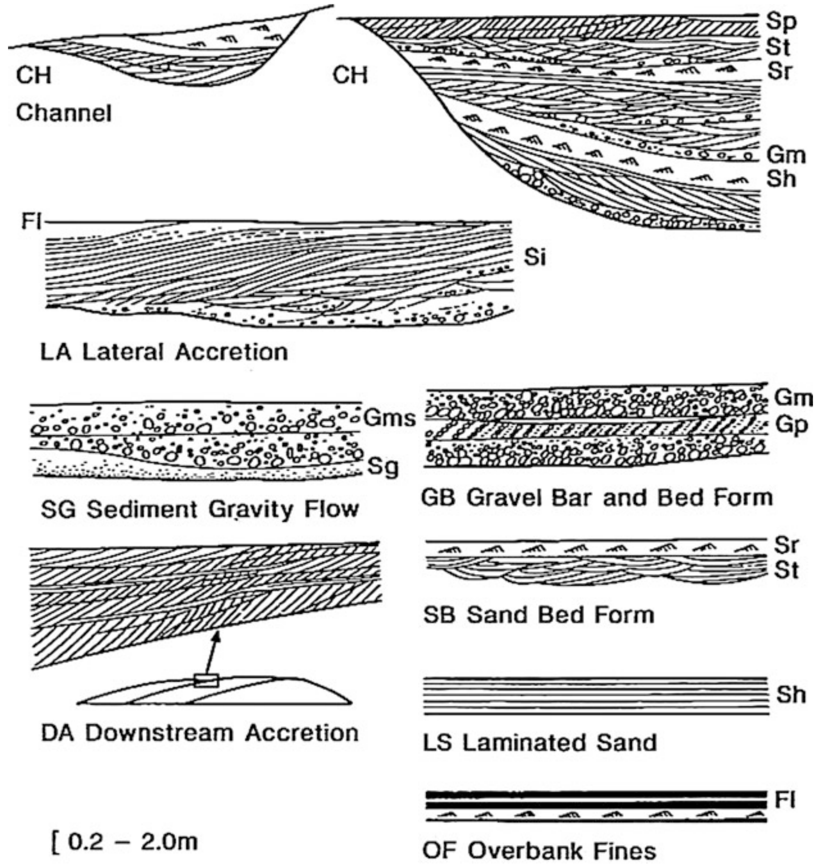
Element	Symbol	Principal facies assemblage	Geometry and relationships
Channels	CH	Any combination	Finger, lens or sheet; concave-up erosional base; scale and shape highly variable; internal concave-up third-order erosion surfaces common
Gravel bars and bedforms	GB	Gm, Gp, Gt	Lens, blanket; usually tabular bodies; commonly interbedded with SB
Sandy bedforms	SB	St, Sp, Sh, Sl, Sr, Se, Ss	Lens, sheet, blanket, wedge, occurs as channel fills, crevasse splays, minor bars
Upstream-accretion macroform	UA	St, Sp, Sh, Sl, Sr, Se, Ss	Lens, resting on bar remnant or LA/DA deposit. Accretion surfaces dipping gently upstream
Downstream-accretion macroform	DA	St, Sp, Sh, Sl, Sr, Se, Ss	Lens resting on flat or channeled base, with convex-up third-order internal erosion surfaces and upper fourth-order bounding surface. Accretion surfaces oriented downstream
Lateral-accretion macroform	LA	St, Sp, Sh, Sl, Se, Ss – less commonly Gm, Gt, Gp	Wedge, sheet, lobe; characterized by internal lateral-accretion third-order surfaces. Accretion surfaces oriented across channel. Typically downlaps onto flat basal erosion surface
Scour hollows	HO	Gh, Gt, St, Sl	Scoop-shaped hollow with asymmetric fill
Sediment gravity flows	SG	Gmm, Gmg, Gci, Gcm	Lobe, sheet, typically interbedded with GB
Laminated sand sheet	LS	Sh, Sl – minor Sp, Sr	Sheet, blanket

Source: modified from Miall (1996)

vertical sequences, presence or absence of minor erosion surfaces, orientation of features, paleoflow directions, and relationship of internal bedding features to the enclosing bounding surfaces (which are described using terminology such as onlap, downlap, parallel orientation, or truncation).

Architectural elements will exhibit a hierarchy based upon the *rank* of the bounding surfaces. Relatively minor changes between sequential elements would be indicative of a *first-order bounding surface*, such as the transition from one cross-bed set to another, indicating changes in transport energy or flow direction between hydrologic events. A more significant change would be represented by *second-order bounding surfaces* (which truncate all first-order bounding surfaces), such as sequential positions of lateral accretion surfaces. The bounding surface ranking system continues to increase in number, representing larger-scale combinations of features and deposits, with each higher rank cross-cutting all lower rank surfaces, until finally reaching the scale of the largest element, such as the valley or paleo-valley.

The analysis of a fluvial system, using architectural element analysis, would reveal lateral changes in the type, scale, and orientation of three-dimensional fluvial features and their related deposits and also vertical changes indicative of the evolution of the fluvial system through time. Such an analysis provides a solid understanding to base interpretations about controlling factors acting on the fluvial system. These might include the effects of tectonics (uplift and subsidence), changes in sea level position (baselevel), changes in sediment supply,



Fluvial Environments, Fig. 5 Architectural element analysis (Miall 1985)

changes in climate, or historical changes due to human impacts (Horn et al. 2012). Predictive models have been constructed to determine the variations in channel types and stacking patterns (single-story channels or multistory channels) and sand-body connectivity under a range of different combinations of controlling factors (Bridge and Mackey 1993).

Summary and Conclusions

Fluvial environments have been widely studied, but unfortunately the literature is highly and artificially compartmentalized, such as making a strong distinction between features (fluvial geomorphology) and deposits (fluvial sedimentology). A more recent approach is to recognize that fluvial environments are constructed from distinctive combinations of genetically related features and deposits (*architectural elements*) separated laterally and vertically from adjacent architectural elements by *bounding surfaces* of different rank. Each architectural element is described by its facies association (group of related lithofacies), scale, geometry, and orientation. *Architectural element analysis* provides an understanding of the processes acting at a particular place and time on a fluvial system. Tracking spatial and temporal changes in architectural elements provides insights into changes of the external and internal factors controlling the fluvial system (tectonics, eustasy, sediment supply, climate, and human activity).

Cross-References

▶ Dams
▶ Facies
▶ Glacier Environments
▶ Lacustrine Deposits
▶ Landforms
▶ Mountain Environments
▶ Reservoirs
▶ Sediments
▶ Volcanic Environments

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Foundations

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Definition

Engineered works that transfer the load of a civil engineering structure or building to the underlying ground.

The design depends on the subsoil conditions and expected amount of loading: if loads are small then shallow footings may be appropriate, depending on the nature of the ground, but if loads are considerable then substantial foundations and/or deep piling may be necessary. Appropriate foundation design depends on good site investigation, a sound understanding of the ground conditions, a working knowledge of soil mechanics and experience in designing and undertaking works.

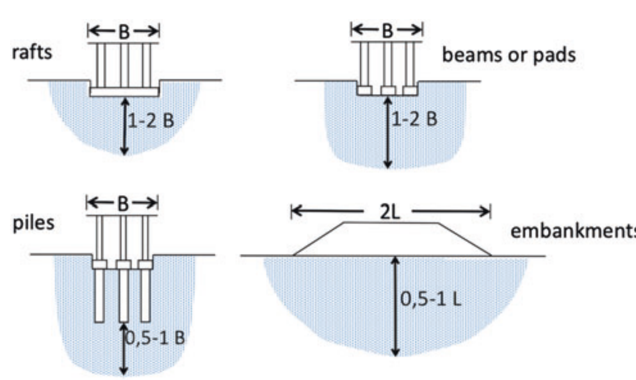
Assessment of the Soil Profile for Design

The design of foundations relies on analytical, numerical, or empirical models using parameters that represent soil response to the stress variations induced by the applied loads. These parameters are part of the *geotechnical model*, which differs from the *geological model*. The latter is aimed at identifying the *geological units*, their origin, and any peculiar feature such as bedding, joints, faults, etc., and providing crucial information to forecast hazards of both geological and hydrogeological nature. The geotechnical model is the idealization of the subsoil needed to carry a specific calculation, implying the quantification of soil behavior through the values of the relevant physical and mechanical parameters. Therefore, the two models (geological and geotechnical) are in strict correlation but may differ in some respects.

Basic Requirements of Foundation Analysis and Design

The Soil-Foundation-Structure System

The foundations are part of a complex system, consisting of the superstructure, the foundations themselves, and the interacting soil. In principle, such a system should be analyzed as a whole. But such an analysis is extremely complex so, in practice, the process is divided in two parts: the soil-foundation subsystem (Fig. 1) and the superstructure. In so doing, it is implicitly assumed that the loads carried out by the foundation are known; they are calculated in a separate analysis of the superstructure with fixed base restraints, or even simply by influence areas.



Foundations, Fig. 1 Extension of the mechanically influential soil volume for some typical foundation schemes

In most cases, however, superstructures are statically indeterminate, and the forces and bending moments acting on the base boundaries are accordingly unknown quantities, depending on the displacements of the foundation, which in turn depend on the characteristics of the foundations and on the mechanical properties of the soil. Therefore, by dividing the whole system in two subsystems, the combinations of loads used in foundation design are only an approximate estimate of the true loads.

Ultimate Limit State

A foundation, as any other structure, must satisfy a number of requirements, to be fulfilled both during its life and during construction. When a structure or parts of it fail to satisfy one or more of the above requirements, a *limit state* has been attained. This section will deal with the *ultimate limit state*, at which a collapse mechanism takes place in the ground; next one will deal with the *serviceability limit state,* at which foundation displacements produce a loss of serviceability of the interacting structure.

The analysis of the ultimate limit state of a foundation is performed evaluating the ultimate unit load q_{ult} (also called *bearing capacity*). The evaluation may be obtained by different procedures (theory of plasticity, limit analysis, limit equilibrium, numerical computation, empirical approaches). Currently there is a general agreement on the analytical formulations that will be mentioned in the sections on Shallow Foundations and on Pile Foundations.

The safety against the ultimate limit state is obtained by imposing that the service unit load q does not exceed the bearing capacity q_{ult}. Current technical regulations (e.g., Eurocodes) control safety introducing partial factors on either the action (to amplify q) or the resistance (to reduce the geotechnical parameters and therefore q_{ult}), and an overall resistance factor that, because of the partial factors, should not be confused with a safety factor.

The analysis of ultimate limit state may appear relatively easy; indeed, the design of a foundation is far from the mere application of a bearing capacity formula, because of the peculiar features of soil behavior. The process actually requires a number of assumptions concerning the selection of proper values of the design parameters.

As an example, when assessing the bearing capacity of a shallow foundation on sand, it is important to select the value of the angle of shear resistance taking into account the relative density of the sand and the expected stress level induced into the soil. As a matter of fact, even small changes produce significant changes of the computed bearing capacity.

Similarly, for footings on clay soil, the operational value of the undrained shear strength s_u has to be selected taking into account at least its dependence on the stress history and the loading path (Jamiolkowski et al. 1985). More complex situations may arise when dealing with nonhomogeneous materials, as in the case of a change of strength with depth or of stratified subsoils (Poulos et al. 2001). Therefore, compliance with the requirements in Codes is not sufficient by itself to ensure safety, and good practice relies on engineering experience and judgment to formulate a proper geotechnical model, to select proper values of soil parameters and to account for sources of uncertainties.

Serviceability Limit State

When dealing with the problem of relating foundation displacements to damage, it must be borne in mind that serviceability and damage are rather relative concepts, in that they depend on the function of the structure and even on the empathic reaction of the users.

When the loads are applied to a foundation during the construction of the superstructure, a settlement takes place and the already existing part of the structure may suffer some distortion. The overall stiffness of the structure increases as the construction evolves, and when cladding is added the stiffness is again substantially increased. Then, live loads start to be imposed. As expected, during the construction process not all components of the structure are subjected to the same relative deflections; this will depend on the location and elevation of the considered component, and only a portion of these deflections will affect cladding and finishes, possibly giving rise to architectural damage.

Forecasting damage to a given structure is therefore not a simple task, because of its dependence on the construction process, the ratio of immediate to delayed settlement and of live to dead loads. Furthermore, it must be recalled that structural movements depend also on factors other than foundation displacements, such as creep, shrinkage, and temperature variations.

The deformed configuration of a foundation (Fig. 2) may be defined by the maximum settlement $w_{\max}$, the differential settlement δ, the deflection Δ, the angular distortion β, the deflection ratio Δ/L. The values of δ, Δ, β, Δ/L are referred to two points that are not necessarily the extremities of the foundation. The maximum value of the deflection ratio Δ/L does not necessarily correspond to the maximum value of Δ. In Fig. 2, for instance, $(\Delta/L)_{\max}=\Delta_{AC}/L_{AC}$, even though $\Delta_{AD}<\Delta_{AC}$.

Experience has shown that the damage to the superstructure is related to the values of δ, β, and Δ/L, more than to the values of the maximum settlement $w_{\max}$. The design against serviceability limit states requires that the overall and differential displacement of a foundation are kept below the corresponding admissible values. Data on the allowable values of the above quantities have been collected by a number of authors. O'Brien and Farooq (2012) distilled from the previous indications the values listed in Table 1.

The use of a single quantity, such as the angular distortion or the deflection ratio, to assess building damage excludes

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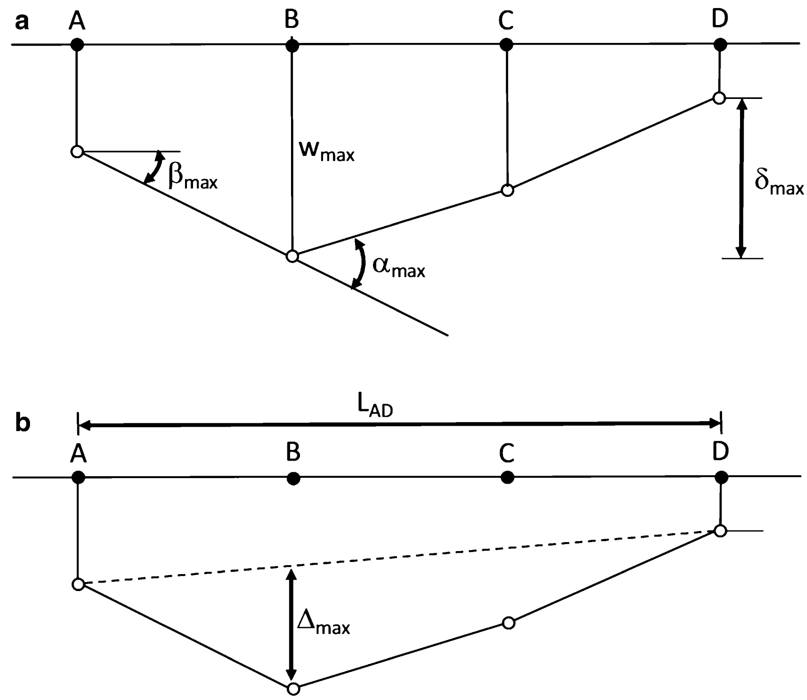

Foundations, Fig. 2 Definition of foundation movements

Foundations, Table 1 Routine limits for allowable movements (Modified after O'Brien and Farooq 2012)

Type of structure	Type of damage	Criterion	Routine limits	Comments
Framed buildings and reinforced load-bearing walls	ULS structural damage	Angular distortion	1 in 150 to 1 in 250	ULS concerns at these limits
Framed building and reinforced load-bearing walls	SLS cracking of walls, cladding, partitions	Angular distortion	1 in 300 to 1 in 500	Typically SLS concerns at these limits
Unreinforced masonry walls	Visual onset of cracking	Deflection ratio	Sagging 1 in 2500 (L/H = 1) 1 in 1250 (L/H = 5) Hogging 1 in 5000 (L/H = 1) 1 in 2500 (L/H = 5)	At these limits there is only the onset of cracking; the damage is very slight Tolerable movements are several times larger H = height; L = length
Steel, fluid storage tanks	SLS leakage	Angular distortion	1 in 300 to 1 in 500	
Utility connections	SLS	Maximum settlement	150 mm	Less for sensitive utilities, such as gas main
Crane rails	SLS crane operation	Angular distortion	1 in 300	Depends on specific crane
Floors, slabs	SLS drainage	Angular distortion	1 in 50 to 1 in 100	Depends on specific falls, alignments
Stacking of goods	ULS, collapse	Tilt	1 in 100	
Machinery	SLS, efficient operation	Angular distortion	1 in 300 to 1 in 5000	Depends on machine type and sensitivity
Towers, tall buildings	Visual	Tilt	1 in 250	Tilt in excess of this will be noticeable, though possibly remote from collapse
Lift and escalator operation	Lift and escalator operation	Tilt after installation	1 in 1200 to 1 in 2000	Sequence of construction and time of installation is important
Bridge	SLS	Angular distortion	1 in 250 to 1 in 500	Depends on deck characteristics
Bridge	SLS	Maximum settlement	60 mm	Typical values
Bridge	SLS bearing	Horizontal displacement	40 mm	Typical values

many important factors, related to the building (flexural and shear stiffness, geometrical configuration), to the subsoil (coarse grained or fine grained soils, and related differences in the rate of occurrence of settlement), to the nature of ground movement profile (e.g., sagging or hogging), to horizontal strain.

Shallow Foundations

Footing Classification

A shallow foundation is the most natural and convenient option when a suitable soil is found at a depth small enough to be easily reached by an excavation.

The types of shallow foundations are schematically represented in Fig. 3. *Pad* foundations (Fig. 3a) may be square, circular, or rectangular and usually support one or two columns. *Strip* foundations (Fig. 3b) are commonly adopted to support a row of closely spaced columns or a bearing wall; the length L of a strip foundation is usually much greater than its width B. A *raft* foundation (Fig. 3c) supports the entire structure or a substantial part of it. A *compensated raft* or *buoyant raft* (Fig. 3d) includes a void, thereby reducing the pressure increase on the underlying ground.

As for all the other parts of a structure, the design of a shallow foundation requires avoiding two limit situations: the ultimate limit state, at which a collapse mechanism takes place in the ground, and the serviceability limit state, at which the displacements of the foundation produce a loss of serviceability of the interacting structure. Once the loads exerted onto the foundation by the superstructure are forecast, the size, in plan, of the foundation must be amended to satisfy the above requirements.

Bearing Capacity

As stated above, the safety against the ultimate limit state requires the calculation of the ultimate load or bearing capacity q_{ult}. It is almost universally accepted that q_{ult} can be expressed as:

$$q_{\mathrm{ult}} = N_q \gamma D + N_c c + N_\gamma \gamma \frac{B}{2} \tag{1a}$$

where N_q, N_c, and N_γ are dimensionless factors, functions of the angle of shear resistance of the soil, γ is unit weight and c the cohesion (if any) of the soil, D is the depth, and B the width of the foundation (Fig. 3).

The formula is valid for an infinitely long strip under a centered vertical load, resting on a homogeneous subsoil with a horizontal ground surface. Different shapes of foundations, eccentric and/or inclined loads, and inclined ground surface may be accounted for by corrective coefficients; details may be found in standard textbooks on foundation engineering (Salgado 2008; Fleming et al. 2009; Burland et al. 2012). The formula has been reported here essentially to underline that the allowable load on a foundation is not a property of the soil (as it is, unfortunately, still widely believed), but it depends also on the geometrical features of the foundation, on the applied loads, etc.

The evaluation of the ultimate load has to be carried out in undrained condition for fine grained foundation soils, and in drained condition for coarse grained subsoil. In the former case, Eq. 1a has to be applied in terms of total stresses and becomes:

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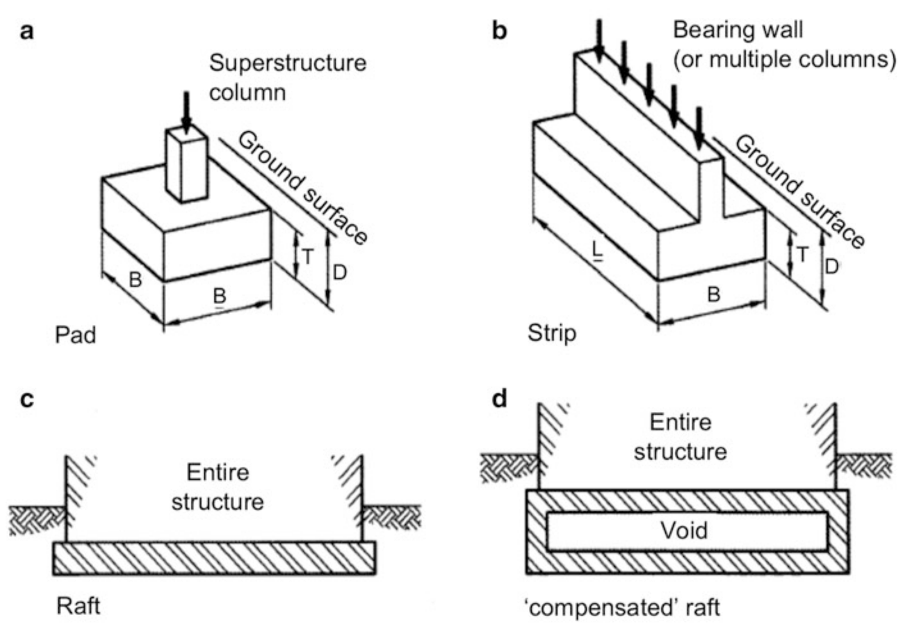


Foundations, Fig. 3 Types of shallow foundations (Modified after O'Brien 2012)

$$q_{u,\mathrm{ult}} = \gamma D + 5.14 s_u \tag{1b}$$

In drained conditions, Eq. 1 has to be applied in terms of effective stresses with $c' = 0$ and a friction angle φ'. It then becomes:

$$q_{\mathrm{ult}} = N_q \gamma D + N_\gamma \gamma \frac{B}{2} \tag{1c}$$

where q_{ult} is the effective load on the foundation, the term γD represents the effective stress at the foundation depth, and γ in the second term has to be the submerged unit weight γ' if the groundwater table is at the foundation depth or shallower.

With the exception of relatively small foundations resting on stiff clays, however, the final design of a shallow foundation is generally determined by the serviceability limit state more than by the ultimate one.

Prediction of Foundation Settlement

A steel structural member has a working stress in the order of 2×10^2 MPa, a Young's modulus of 2×10^5 MPa, and hence a strain of the order of 0.1%; and a concrete member a working stress of the order of 10 MPa, a Young's modulus of about 200 MPa, and a working strain of 0.05%. The integration of such strains identifies displacements rarely exceeding a few millimeters.

A typical working stress for a soil is of the order of 0.1 MPa (incidentally, this is the reason why a foundation is needed), a Young's modulus of 10 MPa, and therefore a strain of about 1%. The integration of such strain within the pressure bulb of a foundation brings to displacements amounting to some tens or even hundreds of millimeters. The prediction of the displacements of a foundation and the evaluation of their admissibility is hence a fundamental design step.

In the large majority of cases, vertical load and vertical displacements are the relevant issues; the vertical displacement of a foundation is called settlement.

Settlement prediction is significantly different, in practice, for fine grained and coarse grained soils. In the case of a subsoil consisting essentially of fine grained soils (silt and clay), the load application occurs under essentially undrained conditions since the construction time is generally much shorter than the time needed to a substantial dissipation of the excess pore pressure generated by loading. At the end of construction, an undrained or immediate settlement w_o will have taken place, resulting from a field of undrained distortional strains at constant volume. After the end of construction, a consolidation process begins, with the dissipation of the excess pore pressure and volume variations. Disregarding the long-term secondary settlement that can be of some relevance only in particular cases (e.g., organic soils), the final settlement w results by the sum of the immediate settlement w_o and of the delayed consolidation settlement w_c:

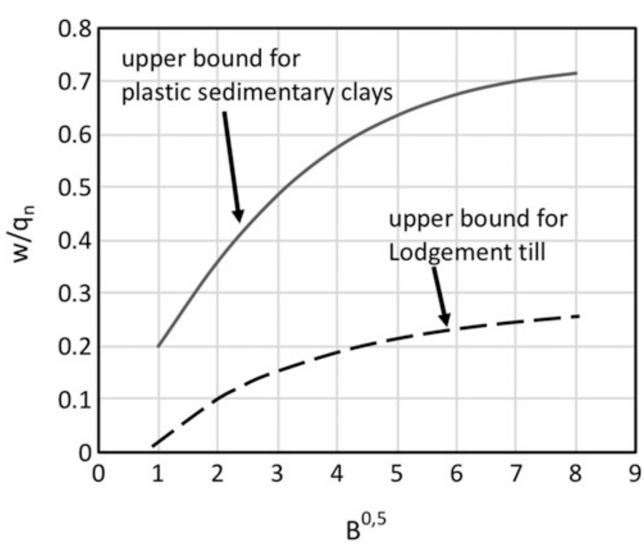
$$w = w_o + w_c \tag{2}$$

The mechanical properties of fine grained soils may be determined fairly reliably by laboratory tests on undisturbed samples or by different in situ tests; accordingly, it is possible to adopt the classical procedures of Soil Mechanics for the prediction of settlement. The immediate settlement resulting from an undrained process is best evaluated in terms of total stress, using the undrained deformability characteristics of the soil. The evaluation of consolidation settlement is, on the contrary, performed in terms of effective stress and of drained deformability parameters of the soil. The so-called oedometric method, based on the simple and reliable oedometer test and widely presented in standard textbooks, is generally adopted with satisfactory results.

Both theory and experimental evidence show that in stiff overconsolidated clay the immediate settlement accounts on average for 50–60% of the total final settlement. On the other hand, in sych soils the consolidation process is relatively fast. For both these reasons, an evaluation of the immediate settlement is of practical relevance. On the contrary, in soft to medium clay (normally consolidated or slightly overconsolidated deposits) the immediate settlement is of little importance being, on average, of the order of 10% of the final amount, and the consolidation process is much slower.

Some indications on the maximum values of the expected settlement of shallow foundations on stiff clay or on tills are shown in Fig. 4.

In the case of a subsoil consisting essentially of coarse grained soils (sand, gravel), the settlement occurs immediately at the load application, and attains its final value at the end of construction; it is a fully drained process. As a



Foundations, Fig. 4 Upper bounds of observed settlement in stiff clay and tills. Foundation width B in m, net foundation pressure q_n in kPa, settlement w in mm (Modified after O'Brien and Farooq 2012) consequence, immediate and final settlement practically coincide. Furthermore, it is practically impossible to recover undisturbed samples for laboratory testing, hence prediction methods are based on correlations with site tests such as standard penetration test or cone penetration test.

In any case, excessive settlement of shallow footings on sand is seldom a practical problem, since most observations indicate that settlement is generally less than 40–50 mm. Figure 5 shows some experimental observations.

If more detailed calculations of settlement on sand are needed, the most reliable methods are that of Burland and Burbidge (1985), based on SPT, and that of Schmertmann et al. (1978), based on CPT. Both methods are extensively described in standard textbooks.

As observed above, damage to the superstructures, more than to the values of the settlement *w*, are related to the values of δ, β, and Δ/L. The expected maximum values of differential settlement and angular distortion, to be compared to the admissible values, are difficult to obtain as the result of a deterministic design analysis, being markedly dependent on random factors as variability of soil properties, installation procedures, and history of construction and loading. It is therefore useful, at least as a first step, to adopt empirical relations between the distortion parameters and some readily evaluable quantity. Figure 6, for instance, shows curves that bound the maximum observed values of the differential settlement δ as a function of the maximum settlement w_{max}.

Figure 7 similarly shows the maximum observed values of the angular distortion β as a function of the maximum settlement w_{max}. The upper bounding envelope of the experimental data in Fig. 7 may be expressed as:

$$\beta_{max} = 9 \times 10^{-4} w_{max} \tag{3}$$

with w_{max} expressed in cm.

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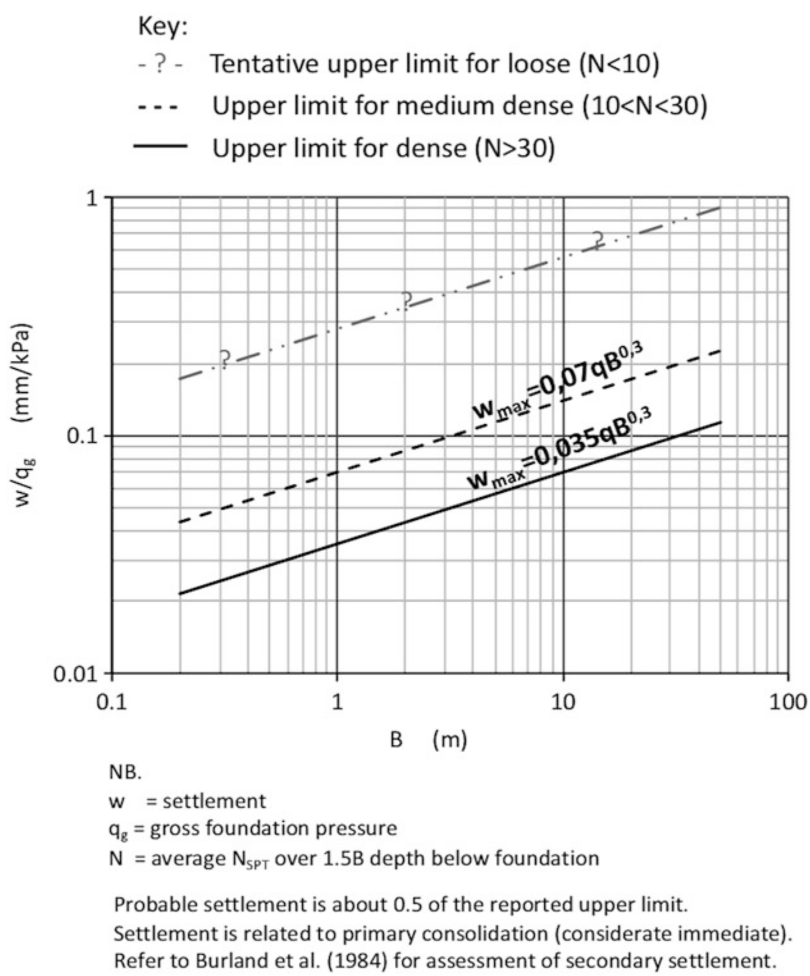
Pile Foundations

Introduction

In the last few decades, developments in equipment and installation techniques and the pressure toward constructing in areas with poor subsoil properties has led to spectacular progress in the piling industry. Available piles range, at

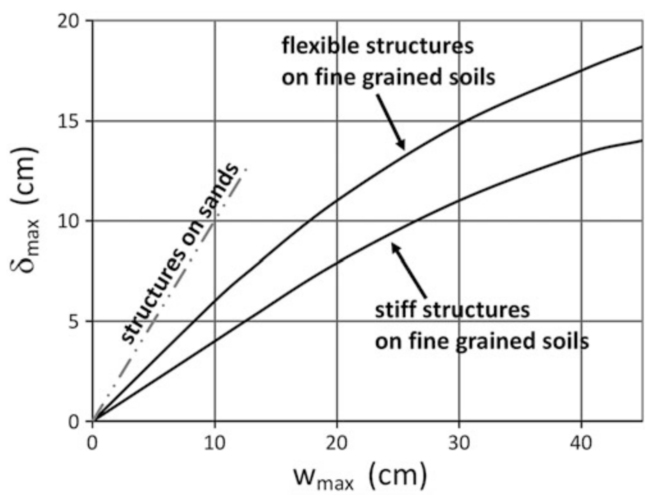
Foundations, Fig. 5 Upper limits of observed settlement in sand and gravel



present, from micropiles with a diameter of 150–250 mm and load capacity of a few tens of tons, to large diameter (up to 2.5 ÷ 3 m) bored piles and large tubular steel piles for offshore structures with diameters up to 3 m, lengths of many tens and sometimes over a hundred meters and load capacities of many hundreds and even thousands tons.

Some of the situation in which a pile foundation is adopted are sketched in Fig. 8.

The most usual case is that of subsoil with poor upper layers, not suitable for shallow foundations. In this case (Fig. 8a) the piles transmit the load to deeper competent layer (point bearing piles). If such a layer is not found within an acceptable depth, the load may be transmitted by side shear (floating piles, Fig. 8b). Side shear may resist upward load (tension piles, Fig. 8c). Horizontal loads may be resisted by vertical piles against bending and shear (Fig. 8d) or by pile groups including raking piles (Fig. 8e). Piles are often used for bridge piers to keep the foundations below the maximum depth of scour (Fig. 8f), or to prevent possible adverse effects of a future excavation (Fig. 8g). If swelling or collapsible soils are found near the surface, piles may be used to reach deeper soils, not affected by seasonal water content variations (Fig. 8h).



Foundations, Fig. 6 Correlation between foundation settlement and maximum differential settlement

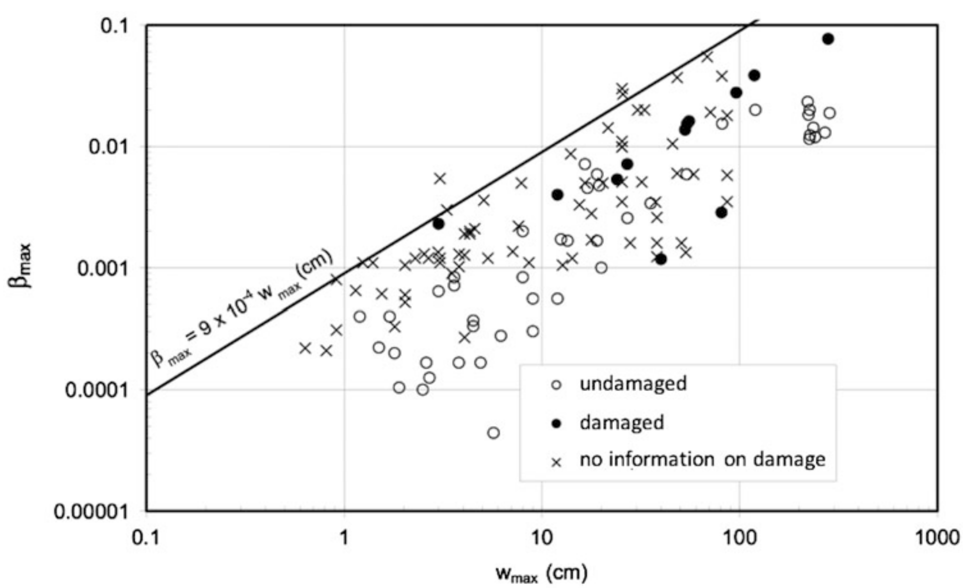
The soil in contact with the lateral pile surface, from which the pile derives its support by side friction, and the soil beneath the pile toe, seat of the point resistance, are both deeply influenced by the pile installation. The behavior of a pile, therefore, is markedly affected by its installation technique. This factor substantially differentiates piles from shallow foundations.

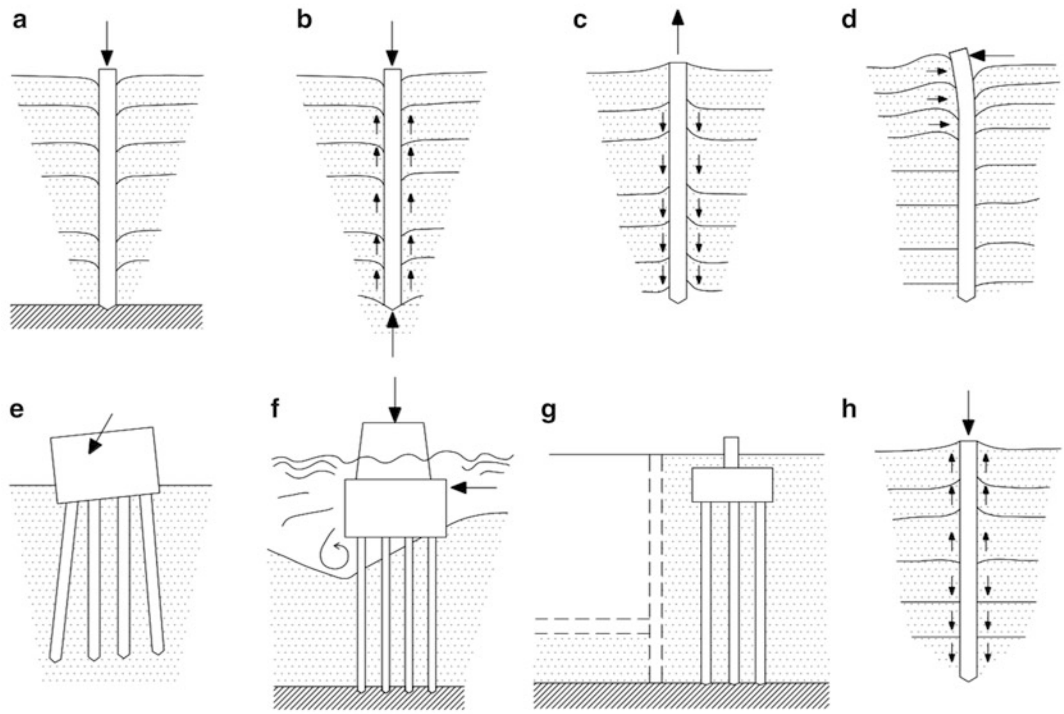
The behavior of piles, accordingly, can hardly be predicted by means of soil or rock mechanics theories; the best that can be done in practice is to apply to the theoretical formulae for empirical factors based on experience and on the results of field loading tests. The proper choice of the values of such factors requires a clear insight of the effects of the various installation techniques.

Review of Pile Types

The number of proprietary piles available on the market is very large and steadily increasing, due to the introduction of new techniques or new details in the existing equipment and techniques; it is important, however, to have a clear idea of the principal pile installation techniques and their effects on the behavior of the pile, since the choice of the pile type is an important part of the design.

Foundations, Fig. 7 Correlation between the maximum settlement and the maximum angular distortion





Foundations, Fig. 8 Typical cases for the adoption of a pile foundation

Foundation piles may be classified following different criteria. Referring to their size, piles can be subdivided into small diameter ($d \leq 250$ mm) medium (300 mm $\leq d \leq 600$ mm) and large diameter ($d \geq 800$ mm). Of course, these limits are largely conventional but they are of some practical use since design criteria are somewhat different for piles of different size.

The installation technique is the most important differentiating factor. The fundamental distinction is that between displacement (driven, pushed, or screwed) piles and replacement (bored, or drilled) piles. In the former, there is no removal of soil, whereas in the latter a hole is previously bored, and the removed soil is replaced by concrete. There are, in addition, quite a number of intermediate pile types. Some of the most common types are listed in Table 2; some proprietary denominations have been listed when they are commonly used to designate a type.

The principal advantages and disadvantages of the various types of piles described above can be summarized as follows.

Displacement piles are in general suitable in cohesionless soils, since their installation increases the horizontal stress and decreases the porosity in the soil surrounding the pile, thus improving its strength and stiffness, and increasing the stiffness and bearing capacity of the pile. On the contrary, they have to be adopted with caution in cohesive soils, since their installation remolds the surrounding soils and induces high excess pore pressures.

Prefabricated driven piles are not liable to squeezing or necking, and they can be inspected for soundness before driving. Their installation is not affected by groundwater, and their projection above ground level may be advantageous in structures in shallow water, such as jetties, harbors,

Foundations, Table 2 Principal types of piles

Replacement (bored, drilled) Small, medium, or large diameter	Percussion or rotary bored, with/without casing, with/without bentonite mud, with/without enlarged base Continuous flight auger (CFA), with grout or concrete injected from the shaft during extraction	
Partial displacement Small, medium, or large diameter	Small displacement (driven H sections; open ended pipes with soil inside removed, casing recovered after concreting) Continuous flight auger with large central shaft, extracted with partial soil removal but allowing the placement of reinforcement before concreting (PressoDrill)	
Displacement (inserted by driving, pushing, or screwing) Small or medium diameter	Prefabricated	Wood Concrete: reinforced, centrifuged, prestressed Steel: closed end pipes, open ended pipes, and H section with plugging
	Cast-in-place	Closed end concrete/steel pipes driven with mandrel, left in place and filled with concrete (Raymond, West) Recoverable casing and expanded base (Franki) Displacement screw piles (Atlas, Fundex, Omega)

etc. However, shortcomings are that they are limited in length and diameter, and cannot readily be varied in length to suit varying depths of bearing layers. They cannot be driven in soils including cobbles or very dense or cemented layers. The noise and vibrations due to driving may be unacceptable, especially in urban environments; the displacement of soil due to driving may lift adjacent piles or damage adjacent structures. They cannot be driven in conditions of low headroom.

Cast *in situ* displacement piles can be easily adjusted to suit varying depth of bearing layers; concreting is generally possible excluding ground-water, and the formation of an enlarged base is possible in some types. Noise and vibrations may be reduced by driving with an internal drop hammer. On the other hand, the concrete in the shaft is liable to be defective in soft squeezing soils or in conditions of artesian water flow, when recoverable tubing is used. The length of some types may be limited by capacity of the rig to pull out the tube casing.

Displacement screw piles have limits in diameter and length; the reinforcement cage has generally to be inserted into the fresh concrete.

Replacement piles, with the exception of CFA, generally exhibit relatively low resistance in cohesionless soils due to inevitable loosening of the soil below the base and surrounding the shaft.

The range of length and diameters available is very broad, from micropiles to large diameter shafts; the length can readily be varied to suit variation in soil profile; any kind of soil, including boulders and rock layers, may be drilled by choosing a suitable technique. Replacement piles can be installed without noise or vibration, and also in conditions of low headroom.

As for all of the cast *in situ* types, the concrete is liable to squeezing or necking in soft soils; special techniques are needed for concreting in water or mud filled holes.

Bearing Capacity of a Single Pile

Piles have rarely been used as single piles (this is changing because of the present availability of large diameter, large capacity piles) but, rather, in groups ranging from a few units for piled footings of single heavily loaded columns to large piled rafts resting on many hundreds of piles.

Design practice, however, is based on the evaluation of the behavior (bearing capacity, settlement under working load) of a single pile; the behavior of the pile groups is then extrapolated from that of the single pile.

In order to minimize the cost of the cap, the piles in a group are usually kept as close each other as possible, the only limit being the possible damage to the already installed piles. The usual spacing ranges between $2.5d$ and $3.5d$ axis to axis, d being the pile diameter.

According to the experimental evidence the settlement of a single pile under axial load at ultimate failure ranges from $0.1d$ (displacement piles) to $0.25d$ (replacement piles).

Such large displacements are rarely attained in load tests and are of little practical significance. It is generally accepted that the bearing capacity of a pile be conventionally defined as the load corresponding to a settlement equal to 10% of the pile diameter.

For the time being, one can agree with Poulos et al. (2001) that it is very difficult to recommend any single approach as being the more appropriate for estimating the axial bearing capacity of single piles. Given the very nature of this problem, the most reasonable approach seems to be developing regional design methods combining the local experience of both piling contractors and designers.

Of course, both the bearing capacity Q_{ult} and the settlement w of a single pile under the service load are best evaluated by a full scale load test on a prototype pile. Such a test, however, cannot be run at the design stage, other than in special cases, because of the high cost.

It is almost universally accepted, however, that the bearing capacity Q_{ult} of a pile can be estimated by summation of the ultimate shaft capacity S_{ult} and the ultimate base capacity P_{ult}. The weight W of the pile is subtracted from the compressive load and added to the uplift capacity. In turn, S_{ult} and P_{ult} may be expressed as a function of the ultimate unit shaft and base resistance s and p. Thus, for compression:

$$Q_{ult} = \pi d \int_0^L s dz + \frac{\pi d^2}{4} p - W \tag{4}$$

and, for uplift:

$$Q_{ult} = \pi d \int_0^L s dz + W \tag{5}$$

where L is the pile length.

Of course, the ultimate value of s is not necessarily the same in compression and uplift. Suggestions for the evaluation of p and s, on the basis of soil properties obtained by laboratory tests or in relation to in situ tests, may be found in standard textbooks on foundation engineering (Salgado 2008; Fleming et al. 2009; Viggiani et al. 2012).

Bearing Capacity of a Pile Group

The bearing capacity of a pile group, Q_{Gult}, is generally expressed as the bearing capacity Q_{ult} of the single pile multiplied by the number n of piles in the group and by a coefficient E called "efficiency" of the group:

$$Q_{Gult} = EnQ_{ult} \tag{6}$$

With the usual values of the spacing axis to axis s among piles ($2.5 \leq s/d \leq 3.5$), in cohesionless soils and for displacement piles the efficiency is always greater than unity. Even for replacement piles, in cohesionless soils the efficiency is

Foundations, Table 3 Values of the empirical parameter M (Eq. 8)

Pile type	Soil type	M
Displacement	Cohesionless Cohesive	80 120
Small displacement (driven H or tube; large stem auger pile)	Cohesionless Cohesive	50 75
Replacement	Cohesionless Cohesive	25 40

generally not less than unity. Accordingly, the efficiency can be conservatively assumed equal to unity (no group effect).

In cohesive soils and in undrained conditions, the efficiency is always less than unity. It may be evaluated either by empirical expressions as a function of the group geometry, or considering the pile group and the soil in between as a block foundation. One of the empirical expressions is the Converse-Labarre formula:

$$E = 1 - \frac{\arctan\frac{s}{d}\left[(f-1)g + (g-1)f\right]}{\frac{\pi}{2}fg} \tag{7}$$

where f and g represent the number of rows and columns of piles in the group ($fg = n$).

Settlement of a Single Pile Under Axial Load

The order of magnitude of the settlement w of an axially loaded single pile under a load $Q \leq Q_{ult}/2.5$ may be evaluated by the empirical expression:

$$w = \frac{d}{M}\frac{Q}{Q_{ult}} \tag{8}$$

Suggested values of M are listed in Table 3.

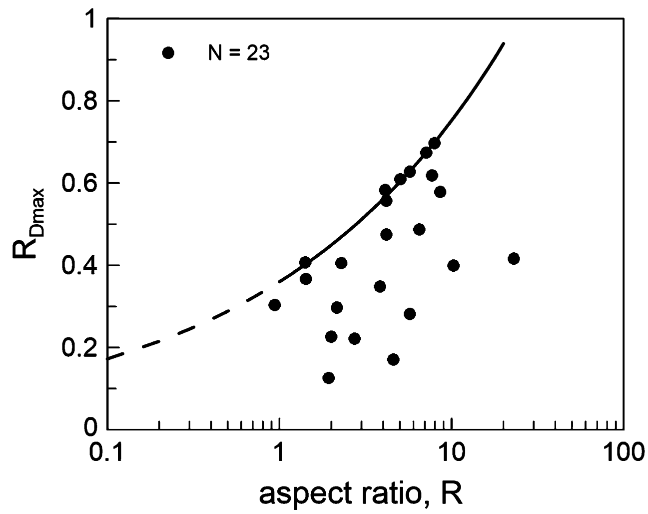
Settlement of a Pile Group

The settlement of a pile belonging to a group, and hence the settlement of the group, is always larger than the settlement of a single pile subjected to a load equal to the average load per pile of the group; this is an effect of the interaction among the piles via the surrounding soil.

The average settlement w_g of a group of n piles may be expressed as a function of the settlement w_s of a single pile subjected to a load equal to the average load per pile in the group:

$$w_g = R_s w_s = nR_G w \tag{9}$$

where w_s is the settlement of a single pile under the average working load Q/n of the group (Q = total load applied to the foundation), R_S is an amplification factor named group



Foundations, Fig. 9 Relationship between R_{Dmax} ($= \delta_{max}/w_g$) and the aspect ratio R

settlement ratio, quantifying the effects of the interaction between piles, and $R_G = R_S/n$ is the group reduction factor. It may be shown that $1 \leq R_S \leq n$ and hence $1/n \leq R_G \leq 1$.

On an empirical basis, the following expressions for the upper limit $R_{S,max}$ and the best estimate of R_S, are useful for preliminary evaluations, and have been obtained as a function of the aspect ratio $R = (ns/L)^{0.5}$:

$$R_{S,\max} = \frac{w_{g,\max}}{w_S} = \frac{0.5}{R}\left(1 + \frac{1}{3R}\right)n \tag{10}$$

$$R_S = \frac{w_g}{w_S} = 0,29nR^{-1,35} \tag{11}$$

Some experimental data concerning the maximum expected value of the differential settlement δ_{max} of pile foundations, expressed as a fraction of the average settlement w_g, are reported in Fig. 9.

Some evidence related to pile foundations in clay seems to indicate that the presence of piles reduces the maximum expected angular distortion compared to that of shallow foundations (Fig. 7). Accordingly, for pile foundations Eq. 3 may be substituted by:

$$\beta_{max} = 6.2 \times 10^{-5} w_{max} \tag{12}$$

Summary

The design of foundations depends on analytical, numerical, or empirical models using parameters representing the response of soil to stress variations imposed by applied loads. The parameters are part of the geotechnical model which is the idealization of the subsoil for the purpose of a specific calculation. There is a variety of types of foundations ranging from shallow foundations, such as strip footings or rafts, to deep piled foundations. Selection depends on the size and distribution of the imposed load and on the characteristics of the soil which determine the amount of settlement and the bearing capacity.

Cross-References

▶ Bearing Capacity
▶ Bedrock
▶ Characterization of Soils
▶ Classification of Rocks
▶ Classification of Soils
▶ Clay
▶ Cohesive Soils
▶ Collapsible Soils
▶ Compaction
▶ Compression
▶ Cone Penetrometer
▶ Consolidation
▶ Deformation
▶ Dewatering
▶ Dissolution
▶ Dynamic Compaction/Compression
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▶ Hydrocompaction
▶ Infrastructure
▶ Instrumentation
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▶ Modulus of Deformation
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▶ Quick Clay
▶ Quicksand
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▶ Soil Mechanics
▶ Soil Properties
▶ Subsidence
▶ Voids
▶ Young's Modulus
▶ Zone of Influence

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Gabions

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Definition

Box-shaped wire baskets that are filled with durable rock fragments and used as retaining walls or for erosion control.

The word “gabion” is derived from Italian “gabbione,” meaning “cage.” The early gabions were sack-shaped wire mesh containers produced by Maccaferri beginning in 1893 (Maccaferri 2016, www.maccaferri.com) for work along the *channel* of River Reno at Casalicchio de Reno near Bologna, Italy. The gabions were positioned where they were needed and filled with rock fragments. The sack shape was replaced by a box shape in 1907; the box-shaped gabions could be placed in efficient arrangements and connected to each other with wire fasteners. The primary use of gabions is for stabilizing *slopes*. Typical applications along river channels were related to *erosion* control. The rock-filled containers maintained their positions simply because of the combined mass of rock fragments act as a gravity *retaining wall*. The rock fragments without finer particles of soil were free-draining, which prevented buildup of *hydrostatic pressure* behind the containers. The initial box-shaped gabions were cubes 1 m on each side and composed of heavy wire available at the time.

Experience with gabion construction and performance led to improvements as technology and manufacturing allowed better wire to be used in a hexagonal mesh configuration; the linkages between hexagons evolved into a double twist for better performance. Some manufacturers use rectangular welded wire fabric. The single-cell box evolved into two and three-cell configurations. These are called “gabion baskets” in some places. For erosion control applications on gently inclined slopes where a relatively small mass of rock fragments would provide adequate stability, a full-height basket is inefficient. Baskets were developed that have cells 1/3 m high, which are called reno mattresses, probably because of the erosion control applications along River Reno.

Large retaining walls, exceeding 20 m high, have been constructed with gabion baskets. These configurations have a base composed of multiple rows of baskets, which requires either a larger excavation at the base of the slope being stabilized or a larger amount of imported fill if the face of the gabion basket wall is 5 or even 10 m away from the toe of slope being stabilized. The front face profile of the gabion wall can be stepped or straight, and the wall can be battered by stepping successively higher baskets back into the slope (Fig. 1) by an amount determined with stability analyses or by tilting the entire wall a few degrees (e.g., 5°). Stability is enhanced by linking successively higher rows of baskets to the lower row by threading a wire that lashes the baskets together. Geofabric is almost always used to prevent migration of fine soil particles of the slope being stabilized into the voids between the rock fragments in the gabion baskets. The rock fragments used to fill the gabion baskets must be durable so that they do not break down with cycles of wetting-drying, heating-cooling, or freezing-thawing. Rounded to subrounded fragment shapes are preferred to angular shapes to minimize scratching of corrosion-resistant coatings on the gabion basket fabric.

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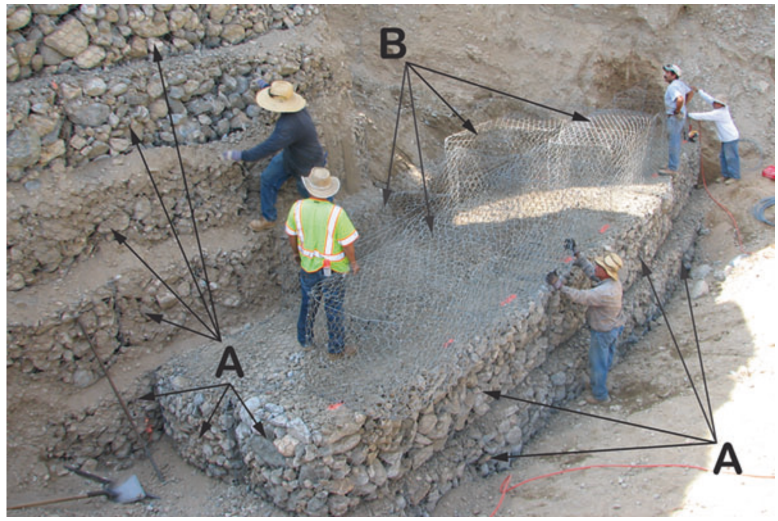

Gabions, Fig. 1 Stacked and stepped gabion baskets for a 10-m-high wall being constructed. (**a**) baskets 1 m × 1 m × 3 m filled with durable rounded rock fragments; (**b**) empty baskets being positioned near the base of the wall

Cross-References

- ► Aggregate Tests
- ► Armor Stone
- ► Current Action
- ► Erosion
- ► Retaining Structures
- ► Stabilization

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Gases

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Definition

Elements and compounds in a dispersed physical state that do not resist changes to shape and can expand indefinitely to fill any container. The term is normally restricted to substances that are gaseous at ambient atmospheric temperatures and pressures (but volatile liquids, under ambient conditions, can produce vapours that enter mixtures of gases).

Synonyms

Airborne compounds; Atmospheric elements; Emissions; Vapours

Introduction

Gases of significance in engineering geology are potentially hazardous and can occur in significant amounts in underground cavities (natural and man-made), soils, landfills, and constructions at ambient temperatures and pressures. These normally occur as mixtures with atmospheric gases. Liquids that are volatile in surface and near-surface conditions (especially volatile organic compounds – VOCs) also enter mixtures of gases especially during heat and pressure changes. Therefore, hazardous "gases" originate from natural processes, human activities, or interactions between these. Gases are not usually problematic if they diffuse into the atmosphere (although there can be exceptions such as evolved methane from swamps that contributes to global warming). Problems arise if gases are trapped and there is inadequate ventilation. Gases may be retained in the soil when atmospheric pressure is high and vented into the atmosphere when pressure is low.

Methane (CH_4) is generated during decomposition of organic materials in natural hydrocarbon reservoirs, coal mines, peat deposits, and landfills. It is less dense than air and readily migrates to levels with lower concentrations or by flow due to differential pressures. The rate of flow is greater in highly permeable ground (e.g., sand and gravel) or underground voids (e.g., caves, mines, tunnels, sewers, and underground service pipes). Gas can travel for long distances vertically and horizontally underground until vented into the atmosphere, accumulating in voids or behind impermeable barriers. Accumulations are explosively flammable in the range 5–15% by volume in air. Methane has low solubility in water but migration of the gas in solution can still be significant (NHBC and RSK Group 2007).

Carbon dioxide (CO_2) is generated during reaction of limestone with acidic rain or groundwater and during decomposition of organic materials in landfills (Powlowska 2014). It is toxic or asphyxiating depending on concentration and exposure. It is denser than air and can accumulate in the lower parts of underground cavities and structures, with a layer of breathable air above, until it fills them. This is a serious hazard to workers because it is possible to breathe easily until bending down when loss of consciousness and death can happen quickly. CO_2 emissions are also known from certain volcanoes and have caused loss of life when the gas flows down-slope as a heavier than air layer (NHBC and RSK Group 2007).

Radon (Rn) is a radioactive gaseous element in the decay series that alters uranium or thorium into lead. It is colorless and odorless and has a short half-life. The precursor is radium. After decay, the daughter element is polonium, which is a radioactive solid. Therefore, if radon gas enters the lungs and decays, the resulting polonium particles lodge in the lungs emitting carcinogenic radiation. Principal sources of radon are: uranium, radium, and thorium-rich igneous rocks (e.g., some granite and mineral veins); uraniferous black shale; phosphatic deposits; some coal and peat; and gravel deposits that contain significant amounts of debris from those sources. Radon emissions are a significant hazard only if the gas enters confined and poorly ventilated spaces that are consistently used by the same individuals (e.g., mines, tunnels, basements, and dwellings constructed from radioactive building stone). The gas can enter directly from bedrock or through porous cover deposits, but the hazard is easily reduced by good ventilation or barriers to migration. The potential hazard can be assessed by monitoring levels in buildings and cavities in relation to a defined action level (e.g., 200 Becquerels per cubic metre in the UK) (Appleton et al. 2000; Health and Safety Executive 2011).

Air Depleted in Oxygen (known to some miners as blackdamp, choke damp, or stythe) is a colorless, odorless, mixture of gases (mainly nitrogen, carbon dioxide, and water vapour) resulting from removal of oxygen during oxidation processes until respiration cannot be supported. This can be present in mines, tunnels, sewers, and wells.

Other Hazardous Gases

These usually occur in insufficient concentrations to cause major problems but can be dangerous. These include:

- Carbon monoxide (CO) from combustion (asphyxiating)
- Hydrogen sulfide (H_2S) from anaerobic bacterial decomposition of organic materials (toxic)
- Di-Hydrogen (H_2) from combustion in landfills (inflammable)
- Sulfur dioxide (SO_2), fluorine, and chlorine, most often associated with volcanic processes (strongly corrosive and causing damage to monitoring devices);

VOCs

There are many sources of anthropogenic VOCs. The main issues are associated with landfills and contaminated land, particularly former coal-burning gasworks. Solvents, paints, cleaning products, refrigerants, benzene, adhesive removers, and aerosols are potential hazards.

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Conclusions

Some gases and VOCs present hazards to people, the environment, and development. Therefore, it is important to:

- Consider potential gas problems in spatial planning and environmental assessment.
- Undertake appropriate detection and monitoring of gases before any people enter confined spaces in which these might have accumulated and before any development (especially if underground spaces are involved)
- Undertake careful monitoring and treatment of gas generating entities such as landfills during use and after closure to meet local regulatory limits

Cross-References

▶ Contamination
▶ Geohazards
▶ Landfill
▶ Mining Hazards

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Geochemistry

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Definition

The science of the chemical composition, chemical processes, and chemical evolution of the Earth.

The Earth consists of chemical elements that mainly formed before the existence of the Earth. During the formation and evolution of the planet and subsequent geological history, these have become redistributed and, in the case of radioactive elements, have partly transmuted into other elements. Initially, redistribution depended wholly on inorganic processes, but once living organisms appeared, they played an increasing part in geochemical processes on land and in surface and groundwater. In recent times, human activities have increasingly affected processes in the shallow Earth's Crust, water, and the atmosphere. Therefore, geochemistry was initially focused on the geological past but has become increasingly important in respect to geological resources and environment protection. The theoretical basis of geochemistry is physical and chemical laws governing the behavior of matter in the ranges of thermodynamic conditions that occur in the geosphere.

Development of Geochemistry

The term "Geochemistry" was first used in 1838 by C.F. Schönbein (Switzerland) for the science of the chemical processes in the Earth's crust, and was widely recognized after the work by V.I. Vernadsky (Russia) (Vernadsky 1954–1960). Other key founders of the discipline were V.M. Goldschmidt (Norway) (Goldschmidt, 1954), F.W. Clarke (USA), who presented the first major summary of the subject, and A.E. Fersman (Russia). It was Fersman who introduced the first high-level University course in the discipline (see Fersman 1934 and 1952–1962). These pioneers focused on the relative abundances of chemical elements and their isotopes, the patterns of distribution of chemical elements in the geosphere, and laws of the behavior of chemical elements in natural processes. To some extent, a parallel development of this science within the former Soviet Union and in Western Europe/USA and some other countries led to some differences in terminology. More recently, these traditions have converged.

Main Aspects of the Discipline

Modern geochemistry is a combination of scientific disciplines, united by common approaches and specific research methods, applied to the Earth's main accessible layers – the lithosphere, atmosphere, hydrosphere, and biosphere – and also, from indirect evidence, to deeper levels. However, organic geochemistry focuses on conditions of accumulation and the geochemical role of once living organic matter. Natural geochemical processes can be divided into endogenous (magmatic, hydrothermal, metamorphic) and exogenous (supergene) processes.

The main aspects of the discipline are the distribution, history, and behavior of the chemical elements and their isotopes in our planet, including migration, transport, and concentration (accumulation) of chemical elements and transitions from one state to another. The main theoretical problem is to ascertain the behavior of chemical elements in the geosphere. Any geological process is accompanied by accumulation of some elements and isotopes and migration and dispersal of others. From that point of view, geochemistry reflects processes of separation due to differences in physical and chemical properties and the nature of the environments in which they are migrating or accumulate. This is reflected in the geochemical classification of elements – the Law of Matter Differentiation of the Earth – which states that the distributions, laterally and vertically of chemical elements, define the distinctive chemical compositions of geological structural units (Goldschmidt 1954).

Geochemistry is based on the concept of chemical elements dispersion in the Earth's space, which is reflected in Vernadsky's Law on the abundance of chemical elements in any volume of terrestrial matter. The current geochemical field is a transitional stage in the long process of redistribution of elements by continuous migration in time and space. F W Clarke (1924) developed the concept of identifying the average percentages of elements in the Earth's crust (estimates known as "clarkes").

Geochemical provinces from planetary to regional scales can be distinguished. These provinces have fairly homogeneous geochemical characteristics that reflect the types of mineralization and other geochemical associations that have developed, in irregular stages, through geological time (Tan and Chi-Lung, 2009). Migration for most elements is cyclical, including the passage of elements from one part of the geosphere to another during geologic time. Migration in the biosphere is carried out partly or wholly with the participation of living organisms. Certain elements are absorbed by living matter during the biological cycle but, on leaving the living matter, the atoms give up stored energy to the environment. Many chemical reactions reflect this biogenic energy, for instance, reduction-oxidation (redox) zoning systems.

The study of the nature of, and changes to patterns of lithogenesis of sedimentary rocks, describes the composition of the past and present atmosphere, hydrosphere, and lithosphere and geochemical factors in the origin and development of life, environments, and habitats. It involves the chemical composition and physicochemical conditions of sedimentary rocks; their evolution in the geological history; the abundance of elements in sedimentary rocks, patterns of behavior, distribution, and migration of elements; and their associations in the processes of sedimentation, diagenesis of sediments, and epigenesis. Lithogenesis of igneous rocks involves the partitioning of elements during cooling of magmas and extends into the hydrothermal processes that lead to the formation and zoning of ore bodies. The genesis of metamorphic rocks is described in terms of the migration and accumulation of elements under ambient temperature and pressure conditions.

Principal Research Tasks

Key tasks have been and are:

- Determining the abundances of chemical elements and average composition of the Earth's crust
- Determining patterns of distribution in the Earth as a whole, and in its main layers, including rocks, minerals, soils, water, living organisms, and man-made systems including natural and anthropogenic concentrations of potentially harmful elements
- Establishing migration behaviors of chemical elements leading to their concentration or dispersal that determine the formation of rocks and mineral deposits
- Understanding geochemical and biogeochemical cycles
- Identifying characteristics of geochemical provinces and landscape zones, including mineral deposits associated with them
- Determining chemical composition changes in the biosphere due to both natural processes and human impacts
- Identifying deficiencies of mineral nutrients that affect both agriculture and people
- Using chemical compositions to identify the sources of transported materials such as dust
- Geochemical mapping and zoning
- Improving methods of physical and chemical analysis of the paragenesis of chemical elements
- Securing quantitative data on the content, distribution, and forms of chemical elements and isotopes and their behavior in all relevant systems
- Developing better mathematical methods of data processing and analysis

Subdisciplines

Names have appeared for subdisciplines, notably:

- Physical geochemistry – physicochemical processes of formation of minerals, rocks, and ores in the Earth's crust, mantle, atmosphere, and hydrosphere
- Thermobarogeochemistry – heat and pressure-related physicochemical processes
- Geochemical ecology – interactions of organisms and their communities with the geochemical environment
- Agro-geochemistry – natural and anthropogenic patterns of change in the geochemical properties of pedological soils and the impacts on quality and quantity of agricultural production
- Isotope geochemistry – behavior of isotopes under the influence of various geological, geochemical, and cosmochemical processes and development of criteria for the use of these data for solving theoretical and applied problems including the reconstruction of the most important events in the history of the elements and, therefore, in the history of the Earth's crust and the Earth as a whole, as well as meteorites and the solar system
- Environmental geochemistry – behavior of chemical elements in the environment including consideration of contamination and pollution and impacts on people and ecosystems
- Landscape geochemistry – concentrations and behavior of chemical elements in landscape zones (terrain units)
- Regional geochemistry – characteristics of specific areas from the large to small scales including geochemical terrain units (geochemical zoning) that can be used in forecasting the existence of, and prospecting for, mineral deposits and also for agricultural and medical assessments
- Radiogeochemistry – types and concentrations of radioactive elements and isotopes in geological processes and is the basis for: exploration for radioactive ores; understanding the energy processes crust associated with radioactivity; determination of the absolute age of rocks, fossils, and minerals and archaeological remains from the accumulation of decay products of radioactive isotopes at known rates of decay; and assessment hazards from radioactive elements deposited from the air (e.g., from plumes emitted during nuclear accidents) or emissions from the ground (e.g., radon)

Relevance to Engineering Geologists

Aspects of geochemistry that are relevant to the work of engineering geologists relate to an understanding of the compositions and diagenesis of soils and rocks; processes of mineral resource formation; contaminants and pollutants; and behavior of construction and building materials. Some aspects are direct but an understanding of other disciplines is required when working alongside hydrologists and environmental scientists.

The established patterns of distribution and concentrations of chemical elements in geological processes form the theoretical basis of identification of prospects for, and commercial exploration of, mineral deposits. Understanding the principles and mechanisms of formation of primary and secondary halos and dispersion trains in and around ore bodies and chemical indications in soils are the basis of geochemical prospecting.

Since the late eighteenth century, increasing industrial activity has significantly affected the shallow geosphere and biosphere in many parts of the World and effects continue to increase. This has caused a sharp change in the scale of geochemical cycling of substances and increased entry of toxic components into the biosphere. Fersman pointed out the inevitability of destabilization of the Earth's biosphere by creating manufactured chemical compounds that generate reactions that are unusual in nature and change the flow rate in many geological and geochemical processes.

Extraction of minerals and civil engineering operations has become comparable in scale to natural processes. Pollution and contamination of soils and water by solid and liquid industrial and municipal wastes, fertilizers, and pesticides has become widespread. The problems of anthropogenic (technogenic) changes were first identified by A.E. Fersman who stated that by developing industry during a historically short period mankind has become a major geochemical agent that is beginning to change chemical elements in nature and subdue the substance of nature to his will.

Geochemistry is important to elucidating the role of these processes in the current and future chemical evolution of the biosphere, essentially geoecological research. Regaining a geochemical equilibrium will require rational development of land and mineral resources and protection of the environment from human activities. That requires field observations, field and laboratory experiments, modeling, and analytical calculations. In recent decades, geochemistry and biogeochemistry have been increasingly important in environmental protection by establishing baseline data for assessment of levels of anthropogenic pollution and contamination and assessing rates and extent. Work has also extended to the study of elements and compounds in the biosphere in support of studies of health impacts, levels of organic nutrients, and has become important in respect of biosphere evolution forecasts and rational measures of environmental protection from pollution. It is also important in forensic geology.

Conclusion

Three main research directions are distinguished in geochemistry, consistently prevailing over three major periods of modern geochemistry development (Jaroshevsky 2012). The first direction (mainly in the first half of the twentieth century) focused on the formulation and solution of fundamental problems of geochemistry, namely, the assessment of the abundance and establishing the main regularities in chemical element partitioning in the natural objects. The second direction (in the second half of the twentieth century) is connected with the development of physicochemical principles for interpreting the behavior of chemical elements in natural processes. The main result in this research was the substantiation of the fruitfulness of thermodynamic and dynamic modeling of natural systems and processes. The third direction (in the second half of the twentieth century – the beginning of the twenty-first century) is the creation of physicochemical models for the behavior of chemical elements in geological processes and, on this basis, the development of geochemical criteria for solving geological problems. Making further progress in geochemistry requires the perfection of mineral matter analysis methods and widespread computer technology use in the geochemical studies.

Cross-References

- ▶ Contamination
- ▶ Diagenesis
- ▶ Igneous rocks
- ▶ Metamorphic rocks
- ▶ Mineralization
- ▶ Pollution
- ▶ Sedimentary rocks
- ▶ Site investigation

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Geohazards

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Synonyms

Geochemical hazards; Geoenvironmental hazards; Geological hazards; Geomorphological hazards; Geophysical hazards; Geotechnical hazards; Hydrogeological hazards; Mineralogical hazards

Definition

A geohazard is a geological source of danger.

There are many different types of geohazard with different natural and artificial processes causing them to occur. However, they all have the potential to create problems for development of the human environment and threats to the safety and well-being of people. Geohazards can develop quickly (seconds or minutes) in response to the processes that drive them, or take tens, hundreds, or thousands of years to develop to a point where they pose a danger. They are found in most parts of the world, including marine and fluvial environments. Their existence is an indication of the continuing evolution of the solid Earth as they are part of the processes by which rocks are eroded and moved and then recreated by deposition, burial, alteration as a result of increased temperature and pressure and brought back to the Earth's surface.

Hazard

The Encyclopedia of Natural Hazards defined "hazard" as *"... an event, phenomenon, process, situation or activity that may potentially be harmful to the affected population and damaging to the society and the environment. A hazard is characterised by its location, magnitude, geometry, frequency or probability or occurrence and other characteristics."* (Nadim 2013). It is the job of Earth scientists, in general, and engineering geologists, in particular, to define these various criteria as they relate to different geohazards. Indeed, since the growth of engineering geology as a subdisciple of geology began in the 1960s, engineering geologists have spent much time developing internationally-recognized classifications to quantify some of these characteristics for the different geohazards. For example, 32 types of mass movement features have been defined (Varnes 1978; Hungr et al. 2014) and sinkholes/dolines have been subdivided into 6 types (Waltham and Fookes 2003). However, Nadim (2013) pointed out that the term "hazard" is also used to *"... describe the temporal probability of occurrence of the event or situation in question."*

The term "geohazard" fits the former definition of the term "hazard" in that it refers to something that actually happens rather than to the probability of occurrence.

Classification of Geohazards

In terms of their classification, geohazards can be subdivided into three main groups.

Primary Natural Geohazards

Primary geohazards such as earthquakes and volcanic eruptions:

- *Appear* to be cyclical in their occurrence
- Have return periods determined by analysis of past events
- Affect regions
- Are controlled by *regional* geology
- Are generally unpredictable as the geological processes are not well enough understood
- Are currently almost impossible to prevent
- Are best mitigated by engineering design and focus on secondary geohazards

Primary geohazards are represented generally as the probability of a certain magnitude event occurring. The problem with this approach is that the probability of occurrence is based on data from very short (in geological terms) periods of time. Earthquake magnitude is a quantitative measurement of the size of an earthquake based on the seismic energy produced. The Richter Magnitude Scale is used for this. This is a logarithmic scale, so a magnitude 7 earthquake, for example, is 10 times larger than a magnitude 6 earthquake. The intensity of the earthquake event is classified descriptively in terms of the severity of the shaking that occurs. The classes of intensity in the Modified Mercalli Intensity Scale are based on how strongly the shaking is *felt* at the lower end of the scale and the amount of damage caused at the upper end.

Secondary hazards arising from earthquake occurrence include ground subsidence, ground rupture, ground motion amplification, liquefaction, and tsunamis, whereas those arising from volcanic eruptions include dome collapse, pyroclastic flows, lahars, debris flows and avalanches, lava flows, ash and tephra falls and larger projectiles, gases, and tsunamis.

Climate is another primary hazard that can trigger secondary geohazards. Landslides are an obvious example of this frequently being triggered by periods of intense and/or prolonged rainfall. Shrinkage of clay-rich soils is usually triggered by periods of drought, as is desiccation.

Human activity (see below) can also be considered as a primary hazard, for example, in relation to ground contamination and groundwater pollution, mining subsidence and collapse, some flooding and abandoned buried structures, and building foundations.

Secondary Natural Geohazards

Secondary geohazards such as landslides and dissolution:

- Are often triggered by primary hazards
- Have return periods that are difficult to determine by analysis of past events because of limited data and non steady-state conditions
- Affect sites and districts
- Are controlled by *local* geology
- Are partially "predictable" from an understanding of geological processes
- Can be controlled to some degree
- Are best mitigated by land-use planning, insurance, and site-specific engineering

The hazard can be represented deterministically because the geological processes that control these geohazards are relatively well understood. Whereas records of past events are often relatively sparse, covering short periods of time, evidence for events for some geohazards that took place tens of thousands of years ago can be observed.

Geohazards Caused by Human Activity

There are four broad causal types:

- Extraction of minerals and its after-effects
- Engineering activities on, or just below, the ground surface
- Alteration of surface water or groundwater conditions
- Waste materials placed in, or on, the ground

Some of these geohazards have very little geological control unless events in the "Anthropocene" are regarded as being geologically as well as human-controlled.

Classification by Causative Process

The many types of geohazard can also be classified depending upon the processes that trigger a geohazard or the properties of the materials that can be hazardous. Broadly, scientists have considered the following processes and properties when grouping geohazards:

- Geomorphological hazards – triggered by near-surface natural processes such as excessive rainfall and the presence of over-steepened slopes
- Geotechnical hazards – where the geotechnical and/or mineralogical properties of a material make that material susceptible to being hazardous
- Hydrological or hydrogeological hazards – where the movement of surface or groundwater results in a hazardous condition
- Geological hazards – those resulting directly or indirectly mainly from volcanic eruptions or earthquakes
- Marine hazards – resulting from the escape of fluid or gas or the movement of parts of the seabed
- Artificial hazards – resulting from human activity in the placing or removal of physical or chemical solid materials, fluids or gases

It is interesting to note that none of these classes of geohazard includes the term engineering geology even though engineering geologists are involved in the investigation of all of the types of geohazard. This is probably because while, say, geomorphologists may not regard themselves as competent to investigate geotechnical geohazards, practicing engineering geologists would regard themselves as having to be competent to investigate all types of geohazard.

Individual geohazards are discussed in more detail elsewhere in this encyclopedia and, so, are not covered further here. However, the different types of geohazards are grouped by causative process below.

Geomorphological hazards

- Aeolian soils (loess)
- Dissolution (karst, sinkholes, voids, etc.)
- Erosion
- Desiccation
- Mass movement features (avalanches, cambering, landslides, etc.)
- Permafrost

Geotechnical hazards

- Acidity
- Collapsible soils
- Compressible soils
- Dispersive soils
- Expansive soils
- Quick clay
- Saline soils
- Residual soils

Hydrological or hydrogeological hazards

- Groundwater level change
- Floods

Geological hazards

- Earthquakes
- Earthquake intensity

- Earthquake magnitude
- Faults
- Ground motion amplification
- Ground shaking
- Liquefaction
- Ground subsidence
- Surface rupture
- Tsunamis
- Volcanic eruptions
- Dome collapse
- Pyroclastic flows, lahars
- Debris flows, avalanches
- Lava flows
- Ash, tephra falls
- Large projectiles
- Gases

Marine hazards

- Coastal erosion
- Submarine landslides
- Fluid escape features (such as liquefaction)
- Gas release (such as from gas hydrates)
- Scour
- Turbidity currents

Artificial hazards

- Acid mine drainage
- Artificial ground
- Brownfield sites
- Contamination
- Landfill
- Mining hazards (ground subsidence and collapse)
- Pollution
- Unfilled, partially filled, and filled excavations and voids

Sources of Information About Geohazards: Maps and Databases

Identification of actual or potential geohazards is part of the site investigation process. For actual geohazards, mapping is the main way in which they are identified and their character and location recorded. The terms "maps" and "mapping" here include analogue and digital and two, three, and four-dimensional models. Geohazard mapping is discussed in detail elsewhere in this encyclopedia.

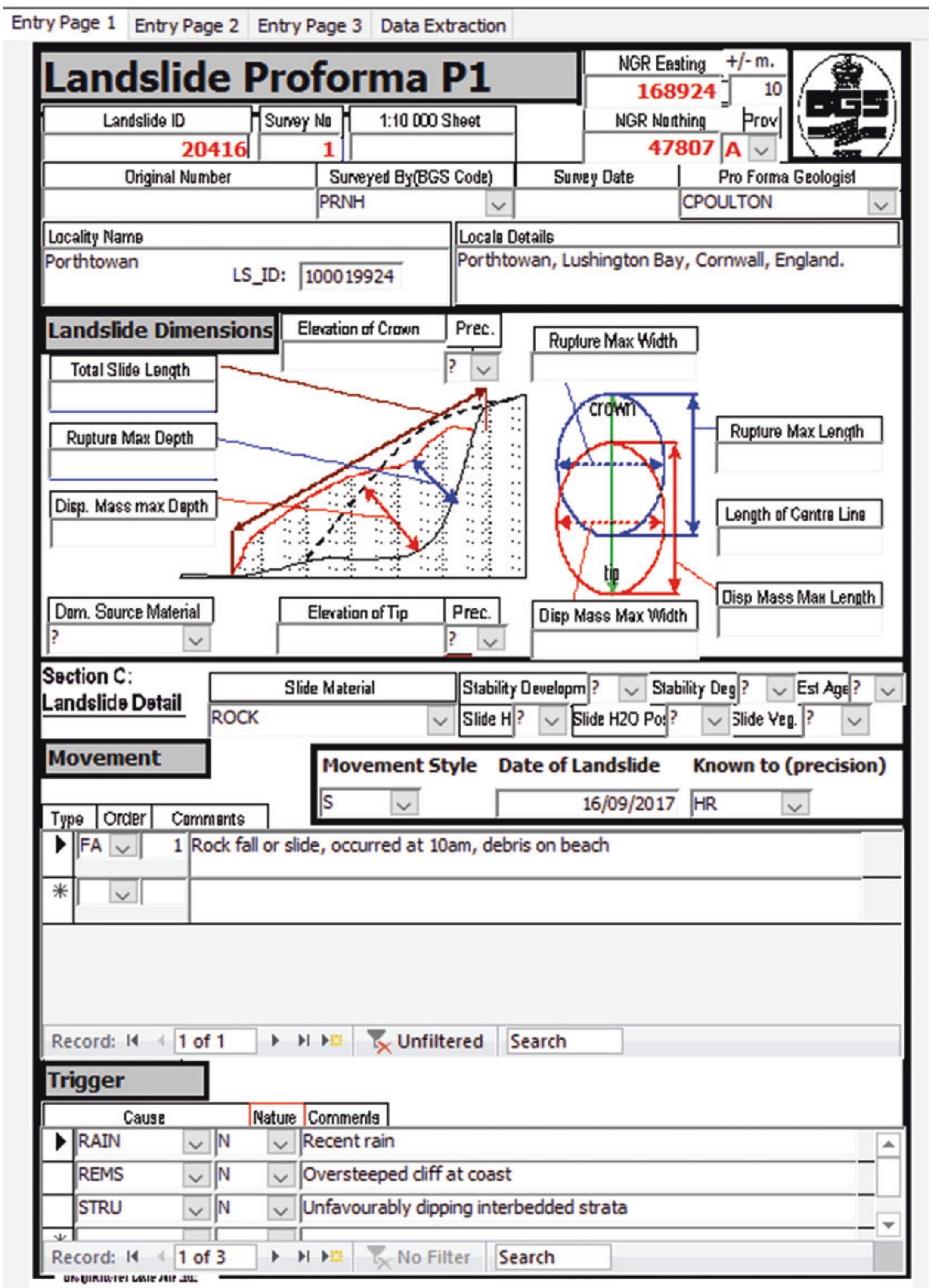
Spatial geohazard information is usually presented in one, or more, of the following four ways:

- **Geohazard inventories**: These include lists, databases, and "maps" defining or showing the location of geohazards that have occurred previously and defining their characteristics. The most common of these are landslide inventories. In these, the landslides will be classified by type, spatially located with their dimensions defined, geological conditions given, and failure processes discussed. Foster et al. (2012) and Pennington et al. (2015) described the National Landslide Database that covers Great Britain. Figure 1a, b shows a typical database entry from the database for a landslide in Porthtowan, Cornwall, south west England in September 2017. As there are no international standards for such databases, they are likely to vary depending upon whether they are national, regional, or local in their coverage, and how the geology and topography varies. These and other factors control the number of landslides in a particular area. This is illustrated by the difference between Great Britain and Italy. In the former, over 17,000 landslides have been recorded (though the database is not yet "complete") whereas the Italian landslide inventory has over 600,000 entries. This reflects the greater landslide activity in Italy caused by seismicity, rainfall, and topography compared with Great Britain which is a much lower risk environment Gibson et al. (2013).
- **Geohazard susceptibility maps**: This type of information involves identifying a range of natural and artificial factors that may increase the likelihood of a landslide occurring. The main factors include lithostratigraphy, geological structure, slope steepness, and surface- and groundwater conditions. However, a large range of other factors might be analyzed. For example, Meng et al. (2016) included ten factors (landslide inventory, slope angle, slope aspect, altitude, profile curvature, geology and lithology, distance from faults, distance from rivers, distance from roads, and vegetation coverage) for landslide susceptibility mapping in the Wolong National Nature Reserve, Sichuan, in south west China, whereas Dagdelenier et al. (2016) included ten topographical factors as well as lithological and land cover (vegetation) in a landslide susceptibility mapping study in the Gallipoli Peninsula of western Turkey. Often these types of studies use a variety of statistical and geostatistical methods to analyze the various factors. As a result, it can be quite difficult to compare different studies.
- **Geohazard maps**: These have been discussed in detail in the section on engineering geological maps. However, difficulties arise in distinguishing between maps that show geohazards by their occurrence – "*. . . an event, phenomenon, process, situation or activity. . .*" (Nadim 2013) and maps showing the *probability* of their occurrence. It should always be remembered that it is very difficult to determine true probability of occurrence if either the period of time for which geohazard occurrence data are available is shorter than the time for which the current environmental conditions have existed or if environmental conditions have changed during the period

for which data are gathered. In either case, the assessment of probability of occurrence is likely to be inaccurate.

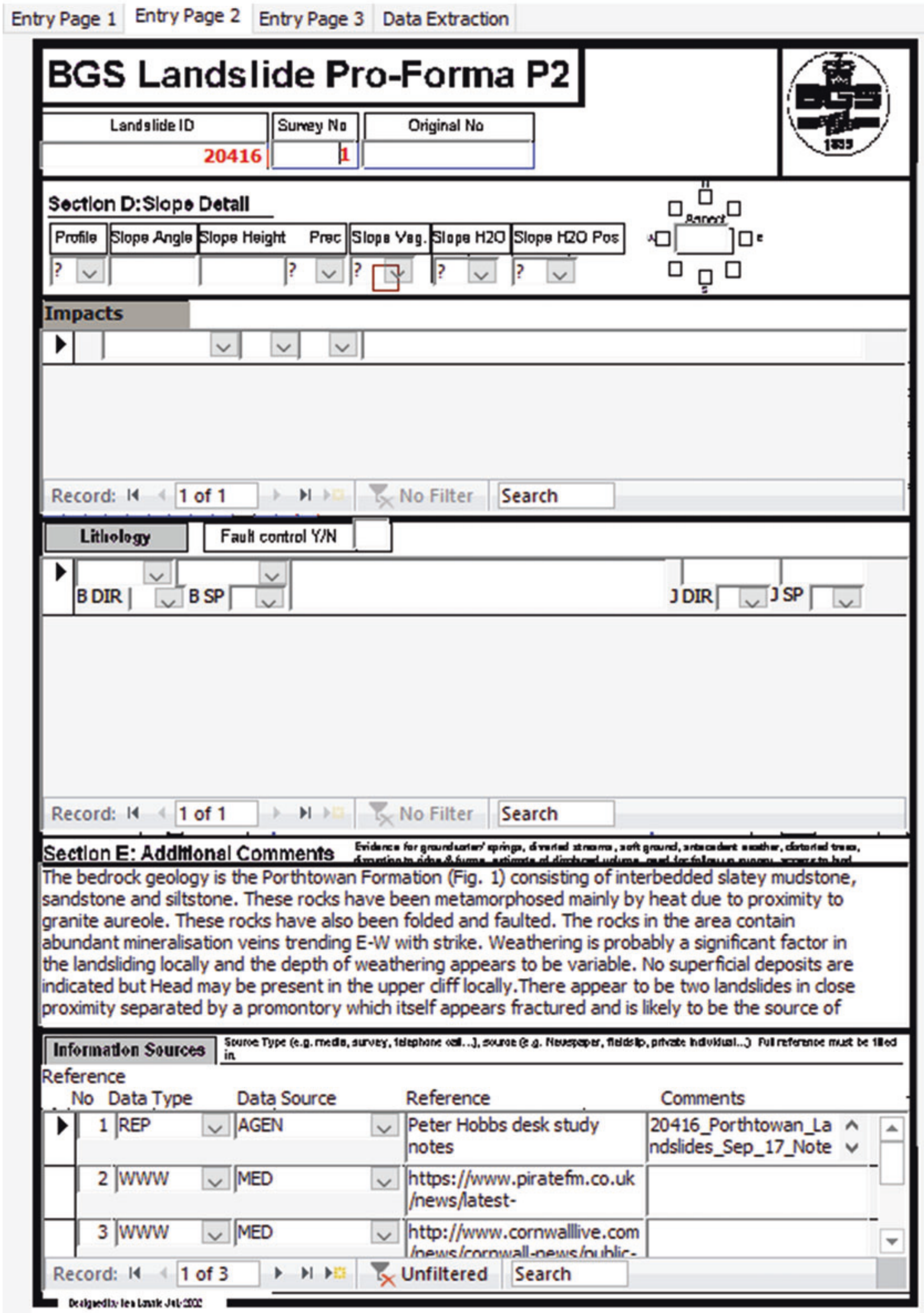
- Georisk maps: These, too, are discussed in the section on "engineering geological maps" and are not discussed further here except to note that true georisk maps are rare in the literature and that van Westen et al. (2006), in relation to landslides, have questioned whether such maps are needed at all.

a



Geohazards, Fig. 1 (continued)

b



Entry Page 1 | Entry Page 2 | Entry Page 3 | Data Extraction

BGS Landslide Pro-Forma P2

Landslide ID	Survey No	Original No
20416	1	

Section D:Slope Detail

Profile	Slope Angle	Slope Height	Prec	Slope Veg.	Slope H2O	Slope H2O Pos
?			?	?	?	?

Aspect

Impacts

Record: 1 of 1 No Filter Search

Lithology Fault control Y/N

B DIR B SP J DIR J SP

Record: 1 of 1 No Filter Search

Section E: Additional Comments

The bedrock geology is the Porthtowan Formation (Fig. 1) consisting of interbedded slatey mudstone, sandstone and siltstone. These rocks have been metamorphosed mainly by heat due to proximity to granite aureole. These rocks have also been folded and faulted. The rocks in the area contain abundant mineralisation veins trending E-W with strike. Weathering is probably a significant factor in the landsliding locally and the depth of weathering appears to be variable. No superficial deposits are indicated but Head may be present in the upper cliff locally.There appear to be two landslides in close proximity separated by a promontory which itself appears fractured and is likely to be the source of

Information Sources Source Type (e.g. media, survey, telephone call...), source (e.g. Newspaper, fieldslip, private individual...) Full reference must be filled in

Reference

No	Data Type	Data Source	Reference	Comments
1	REP	AGEN	Peter Hobbs desk study notes	20416_Porthtowan_Landslides_Sep_17_Note
2	WWW	MED	https://www.piratefm.co.uk/news/latest-	
3	WWW	MED	http://www.cornwalllive.com/news/cornwall-news/public-	

Record: 1 of 3 Unfiltered Search

Geohazards, Fig. 1 (**a** and **b**) Data entry sheets for the Great Britain National Landslide Database for a September 2017 landslide in south west England

Geohazard Investigation

In a way, it is misleading to consider the investigation of geohazards as a separate activity. Rather, the site investigation process should be carried out to determine the ground conditions in relation to the engineering (or other) activity that is to take place in the location of interest. For example, in the investigation of a local site for the construction of light structures (such as houses or small factory units or low cost roads), the main objectives of the site investigation will be to determine the geological and hydrogeological variability of the site, the geotechnical and hydrogeological properties of the materials on it, and the presence of any geohazards that may affect it. The information gathered during the site investigation process should be incorporated into the developing ground model for the site (Parry et al. 2014). Of course, if no geohazards are present, then the site will be unaffected by geohazards!

The wide range of direct and indirect site investigation techniques are discussed at length in national and international standards (e.g., EN1997-2 2007), textbooks (e.g., González de Vallejo and Ferrer 2011), and academic papers. However, in recent years, a number of indirect techniques have been developed that have applications to the investigation of some types of geohazard.

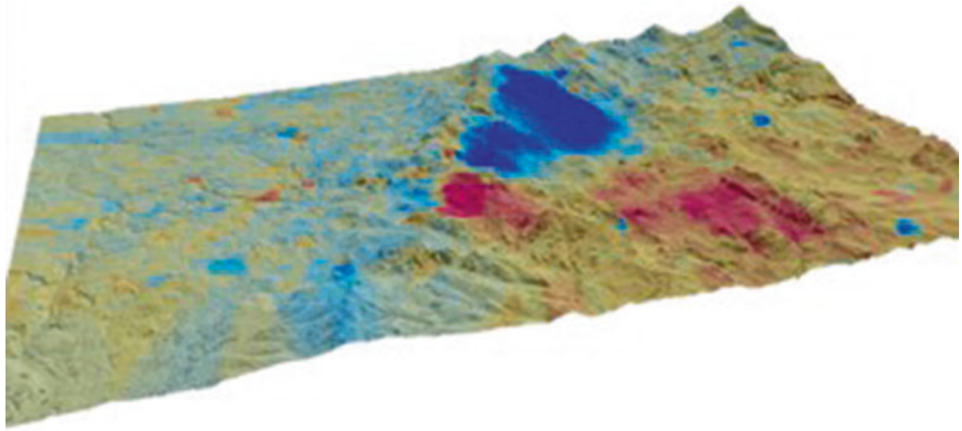
For example, Uhlemann et al. (2016a) have developed the use of electrical resistivity tomography (ERT) techniques to provide a 4D landslide monitoring system. The subsurface structure of the landslide can be observed and changes to the groundwater conditions, which may be a precursor to reactivation of a landslide, monitored. ERT techniques were also used by Barron et al. (2016) to identify gulls in cambered strata in the Cotswold Hills escarpment of Gloucestershire, south west England. The graben-like structures were at least 80 m deep and partially infilled. Uhlemann et al. (2016b) also applied a range of monitoring methods to a landslide complex in the Jurassic, Lias Group, Whitby Mudstone Formation in North Yorkshire, UK over a 5 year period. The methods used to monitor movement and other environmental changes included Real Time Kinematic-GPS (RTK-GPS) monitoring of a ground surface marker array, conventional inclinometers, Shape Acceleration Arrays (SAA), tilt meters, active waveguides with Acoustic Emission (AE) monitoring, and piezometers. The authors showed that the use of a range of monitoring techniques with high spatial and temporal resolutions was essential to provide a good understanding of the movement of the slope. P-wave refraction methods have been used to monitor seasonal effects in potentially unstable nineteenth century embankments and cuttings (Bergamo et al. 2016).

Terrestrial laser scanning has been used to monitor volume change in landslides and cliff top recession. Multiple scans are made over a period of time from a distance of 100–150 m and changes in the nature of the ground surface can be identified. This may show where soil/rock is being removed from a slope or cliff (e.g., Hobbs et al. 2002). The technique can also be used in other situations where changes in the ground shape/profile are taking place (e.g., volcanoes; Hunter et al. 2003). Feng and Röshoff (2015) reviewed the use of 3D laser scanning in rock mechanics and rock engineering and concluded that the technique can provide the "*. . .quick capture of an object in 3D with large coverage and high resolution without illumination.*" Their main concern was with regard to the development of appropriate software applicable to rock mechanics.

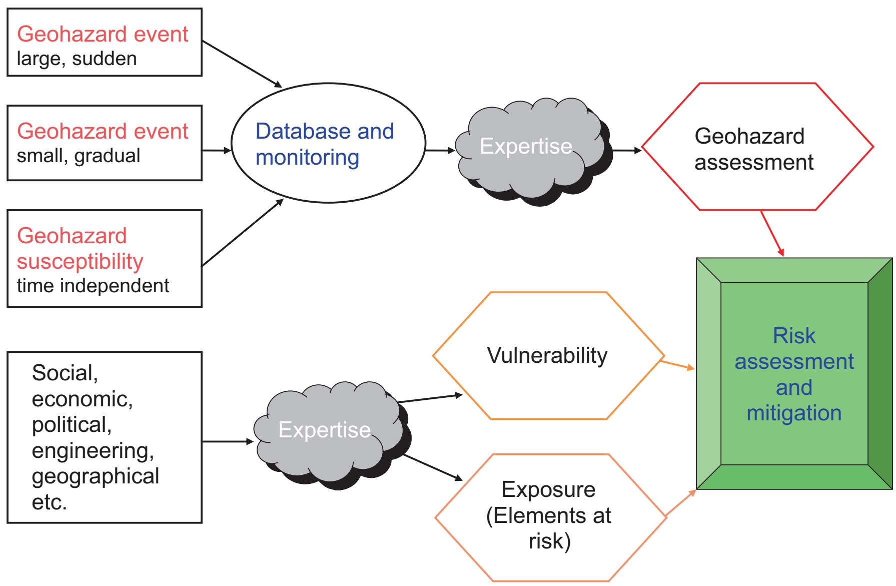
With the rapid development of various new satellite imagery techniques, these are being increasingly used both to identify geohazards on the Earth's surface and to monitor changes in elevation of the ground surface. The Synthetic Aperture Radar system (SAR), which is a microwave imaging system, looks at the same target from different positions. The system allows the accurate measurement of the travel path of the electromagnetic radiation transmitted from a satellite and the reflection received back. Initially, the data were used to investigate the potential of SAR interferometry for digital elevation modeling (DEM) and ground surface movement monitoring. An alternative method of processing SAR data is known as the Permanent Scatterers technique (PSInSAR). In this method, radar reflections from millions of points on the ground (for a 100×100 km^2 scene) are combined for repeat passes of the satellite. Over time, changes in position of points on the ground can be monitored. Culshaw et al. (2009) used PSInSAR data for The Potteries area of central England and showed that both subsidence and uplift have been taking place over an area of former coal mineworkings. Uplift seemed to be associated with the area of older mining in the northern Potteries where coal was extracted by partial extraction methods and where groundwater levels have returned towards their "natural" levels. Subsidence was observed in the southern Potteries where deeper mining using total extraction methods ended much more recently (Fig. 2).

Some of the above techniques and a number of other geophysical ones were discussed by Culshaw et al. (2004) in relation to their usefulness for the investigation of a number of geohazards. These included the use of cross-hole seismic tomography to characterize solution features, thermal imaging for identifying the location of mine entrances, high resolution airborne multisensors for the detection of contaminant plumes, and the use of LIDAR and PSInSAR for the detection of ground movement. However, it should be noted that the technologies can advance very rapidly, so it is essential to consult up-to-date experts before considering the use of these and other techniques.

These techniques do not apply in the marine environment, though a number of marine geophysical methods have been developed. Jia et al. (2016) reviewed many of the marine



Geohazards, Fig. 2 Interpolated PSInSAR data overlain on elevation map. Blue indicates areas of uplift above the older mined area and red areas of subsidence over more recently mined ground



Geohazards, Fig. 3 Contributions the engineering geologists make to the geohazard risk assessment and mitigation process

geohazards and investigation methods. The indirect investigation methods include multibeam bathymetry (to determine water depth and seabed topography), side scan sonar (for identifying seabed features and/or obstructions), high-resolution 3D seismics (for determining the seabed geology), and magnetometers to detect ferrometallic objects and debris such as pipelines, cables, wrecks, and military ordnance.

Risk Mitigation for Geohazards

Geohazard mitigation lies on the margins of what is engineering geology. However, presenting a summary of ways in which the risks resulting from geohazards can be mitigated is useful in that it puts the engineering geologist's contribution to risk mitigation in perspective. Figure 3 shows the contributions to risk assessment and mitigation from engineering geologists and a wide range of other professionals.

The following list of geohazard mitigation strategies comes from the outline of a course at the University of Oslo, now discontinued, with some added comments (http://www.uio.no/studier/emner/matnat/geofag/GEO4180/h08/undervisningsmateriale/GEO_4180_Geohazard_risk_mitigation_strategies.pdf):

- Land use plans – but in many countries, these are inadequate or do not include sufficient information on geohazards
- Enforcement of building codes and good construction practice – and the carrying out of a thorough and adequate site investigation
- Early warning systems
- Community preparedness and public awareness campaigns
- Measures to pool and transfer risks
- Construction of physical protection barriers
- Network of escape routes and "safe" places

An additional mitigation measure would be the use of insurance, though this is not necessarily available or affordable in many countries.

Conclusion

Essentially, a geohazard is a potentially dangerous event arising from one or more geological and/or environmental processes. These hazards can be subdivided into various types depending upon the nature of the geohazard – the main types are:

- Geomorphological hazards
- Geotechnical hazards
- Hydrological or hydrogeological hazards
- Geological hazards,
- Marine hazards
- Artificial hazards

However, all are investigated and studied by engineering geologists.

While "conventional" site investigation methods are relevant to the investigation and interpretation of geohazards, new or specialist techniques have developed, particularly with regard to remote sensing and geophysics.

Cross-References

▶ Acid Mine Drainage
▶ Acidity (pH)
▶ Aeolian Processes
▶ Artificial Ground
▶ Avalanche
▶ Brownfield Sites
▶ Cambering
▶ Collapsible Soils
▶ Compression
▶ Contamination
▶ Desiccation
▶ Dispersivity
▶ Dissolution
▶ Earthquake
▶ Earthquake Intensity
▶ Earthquake Magnitude
▶ Erosion
▶ Expansive Soils
▶ Faults
▶ Floods
▶ Gases
▶ Ground Motion Amplification
▶ Ground Shaking
▶ Groundwater Rebound
▶ Karst
▶ Landfill
▶ Landslide
▶ Liquefaction
▶ Loess
▶ Mass Movement
▶ Mining Hazards
▶ Permafrost
▶ Pollution
▶ Quick Clay
▶ Residual Soils
▶ Saline Soils
▶ Sinkholes
▶ Subsidence
▶ Surface Rupture
▶ Tsunamis
▶ Voids
▶ Volcanic Environments

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Geological Structures

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Synonyms

Deformation structures; Tectonic structures

Definition

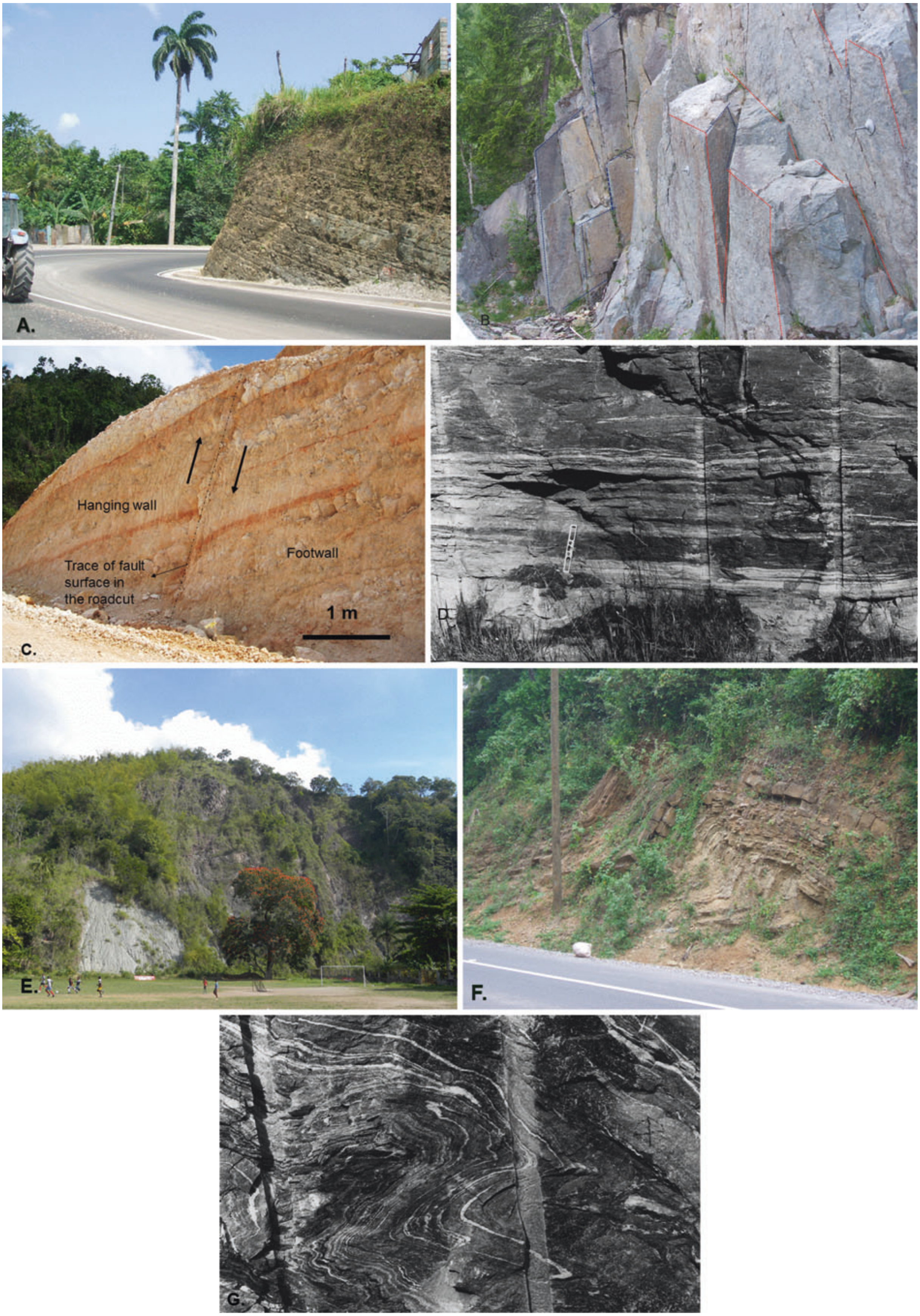
Geological structures are defined geometrically as features superimposed on rocks and landforms through the action of stresses arising from inherent Earth forces, gravity, and/or temperature, which distort rocks and change their original form and shape, as well as their location. Such changes referred to as deformation may occur from submicroscopic to regional scales.

Introduction

Structure is an inherent geometric attribute of igneous, sedimentary, and metamorphic rocks. Primary structures such as bedding (Fig. 1a), dykes, veins, and joints are characteristic of sedimentary and igneous rocks. Tectonic structures are superimposed geological structures on preexisting rocks by deformational processes related to tectonic plate movement forces. Preexisting rocks respond to deformation through the mechanisms of folding, shear rupture, extensional fracture, and changes to rock fabric (Twiss and Moores 2006). These structures are manifested as brittle planar structures which include joints (Fig. 1b), faults (Fig. 1c), foliation (Fig. 1d) are referred to as rock discontinuities, structurally controlled landforms, fault scarps (Fig. 1e), and folds, as ductile structures (Fig. 1f). Many deformed areas record successive geological structures responding to deforming forces that may have affected a terrain repeatedly over a long period of time. These are often seen in metamorphic rocks. Chronological development of geologic structures is determined by their overprinting relationships, the earlier structures are overprinted by later structures, for example, refolded folds (Fig. 1g).

Geological history of rocks and regions and their landforms are regarded incomplete without a full description and interpretation of their geological structure. Diversity of geological structures and their origin are well documented in the geological and engineering geology literature (Goodman 1993; Parks 1997; Fookes 1997; Davis 2009; Wyllie and Mah 2004; Twiss and Moores 2006; McClay 2013; Allmendinger 2015). A study of geological structures contributes to an insight of Earth processes, plate tectonics, physical properties of rocks, and their behavior under environments different from their origin.

On a global scale, all three types of tectonic plate boundary regimes: convergent (shortening), divergent (extensional), conservative (strike-slip) display characteristic assemblages of geological structures and landforms. Major geological structures in a strike-slip deformation zones are illustrated using the tectonic setting of the island of Jamaica which marks the southern boundary of the Neogene Northern Caribbean Plate Boundary Zone of left-lateral strike-slip deformation (Mann et al. Ahmad 1995). The throughgoing Enriquillo-Plantain Garden Fault



Geological Structures, Fig. 1 (continued)

Zone makes a restraining bend in Jamaica (Fig. 2) where block convergence has resulted in active faults and ubiquitous rock discontinuities and uplift of the Blue Mountains in eastern Jamaica. The neotectonic setting controls the pattern of geological structures and landforms. Rock discontinuities are a major concern to engineering geologists in Jamaica. Active faults as a geologic structure reflect areas where earthquake shaking may be more frequent and pronounced.

Primary Data on Geological Structures

As far as engineering geology practice is concerned, primary data on geological structures are collected during field investigations in an early phase of a project. These data are compiled on an engineering-geological map. Field equipment includes compass-clinometer (Brunton, USA, and Freiberger, Germany) used for orientation measurement of planar and linear structures, stereonet, hand lens, geological hammer, and camera (Barnes 1995; Coe 2010).

Structural investigations and interpretation are usually carried out at different scales: (i) global – tectonic plate boundary zones, (ii) regional – mountain belt, small island, (iii) macroscopic – involves interpretation at map scale, (iv) mesoscopic – involves identification, description, and orientation of geological structures at levels of outcrops, series of outcrops, and hand specimen, (v) microscopic and submicroscopic. In the engineering geology practice, most data are collected at mesoscopic scale and synthesized at macroscopic scale. Data on rock discontinuities are critical for rock slope stability. Global and regional level information of an area guides in project formulation and planning of field studies. Data on mesoscopic geological structures help to make ground models (Fookes 1997).

Geometrical Description and Orientation Measurement of Geological Structures

The most important and time-consuming activity of an engineering geologist in the field phase is the description of geological structures, orientation measurement, and their significance. Field techniques and procedures are available in Goodman (1993), Barnes (1995); Fookes (1997), Johnson and DeGraff (1998), Coe (2010), Wyllie and Mah (2004, a free online source), and McClay (2013).

Geological structures, three-dimensional features, can be reduced to simple geometrical features lines and planes. Linear geological structures approximate geometry of lines, for example, fold axis and elongated mineral aggregates. Planar structures, such as bedding, fault, joints, approximate the geometry of a plane. The Brunton compass is generally used to measure the orientation of planar and linear structures in the field. There are three methods to record orientation of planar structures: (i) Quadrant method; (ii) Azimuth of strike and dip amount, known as the right-hand rule; (iii) Dip azimuth and dip amount. Figure 3a and b illustrates the geometry of a plane and fault plane, and logging orientation measurement of strike, dip direction, and amount of dip. Lines are generally recorded directly as trend and plunge or orientation of line with respect to the strike of the plane that contains the line. Linear geological structures are generally recorded as (i) their trend or azimuth and plunge measured in a vertical plane, and (ii) pitch or rake of the linear structure with respect to the orientation of the plane that contains the line. These data are plotted on a geological structural map. Stereographic representation of orientation measurement of geological structures is an elegant way to highlight their angular relationships in space, relationship with other structures, and statistical significance. Wyllie and Mah (2004) have described the principles of stereographic projections, procedures to data plotting, and applications in rock stability.

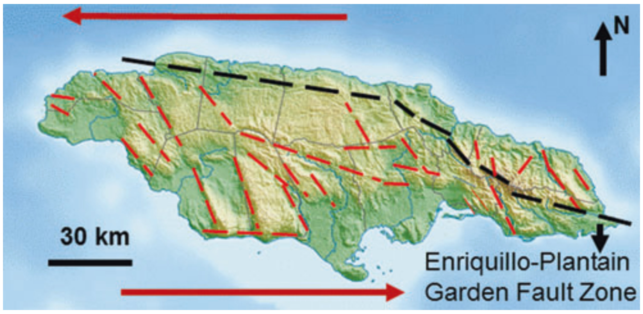
Classification of Geological Structures

Geological structures may be classified as continuous structures (e.g., fold, Fig. 1f), and discontinuous structures (e.g., joints, Fig. 1b; faults, Fig. 1c; foliation, Fig. 1d). Further, the structures may be penetrative or nonpenetrative. This attribute is governed by the scale of observation.

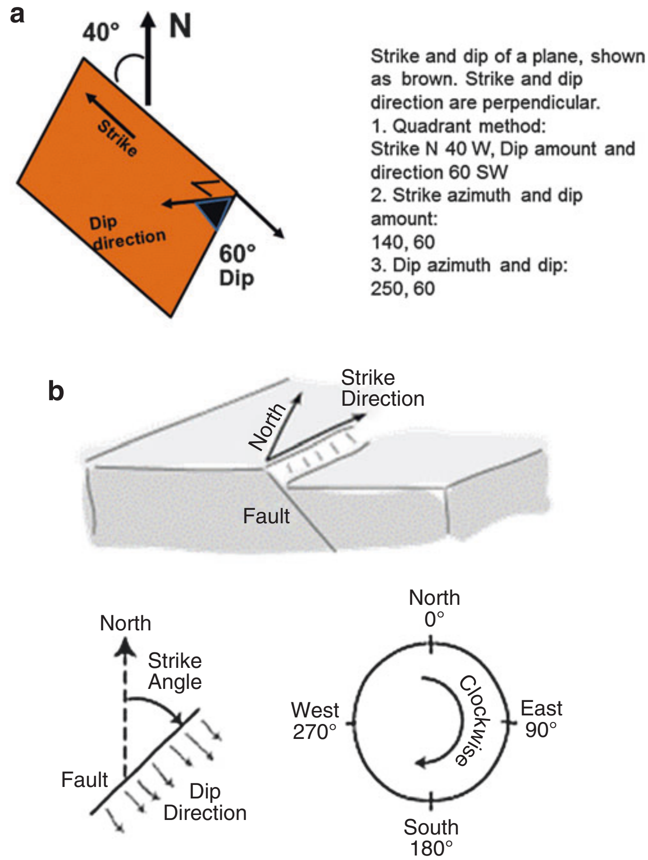
Discontinuous Structures

Rock discontinuities arise from three fracture modes: Mode I: opening, extension; Mode II: sliding; and Mode III: tearing (Allmendinger 2015). Joints represent Mode I fractures.

Geological Structures, Fig. 1 (**a**) Rhythmically interbedded sandstone and shale in a stable road cut along the North Coast Highway, St. Mary, Jamaica. Bedding planes inclined at 10° into the hillslope away from the road (Photo by Rafi Ahmad); (**b**) Orthogonal set of joints highlighted by blue and red in gneiss along a road cut on a highway in Norway. Joints are a penetrative structure of rock mass defining rock slabs. The red joint surface is bolted to avoid topple failures (Photo by Rafi Ahmad); (**c**) A thrust fault exposed in a limestone road cut near Ewarton, St. Catherine, Jamaica. Arrows indicate the relative sense of displacement of hanging wall and footwall rock blocks (Photo by Rafi Ahmad); (**d**) Horizontal gneissic foliation in amphibolite vertical roadcut along Highway E75, east of Åre, Sweden (Photo by Rafi Ahmad); (**e**) Fault scarp of the Wagwater Fault, along the Wagwater River, Golden Spring, Saint Andrew, Jamaica. The Wagwater Fault is classified as an active fault (Photo by Rafi Ahmad); (**f**) Folded sandstones and siltstones along the North Coast Highway, Saint Mary, Jamaica (Photo by Rafi Ahmad); (**g**) Refolded folds in schists in a vertical roadcut along Highway E75, east of Åre, Sweden (Photo by Rafi Ahmad)



Geological Structures, Fig. 2 Simplified fault pattern and tectonic setting on the island of Jamaica which is located in a left-lateral strike-slip deformation plate boundary zone. The throughgoing Enriquillo-Plantain Garden Fault Zone makes a restraining bend on Jamaica. Major faults define E-W and NW-SE trends. Base map is a three-dimensional model landform on the island of Jamaica; from Wikipedia https://upload.wikimedia.org/wikipedia/commons/8/83/Jamaica_relief_location_map.jpg. Accessed on 15 January 2018



Geological Structures, Fig. 3 (**a**) Methods of recording strike and dip of planar geological structures (Modified from Allmendinger (2015)). (**b**) Strike and dip of a fault plane; strike trend is measured relative to north; dip amount is inclination from a horizontal surface, from: https://earthquake.usgs.gov/learn/glossary/images/STRIKE.GIF. Accessed on 7 January 2018

Faults are fracture discontinuities in modes II and III in which one rock block has slipped past the other across the fault. Following Wyllie and Mah (2004), rock discontinues may be classified as bedding, joint, fault, and foliation (Fig. 1a–f). Characteristic properties of discontinuities include spacing, persistence, roughness, aperture, infilling/width, seepage, and number of sets (Fig. 4). Weathering of rock masses influences their shear strength. Weathering grades have been standardized (Goodman 1993; Fookes 1997). Most of these properties are usually quantitatively measured during field investigations. Spacing of fracture discontinuities in a rock mass may be narrow (<1 cm) or extremely wide (> 2 m). This property controls rock mass strength and topple failure. Persistence is defined by the length and volume of fractures in a rock mass. Surfaces of fractures may be smooth or rough. Aperture is defined as the perpendicular distance between walls of fractures. In some instances, fracture aperture is infilled by mineral matter which can alter strength relatively to being open. Open fracture aperture facilitates water seepage. Geometry, dimension, and persistence are critical attributes of rock discontinuities that control the strength of rock masses and their slope stability.

Joints

Joints, Mode I extensional fractures, are ubiquitous planar geological structure over a large area in all the rock groups. It is common to find more than one set of regularly spaced parallel joints in a rock outcrop which define systematic joints. Joint sets may have an orthogonal geometry (Fig. 1b). Non-orthogonal joint sets define a wedge which gives rise to a wedge failure. In granitic rocks, curved upward joints called sheet joints or exfoliation occur at shallow depths parallel to the surface topography as thin sheets. Terrains in areas of folded rocks may exhibit a systematic relationship of joints with fold geometry. Joints may occur parallel, perpendicular, and oblique to the fold axis. Fault zones are often characterized by closely spaced parallel joints. Vein structures are joints infilled by minerals. Open joints may be sites of ground water seepage.

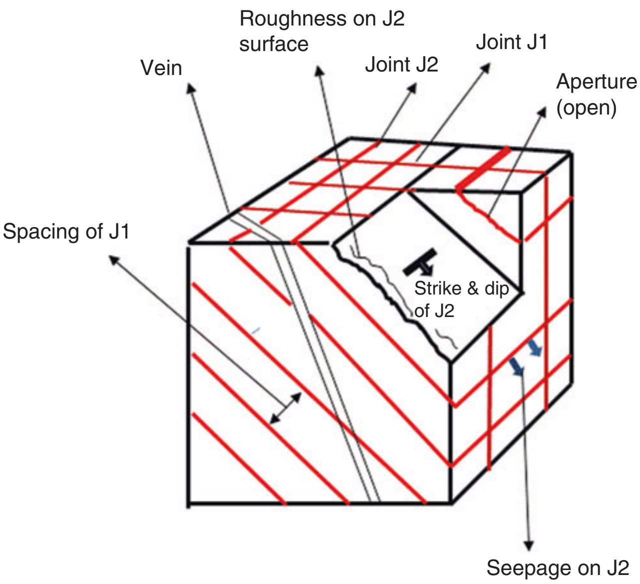
Faults

Faults are planar fractures in Modes II and III where adjacent hanging wall and footwall rock blocks have slipped past the other across the fault (Fig. 1c). Commonly fault surfaces are described as a flat planar geological structure. However, in fold mountain belts, fault surfaces may be shovel shaped; concave upward structures are termed listric faults. Fault surfaces may be inclined (Fig. 1c) or vertical.

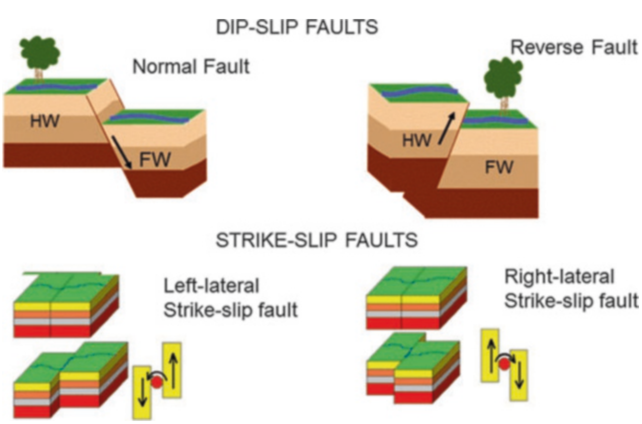
Fault Classification Fault structures may be classified as dip-slip faults and strike-slip faults based on the sense of relative displacement of rock blocks across the fault surface and the orientation of fault surface (Fig. 5). In oblique slip faults, slip offset is a combination of dip-slip and strike-slip movements.

Dip-slip faults Dip-slip faults are subdivided in two groups: normal faults and reverse faults (Fig. 5).

Normal faults In normal faulting, the hanging wall block is displaced down with respect to the footwall which leads to horizontal extension, typical of the divergent plate boundary zone. This type of movement results in juxtaposing younger strata against older. High-angle normal faults have a steep fault surface, > 45°, whereas fault surfaces <45° are low-angle normal faults.



Geological Structures, Fig. 4 Characteristic properties of rock discontinuities in a rock mass: orientation of joints, joint J2 daylights in the right-hand panel, joint sets, spacing of joints, roughness, aperture, seepage, and vein (Modified from Wyllie and Mah (2004))



Geological Structures, Fig. 5 Classification of faults based on sense of movement. From: https://earthquake.usgs.gov/learn/glossary/images/FAULT.GIF. Accessed on 2 January 2018; https://earthquake.usgs.gov/learn/glossary/images/LEFTLAT.GIF. Accessed on 2 January 2018; and https://earthquake.usgs.gov/learn/glossary/images/RIGHTLAT.GIF. Accessed on 2 January 2018

Reverse faults Reverse faults are characterized by up movement of the hanging wall with respect to the footwall manifests in juxtaposition of older strata against younger. This movement results in horizontal shortening which is typical of folded mountain belts in the convergent plate boundary zones. Thrust faults are low-angle, dip <45°, reverse faults. High-angle reverse faults have fault surface dip > 45°.

Strike-slip faults Strike-slip faults are vertical shear fractures typical of the conservative plate boundaries. These faults are further subdivided as right-lateral (dextral) and left-lateral (sinistral) faults (Fig. 5). Right-lateral strike slip faults identified as an observer looking across the fault trace, the marker features (streams, ridges, linear structures) are offset to their right-hand side. In left-lateral strike-slip faults, marker features are offset to the left across the fault.

G

Simple Fault Kinematic Indicators Kinematic indicators are features which help to determine the sense of slip along faults. Fault surfaces may be decorated with a variety of morphologies of minor structures and mineral precipitate. Slickenside striations are elongated mineral matter on fault surfaces (Fig. 6). Slickenside striations may be used to determine sense of relative slip on rock surfaces. The surface down slip direction feels smooth. In fault zones, termination of rock layers against faults surface may exhibit small-scale and localized folds. These structures are referred to as drag folds which indicate the sense of movement across the fault surface.

Geological Structures, Fig. 6 Slickenside striations on a fault plane near Robin’s Bay, Saint Mary, Jamaica (Photo by Rafi Ahmad)

Faulted Rocks The process of faulting results in breaking and bending of rocks in a fault zone. Fault rocks are further divided based on their strength (cohesive/non-cohesive) and volume percent of rock fragments. Noncohesive fault rocks are fault breccia and cataclasites with > 30% of rocks fragments and fault gouge fault rocks with <30% fragments. These types of rocks are associated with shallow faults occurring in upper levels of the crust (about 1–4 km depth) and typically low-temperature environment. Cohesive fault rocks, cataclasites and layered mylonites, on the other hand, occur in relatively high temperatures (250–350 °C) and deeper depths (4–15 km depth) typical of shear zones.

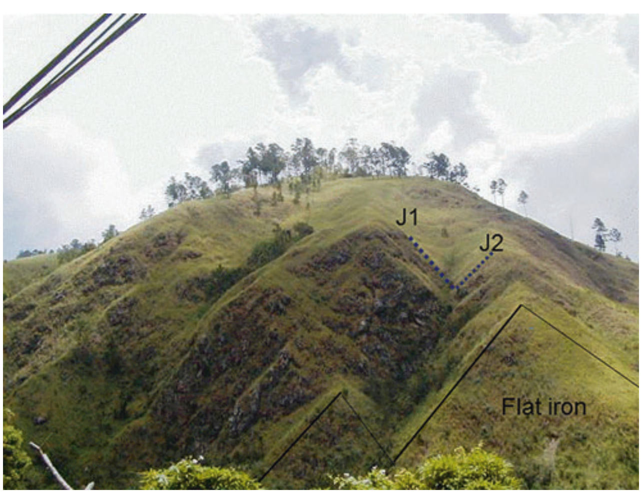
Fault scarps are surface topographic expression of effects of faulting (Figs. 1a and 5). Fault scraps associated with normal faults often show flat irons structure (Fig. 7). Strike-slip fault zones may be identified with off-set streams (Fig. 5).

Continuous Structures

Folds

Bending and flexing of layered rocks produce fold structures (Fig. 1 f and g). Folds may occur at microscopic to macroscopic scales. Anticline is a convex upward fold with stratigraphic older rocks in the center of the structure, whereas, syncline is concave upward fold structure with younger rocks in the center. Fold shapes may be symmetric as a structure with equal fold limbs length and dip. Asymmetric folds are characterized by unequal lengths limbs and dip.

Geometric description of folds: In a two-dimensional view of a fold structure, a zone of maximum curvature of folded layer is known as the hinge point of the fold (Fig. 1f). In a three-dimensional view, a line connecting hinge points of a folded layer is referred to as a hinge line or fold axis (Fig. 8a).



Geological Structures, Fig. 7 Flat irons structure in a fault scarp in andesitic volcanic with a set of penetrative non-orthogonal joints, J1 and J2, near Mavis Bank, Saint Andrew, Jamaica (Photo by Rafi Ahmad)

A plane containing successive hinge lines of folded layers is known as the axial plane of the fold (Fig. 8a). Fold axis or hinge line is a linear structure specified by trend and plunge. Axial plane is a planar structure measured as strike, dip amount, and dip direction.

Fold structures are classified based on the following criterion: (i) Orientation of fold axis and axial plane: Folds with a horizontal fold axis and vertical axial plane, and a horizontal fold axis and axial plane are termed vertical fold and recumbent fold, respectively. Folds with a plunging fold axis and variously inclined axial planes are known as plunging folds (Fig. 8a). (ii) Fold tightness as the interlimb angle viewed in a plane normal to fold axis. Isoclinal folds have parallel limbs. Tight folds have 10–90° interlimb angles and open folds have 90–170° interlimb angles. (iii) Fold shapes, as chevron folds (Fig. 8a) have sharp angular hinges and planar limbs.

Description of folds

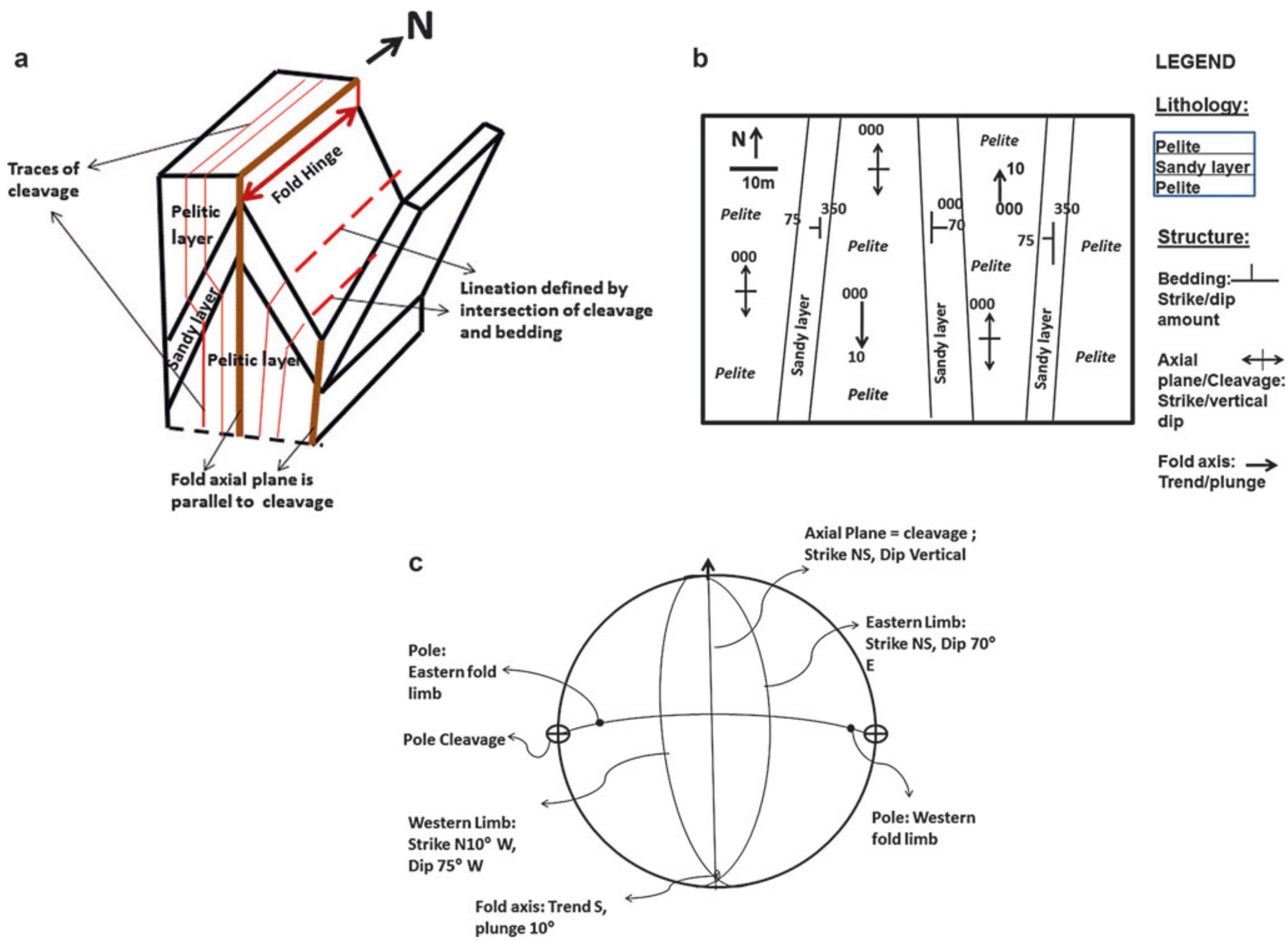
A complete description of folded rocks involve field mapping of the structure, stereographic representation of fold geometrical attributes and related structures, and a three-dimensional model of the fold. Figure 8 represents a description of a hypothetical fold structure based on an actual field example.

Minor Geological Structures in Metamorphic Rocks

Minor geological structures, foliation and lineation, are a common feature of metamorphic rock fabric which are observed at a mesoscopic scale and may be related to fold geometry (Fig. 8). Foliations are planar structures characteristic of slate (cleavage, crenulation cleavage), schist (schistosity), and gneiss (gneissosity in Fig. 1d) defined by preferred orientation of minerals. Bedding in layered sedimentary rocks is also regarded as a foliation. Lineations include minor fold axes, mineral lineations (commonly elongation of aggregate of minerals), and intersection of planar structures in rock masses.

Summary

Geological investigations, particularly bedrock structure, are an important component in the siting and designing of civil engineering structures (Jaegar 1980; Fookes 1997). Without knowing the bedrock structure with a rock mass, its actual strength and ability to resist strain is not fully known. Rock discontinuities are common place geological structures controlling foundation suitability and the stability of natural and engineered slopes. Although built environment practitioners and engineering codes recognize the critical role of geological structures in civil engineering structures, it is a challenge to



Geological Structures, Fig. 8 (**a**) Three-dimensional sketch of a gently plunging, asymmetric chevron folds in a slate. Sedimentary layering, pelitic and sandy layers, are overprinted by a slaty cleavage which parallels the fold axial plane. Intersection of cleavage and bedding defines an intersection lineation which parallels fold axis. (**b**) Sketch map representation of Fig. 8a. (**c**) Stereographic representation of Fig. 8a (Figure by Rafi Ahmad)

integrate the two disciplines. Rock slope engineering, popularized by Evert Hoek and J. Bray, are widely available, also a free online publication (Wyllie and Mah 2004). There are many instances where failure of engineering structures is ascribed to deficiency of attention of geological structures (Goodman 1993; Bosela et al. 2012). The 1963 Vaiont Reservoir Disaster underscores the importance of geology and rock discontinues in stability of reservoirs (Semenza and Ghirotti 2000).

Cross-References

- ▶ Bedrock
- ▶ Cross Sections
- ▶ Deformation
- ▶ Engineering Geology
- ▶ Engineering Properties
- ▶ Faults
- ▶ Geology
- ▶ Geotechnical Engineering
- ▶ Igneous Rocks
- ▶ Landforms
- ▶ Metamorphic Rocks
- ▶ Normal Stress
- ▶ Physical Weathering
- ▶ Rock Mass Classification
- ▶ Rock Properties
- ▶ Sedimentary Rocks
- ▶ Shear Strength
- ▶ Shear Zone
- ▶ Strain
- ▶ Stress
- ▶ Tension Cracks

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Geology

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Synonyms

Earth science

Definition

Study of the Earth's structure, constituent materials, Earth processes, and the development of life, from the Earth's formation to the present day.

The science of geology considers:

- Earth materials (minerals, rocks, soils, and contained water and gases)
- Surface and subsurface Earth structures and the ways that they form
- Earth processes within, and at, the surface of, the planet
- Earth history from the formation of Earth to recent times

It draws on the fundamental disciplines of physics, chemistry, mathematics, and biology. Geoscientists require a working appreciation of all of these but, depending on their specializations, more detailed appreciation of some. Specializations can be academic or applied (Table 1). Introductory texts on some aspects include Parriaux (2009), Plummer et al. (2016), and Pohl (2011).

This Encyclopedia is devoted to engineering geology but, more generally, applied activities of geoscientists include:

- Exploration for, and evaluation and economic exploitation of natural resources (mining and quarrying of minerals and rocks, and extraction of natural gas, oil, and water through boreholes) and evaluation of their properties and suitability for use
- Identification and development of suitable surface and underground facilities for storing resources (such as underground gas storage and surface water reservoirs) and associated infrastructure
- Investigation, analysis, and remediation of physically and chemically damaged land including problems of contamination, pollution, and rehabilitation of landscapes after, for example, mining or quarrying
- Assessment and monitoring of ground conditions, including properties and behavior of soils, rocks, and groundwater, and design of precautionary and remedial structures and foundations for surface and underground construction
- Management of wastes including landfill, evaluation of the potential for recycling of mineral components and underground storage of radioactive wastes
- Establishing the nature and extent of geological hazards (earthquakes, volcanoes, landslides, subsidence, tsunami, and floods), undertaking risk assessment and contributing to risk reduction through, as appropriate, land use planning, design of emergency responses, and, if feasible, designing and delivering precautionary and remedial works

Engineering geologists contribute to all of these activities although often in collaboration with experts from allied disciplines (e.g., during environmental impact assessments). Applied activities must be founded on high-quality basic geoscience. Also, engineering geologists have to interact with administrators, officials, and the general public. Therefore, an ability for scientists to communicate in nontechnical terms is important.

Applied geological techniques are varied including: direct observation, remote sensing, mapping, modeling, sampling, testing, and monitoring at scales ranging from broad-scale

Geology, Table 1 Current subdisciplines of geology

Crystallography	Study of crystalline substances for identification and determining of physical and chemical properties and behaviors
Engineering geology	Geology applied to ground surface and underground development
Environmental geology	Geology applied to environmental problems such as contamination and pollution and health impacts
Forensic geology	Geological techniques applied to searches and investigations of matters raised by legal systems (e.g., in crime scene investigations, suspected fraudulent mining schemes)
Geochemistry	The chemical nature and behavior of rocks, soils and groundwater including contamination, pollution, impacts on human health, crops, livestock and the environment but also the significance of elements and compounds as nutrients, and the implications of reactivity for construction materials
Geochronology	Dating of geological materials either relatively (e.g., through structural relationships of strata and contained fossils) or more absolutely (through, for instance, radiometric, dendrochronological and thermoluminescence techniques)
Geomorphology	Study of ground surface morphology and processes developing and changing landforms (shared with physical geography)
Geophysics	Detection and utilization of physical phenomena to understand Earth structure, materials, and hazards including seismological, electromagnetic, and radioactive phenomena, as appropriate, in remote sensing and ground investigation and monitoring
Geotechnics	An essentially mathematical approach to applied geosciences issues such as the interface between soil and rock mechanics, natural hazards and construction. That is important. But it is wise for geotechnical engineers and engineering geologists to discuss matters of mutual interest at an early stage in any project
Hydrogeology	Study of the nature, composition, and behavior of groundwater and interactions with surface water which are the subject of study in hydrology
Medical geology	Essentially geochemical work to detect, evaluate, and deal with harmful elements and compounds in soils, waters, and the air (mineral dust) and also to establish the availability of nutrients for people, livestock, and crops
Mineralogy	Study of properties and behaviors of minerals
Paleoclimatology	Deduction of past climatic variations from sedimentological, paleontological, and geomorphological evidence and geochemical properties of rocks and groundwater, linked to study of present-day climatology
Paleontology and paleoecology	Study of previously living organisms, their morphologies and evidence of behavior which is informed strongly by understanding of the classifications, genetics, and adaptive morphologies of present living organisms (biology and ecology)
Petrology	Study of the mineralogical compositions of rocks to determine the mineralogical constituents and processes of formation
Sedimentology	Study of sediments, depositional processes and lithification and environments of deposition
Seismology	Study of vibrations and impacts caused by natural earthquakes and anthropogenic ground tremors and utilization of vibrations in investigating subsurface structures
Stratigraphy	Study of sequences and changes in stratified deposits and their relationships with other structures and rocks as a basis for establishing earth history
Structural geology	Study of geological structures ranging from large-scale folds and faults to smaller scale phenomena such as joints in order to reconstruct the processes that have formed the Earth's crust
Volcanology	Study of the nature and behaviors of volcanoes

satellite observation to electron microscopy and spectrographic studies. Although laboratory work is essential, high-quality fieldwork and site investigations are also essential to ensure that sampling, testing, interpretation, and modeling are properly based on the actual circumstances at the sites. William Whewell in 1832 propounded the view that the present is the key to the past (Anon 2016), but it is now clear that present and past are, together, also a basis for assessing the future particularly in terms of environmental, climatic, and biodiversity changes.

Cross-References

▶ Engineering Geology
▶ Engineering Geomorphology
▶ Geochemistry
▶ Hydrogeology

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Geophysical Methods

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Definition

A suite of techniques which seek to determine the location and arrangement of materials and features in the subsurface by remotely detecting their physical properties and/or contrasts in their physical properties with the surrounding ground.

Introduction

Geophysical surveying is just one set of tools available to investigate the subsurface. Normally, the acquisition and interpretation of geophysical data are planned as part of an integrated ground investigation to obtain representative information across large continuous areas of the site and/or to provide specific targeted information on areas of concern highlighted by desk study information or previous phases of ground investigation.

When considering the use of geophysics as part of the investigation, the engineer should consult a qualified professional geophysicist to advise on the likelihood or otherwise of the survey returning the information required by the client and also establish the accuracy and resolution that is expected to be achieved. Badly specified surveys or those undertaken by inexperienced or unqualified personnel risk returning no useful information. Such cases have, in the past, contributed to the number of engineers practicing today who consider that geophysics "doesn't work." Fortunately there are excellent examples of geophysics done well which demonstrate the value to clients of well-specified investigations. This is reflected in the increasing number of tenders, particularly for high-profile high-value projects, where geophysical techniques are being specified.

Boreholes and trial pits provide direct information on a tiny fraction of the volume of ground (Fig. 1). The balance of risk to the client therefore hangs on the likelihood that some variation, problem, or obstruction of engineering significance exists in the large spaces remaining, whose discovery later in the project would cause expensive investigation, redesign, and delay. This risk depends on the site history, on its geology, as well as on the scale and complexity of the project. Assessing risk relies on the expertise of the ground engineering specialist. What ground investigation is commissioned depends on the informed risk decisions made by the client supported by the consulting engineer and geophysicist.

To reduce the risk of unforeseen ground conditions, the choice of the correct geophysical method, or combination of methods, is essential, both from the point of view of maximizing the success of the survey in reducing the risk of unforeseen ground conditions and obtaining the desired geological information on a cost-effective basis.

Geophysical techniques can be used on land, at sea, and from the air. In each case, the basic techniques are modified, but the same physical properties are involved irrespective of the environment. This entry provides a brief overview of commonly used land-based methods. For detailed technical background and case studies for each technique, more complete texts are available, e.g., Styles (2012) and Tuckwell (2015).

Of importance, and not described in detail here but which are covered in other entries of this Encyclopedia, are borehole methods which can be considered in two categories: those deployed by "logging" tools which measure the physical properties of the rock in the borehole wall or in the immediate vicinity of the borehole and "downhole" tools that measure the physical properties of the ground in a larger area around the borehole, between the surface and the borehole, or between boreholes. For further information, reference should be made to other sections of the Encyclopedia and specialist textbooks, such as Keys (1996).

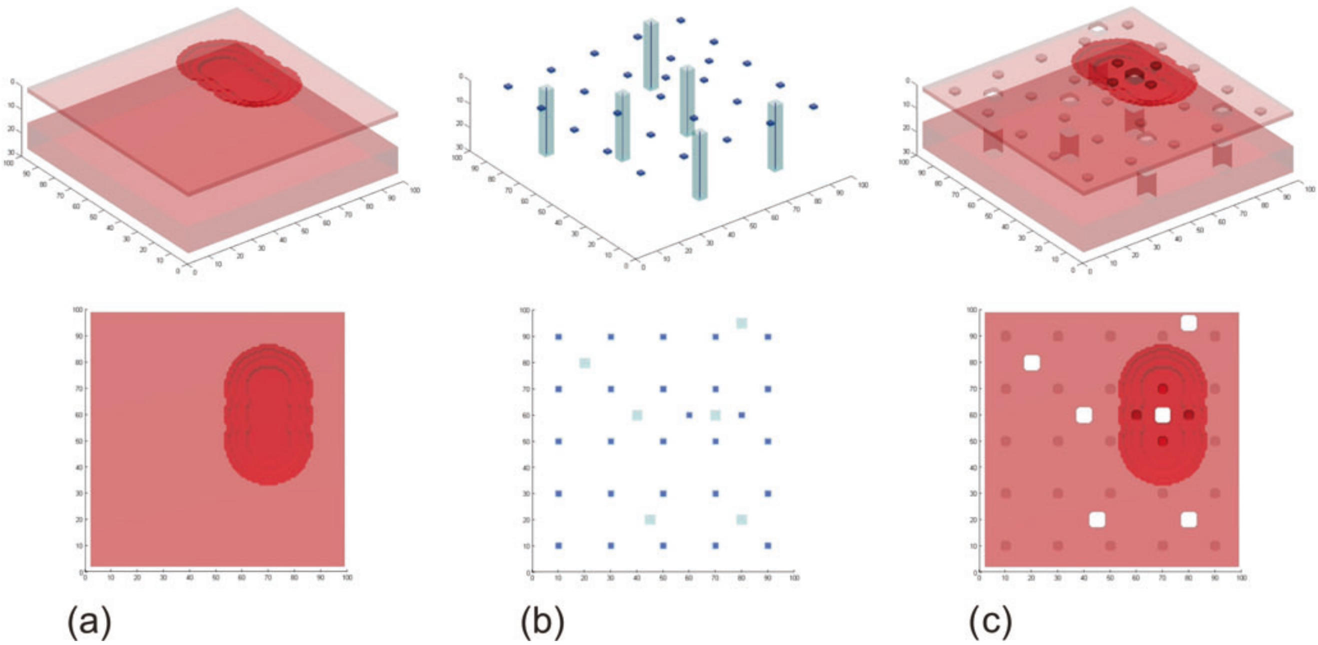
Electromagnetic (EM) Methods

EM methods generally consider the propagation of an alternating electromagnetic field into the ground to detect subsurface variations. These techniques offer options for the rapid characterization of soil, detection of buried objects, and identification of areas for further investigation. A huge logistic advantage of the EM methods is that there is no physical ground contact so they are much quicker to cover survey areas than some other techniques.

EM Frequency Domain Ground Conductivity Surveys

The EM frequency domain method uses two electrical coils to send and detect an electrical signal that is modified according to the electrical conductivity of the subsurface. The amplitude of the signal in the secondary coil is directly related to the bulk electrical conductivity of the ground, and the phase of the signal indicates the presence or otherwise of metals. The depth penetration is determined by the electrical properties of the ground and the separation of the coils and by the ability to generate a sufficiently powerful secondary field at depth to be detectable at the surface.

An EM survey can provide rapid reconnaissance information regarding the presence and location of UXO, buried



Geophysical Methods, Fig. 1 Conceptual visualization of information and risk. (**a**) *Red* colors represent the absence of information, which equates to a risk of unforeseen ground conditions. Deeper reds indicate higher risk based on information available, e.g., fill material or possible contamination. (**b**) *Blue* colors indicate direct information revealed by boreholes or trial pits. (**c**) *Red* colors indicate the volume of ground following the intrusive ground investigation for which no direct information is available and therefore where risk of unforeseen ground conditions may remain

services, foundations, storage tanks, obstructions, waste, groundwater, and contamination (Fig. 2).

Transient Electromagnetic (TEM)

Electromagnetic energy can also be applied to the ground using transient current pulses instead of the continuous waves described above. The transient secondary magnetic field is related to the conductivity and geometry of features in the subsurface. Typical TEM systems can provide rapid measurements from a few meters down to several hundred meters. TEM is a well-established technique for mineral exploration and is increasingly being applied to hydrogeological mapping (especially saline intrusion) and to engineering site investigation studies, e.g., Kirsch (2009).

The inversion of TEM data determines a profile of conductivity with depth which is then interpreted in terms of a layered Earth model with the principal parameters being layer thickness and layer conductivity. As with other profiling techniques, it suffers from the problem of nonuniqueness of solutions and, so, benefits considerably from constraining information from complimentary ground investigation methods.

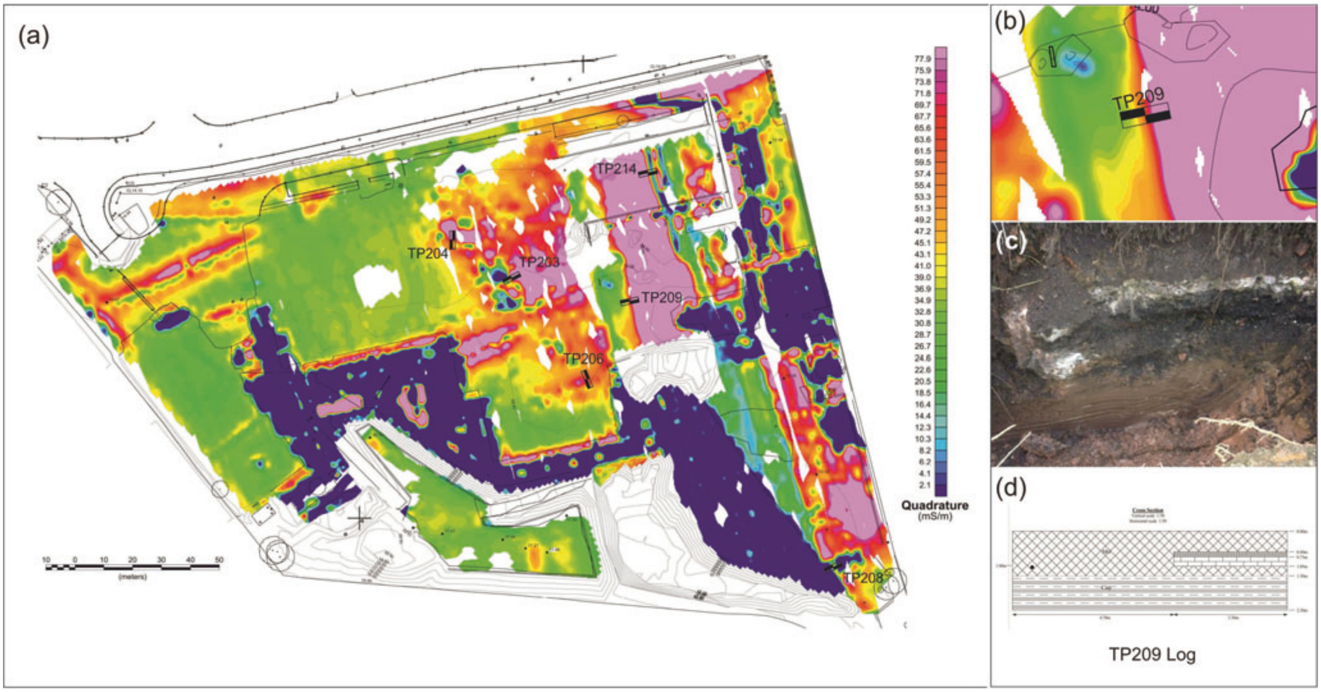
Ground-Probing Radar (GPR)

The GPR technique operates by transmitting a pulse of high-frequency (radar band) electromagnetic radiation into the ground which is reflected back to the instrument at boundaries between materials with contrasting electrical properties. These reflections are recorded and displayed in cross sections or 3D volumes for processing and interpretation (Fig. 3).

The technique is commonly used in environmental and shallow engineering surveys where information is required from the upper 5 m. The penetration depth of GPR increases with decreasing frequency, with the lowest frequencies of 25 MHz or 50 MHz typically deployed for geological applications where depth penetration in favorable conditions can be 40–80 m.

The most common application is in utility detection, where GPR is used in conjunction with accurate surveying, and other utility tracing techniques including electromagnetic location. However, the GPR technique also images any other feature in the shallow subsurface which constitutes a contrast in electrical properties and can therefore be used to locate and map underground storage tanks (USTs), buried obstructions, foundations, basements, and natural and mining voids. High-frequency, high-resolution surveys provide a nondestructive testing tool for concrete and other construction materials, allowing their internal structure and flaws to be imaged *in situ* without coring or breaking out.

The main physical limitation of the technique is that the signal is rapidly attenuated in conductive ground conditions (e.g., waterlogged clay). As with all geophysical techniques, what is achieved can also be limited by the skill of the operator and by the effort put into processing and interpreting



Geophysical Methods, Fig. 2 (**a**) Frequency domain EM ground conductivity map of a brownfield site. (**b**) Anomalies and features indicated in the geophysical data are targeted for investigation by trial pits. (**c**) Photo of trial pit TP209 showing the presence of a buried concrete structure within the made ground, with clay beneath (**d**) trial pit log for TP209

the data once acquired. It should be remembered that the results of a GPR survey, as with other geophysical surveys, are interpretative and rely on the competence and experience of the geophysicist.

Potential Field Methods

Potential field methods passively measure the potential field at a particular point, typically near to the ground surface. They have the advantage that they are not depth limited in the sense that an active signal is sent into the ground and needs to return to the surface with sufficient strength to be measured. The only restriction on depth is that the potential field from the causative body is sufficiently large to be measured at the surface and distinguished from any other fields being generated by other features, some of which may be nearer to the measurement point.

Magnetic Mapping

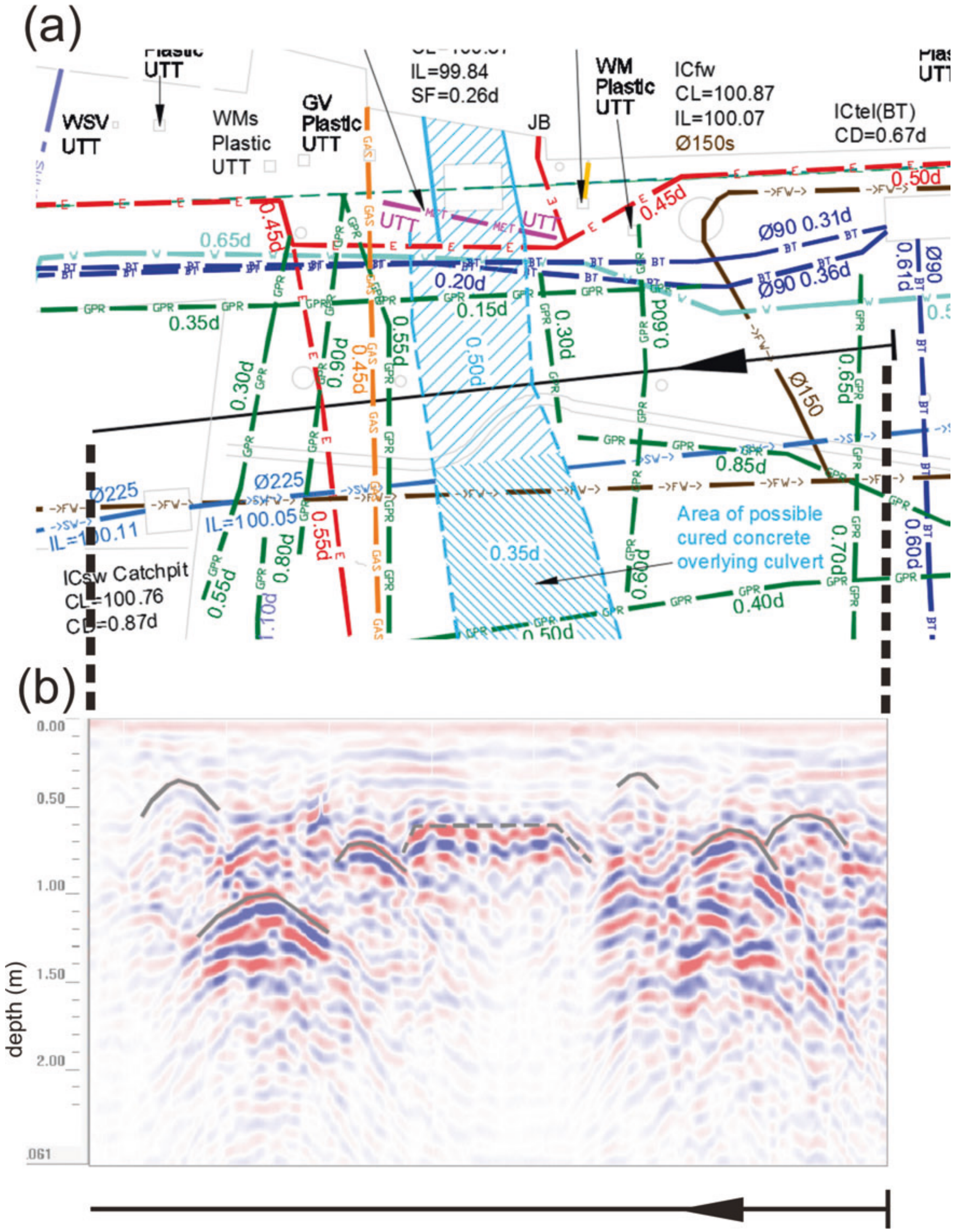
Local variations in the electromagnetic properties of the subsurface act to cause small perturbations in the magnetic field. The technique is used extensively in archaeological investigations to identify historic structures and associated variations in the shallow subsurface but is equally applicable to detection and mapping of buried ferrous objects; pipes, drums, cables, and military ordnance (UXO); buried structures such as air raid shelters; historical mine workings; and other buried foundations from recent activity on a site.

The technique can be heavily influenced by "cultural effects" where aboveground surface features mask the response from the targets belowground. Care should be given to application of the technique in urban or developed areas to ensure that the signal of interest will be detectable.

The processing and interpretation of magnetic data have benefited from the development of more computationally intensive algorithms to process, filter, and model the data (Stavrey et al. 2009).

Gravity Mapping

The density contrast between different Earth materials provides a useful means for mapping subsurface structures through measurement of associated localized variations in the Earth's gravitational field. Measurements at or near to the Earth's surface vary with location due to a number of factors including tides, latitude position, and height. Within each observation is the relatively small contribution made by the density, volume, and distribution of materials in the subsurface. In order to isolate these, all other contributions must be calculated and removed. What remains is a gravitational anomaly map in which variations relate directly to the density distribution of the subsurface and can be interpreted in terms of an engineering ground model (e.g., Tuckwell et al. 2008).



Geophysical Methods, Fig. 3 (**a**) Section of a typical drawing delivered following a detailed GPR survey showing the location and depth of buried utilities and obstructions identified. (**b**) Example radargram showing the reflections generated by buried utilities and obstructions

The gravity technique can be used to detect subsurface voids (caves, mine workings, basements, etc.), buried structures (i.e., foundations, storage tanks), regional mapping of bedrock, and mapping fault zones. The principal limitation on the use of gravity is the time required for careful acquisition and data processing. Unlike many geophysical techniques, gravity surveys can be carried out inside buildings.

Once an anomaly map is obtained, a number of approaches can be used to obtain a robust interpretative ground model. An experienced geophysicist will be able to provide a qualitative interpretation of the geology based on the anomaly map alone. Euler decomposition and wavelet analysis, for example, have been shown to have equal application to both magnetic and gravitational data (Cooper 2006). The most robust interpretations are obtained by iterative forward modeling constrained by information from complimentary ground investigation methods in which a ground model is refined until the predicted gravity anomalies provide a close match to the recorded data.

Electrical Techniques

Electrical methods have seen major improvements in instrumentation, data processing, visualization, and analysis techniques over the past 25 years; electrical resistivity data are now commonly utilized in site investigation (Binley 2015). Notable developments have also been made for the methods of induced polarization (Aristobemu and Thomas-Betts 2000) and self-potential (Wilkinson et al. 2005), both of which show promise in practical engineering applications.

Electrical Resistivity Tomography

The electrical resistivity method employs a number of electrodes that are deployed along a survey line (to produce a cross section) or in a grid (to obtain a 3D image) and between which electrical current is driven and ground resistivity measurements are taken. By making measurements between different combinations of electrodes, the resistivity at different locations and depths is recorded to build up the data set. By varying the electrode spacings and positions, the spatial variation of electrical resistivity (laterally and with depth) can be determined and related to changes in subsurface properties (Fig. 4) (Wilkinson et al. 2005).

Different geological strata have different electrical properties, and as such variations in the subsurface, resistivity can be correlated with geological boundaries. In addition, the presence of pore fluids and their electrical properties can significantly modify the measured electrical properties of the ground. Discrete features can also be imaged in certain circumstances, for instance, mine workings, tunnels, natural voids, and contamination plumes.

The depth penetration is limited by the current that is possible to drive through the subsurface at depth. Typically a practical limit is within the upper 100 m below surface. Resolution decreases with depth so that deeper features have to be increasingly large in volume to be visible to the technique.

The inversion process is more robust, and the practical noise levels are minimized, where the maximum density of

Geophysical Methods, Fig. 4 Electrical resistivity tomography data across a former landfill indicating the extents of the waste material, the boundary with the natural clay beneath, and the position of leachate fluids within the waste volume data are acquired for the particular array deployed. An amount of "oversampling" can allow the noise and repeatability in the data to be estimated, which in turn informs the interpreted accuracy of the resultant ground model.

Seismic Techniques

Seismic Refraction

Seismic energy is injected into the ground, and the resulting elastic waves are refracted along the boundary between units with contrasting seismic velocities. Detection and analysis of the arrival of these waves at the surface can provide information on the distribution of seismic velocity properties within the subsurface, which in turn can be interpreted in terms of a geological model.

The technique is typically deployed to identify major subhorizontal contrasts such as the top of competent bedrock. The velocity information obtained can be used to evaluate engineering rock properties (Caterpillar Inc. 2000).

Acquisition is typically undertaken as a series of 2D sections. Survey design should be tailored for the expected geological features and the depth from which information is required. The resolution of the technique varies both with the deployment geometry and the geological conditions and should be considered carefully for each case. Interpretation of the data provides layer thicknesses and seismic velocities which can be used as the basis for a detailed interpretative ground model.

Multichannel Analysis of Surface Waves (MASW)

Surface-wave measurements provide an *in situ* and noninvasive method of determining shear wave velocities with depth at a single point or along a line. The velocities of the waves are determined by the elastic shear modulus of the ground and its density and can be used to calculate the dynamic shear stiffness of the ground (G_{max}) and to calculate Vs_{30} values for input into seismic hazard risk assessment studies (Foti et al. 2014).

Longer-wavelength, lower-frequency, signals sample the ground to greater depths, and therefore by measuring at a range of frequencies, a profile of velocity with depth can be recorded.

Ambient noise typically contains a broad range of frequencies that can be used as the input source for this technique. Not all frequencies across the useful spectrum are necessarily present within the natural ground noise, and as such an active source (vibration or impulse/explosion) can also be deployed.

Ground models are typically built as a one-dimensional profile of velocity or elastic stiffness. Adjacent profiles can be merged to give continuous cross section images of the shear velocity of the subsurface and will identify lateral as well as vertical changes. Discrete layers at depth can be difficult to resolve, and the technique is most useful for the identification of major changes in the geology with depth such as the upper interface of bedrock or basement rocks, the provision of engineering properties, and the evaluation of ground improvement schemes (e.g., dynamic compaction).

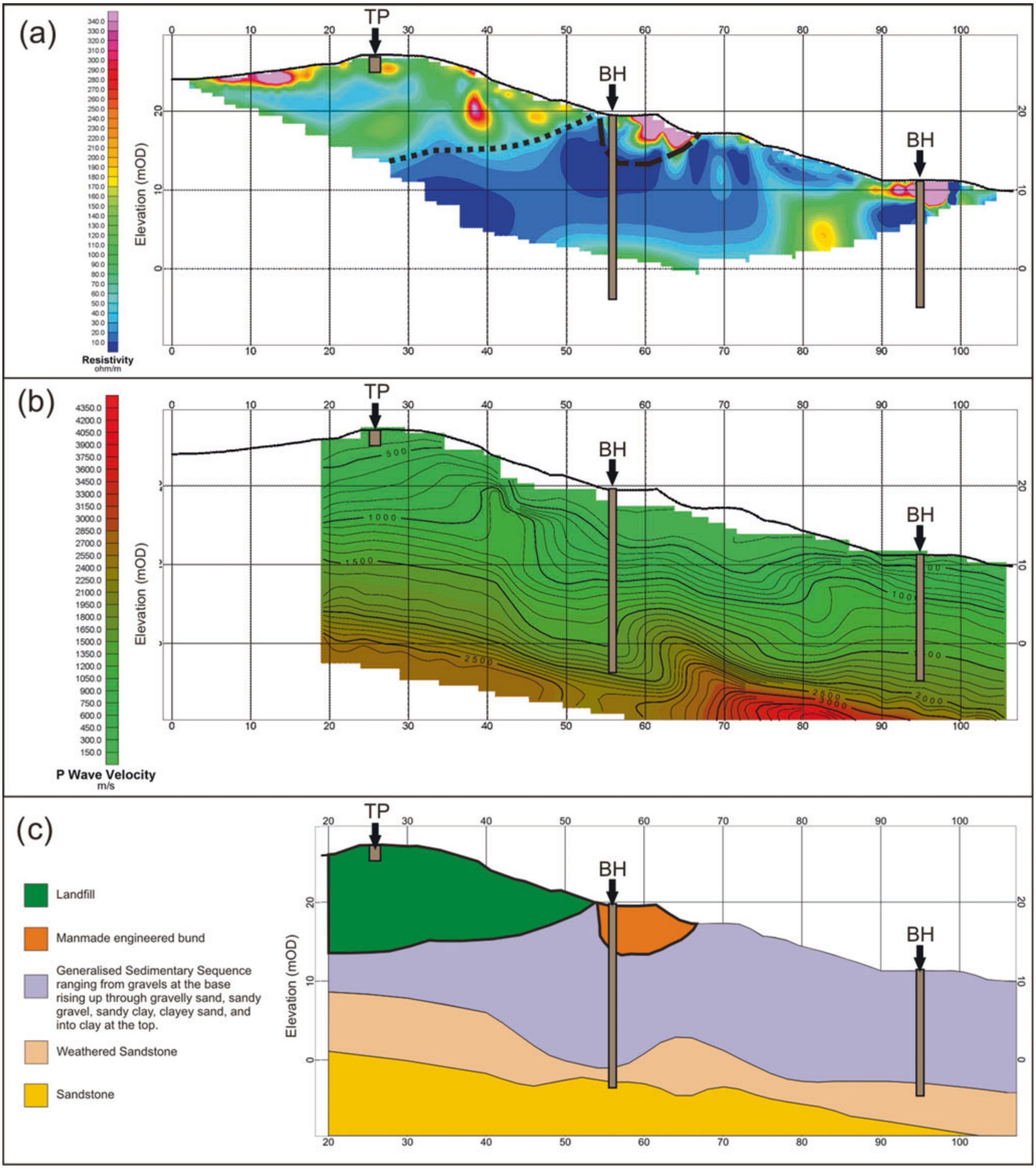
Seismic Reflection

The seismic reflection method concentrates on recording and isolating seismic energy reflected from layer boundaries in the subsurface where there is a contrast in elastic properties. Any geological boundary can represent a contrast that would generate a reflection. As such the technique can provide detailed information on the geometry of sedimentary sequences, structural faults, igneous intrusions, and evaporite deposits. The reflection method has the advantage over the refraction method of being able to image more steeply dipping and more rapidly varying lateral structure. In can also detect both increases and decreases in seismic velocity with depth; however it can be difficult to obtain good information from the very shallow (<50 m) depth range in the absence of strong sharp contrasts in the geological layers, which has to date restricted the use of the method almost exclusively to those projects where deeper geological information is required.

Where appropriately specified data have been acquired, advanced processing can be undertaken to obtain other characteristics of the seismic waves that can be linked to specific physical properties of the rock mass such as fracture density and orientation, porosity, and rock mass strength.

Concluding Remarks

The geophysical methods described here each have their place in the ground investigation toolbox available to the engineer. Their greatest value to clients is realized as part of an integrated investigation of a site. Complementary data sets can deliver much more value than the sum of their parts (Fig. 5). For example, a gravity survey constrained by dynamic probe data can give a detailed ground model which includes the depth and volume of voids and poorly compacted ground; a resistivity survey constrained by borehole data can provide the material waste distribution and accurate volume calculations for a landfill; a combined refraction and surface wave survey can discriminate between clay rich and granular materials; and an EM conductivity survey followed up with targeted trial pits can map and characterize all of the shallow obstructions, including testing representative samples for contamination. A summary of targets for each geophysical technique described in this chapter and typical data combinations that provide excellent enhanced value to the client are provided in Table 1.



Geophysical Methods, Fig. 5 Complimentary (**a**) electrical resistivity tomography, (**b**) seismic refraction, and intrusive data combined to generate a detailed ground model (**c**) in which the location, lateral variation, extent and nature of landfill waste material, engineered ground, drift sedimentary sequence, weathered sandstone bedrock, and competent sandstone bedrock are delineated with confidence

Each geophysical method, in isolation or in combination, delivers a particular type of information which in many cases is not available through other means or which would otherwise entail extensive and expensive intrusive investigations. Geophysical methods are not appropriate to every site however when used intelligently and in the right circumstances are able to reduce the risk of unforeseen ground conditions.

Geophysical Methods, Table 1 Common targets for geophysical techniques and synergies between ground investigation methods

Technique	Common targets	Complimentary ground investigation data
Electromagnetic (EM) frequency domain conductivity	Buried obstructions, voids, groundwater, contamination, buried waste, storage tanks, foundations, landfill, leachate, buried utilities	Can be used as a quick reconnaissance tool for a number of targets, with subsequent investigations targeted on the conductivity anomalies located: GPR can provide depth control on buried features; trial pits will prove the ground conditions for each anomaly and identify specific obstructions; gravity will confirm voids and mine workings
Transient EM (TEM) conductivity	Geology, groundwater, saline intrusions, waste, landfill, voids, mineralization, foundations, buried obstructions, storage tanks	Reconnaissance shallow surveys can be combined in the same way as frequency domain measurements above. Deeper investigations for geological or hydrogeological targets will benefit from constraints on geological layering from borehole or other geophysical techniques either undertaken previously or targeted on TEM anomalies and by confirmation of groundwater anomalies from targeted boreholes
Ground-penetrating radar (GPR)	Buried obstructions, foundations, basements, concrete structure and condition, voids, bedrock, water table, storage tanks, archaeology, buried utilities	Most commonly followed up by trial pits or other shallow excavations (using safe digging techniques when near buried hazards such as storage tanks and live utilities) to confirm the nature of features imaged by the GPR data. Deeper GPR surveys may be combined with electrical resistivity or seismic surveys to provide a more complete ground model or by probing or boreholes to prove voids or geological variations
Magnetic mapping	Archaeology, buried obstructions, unexploded ordnance (UXO), storage tanks, foundations, geological features (esp. igneous intrusions) mineralization, buried utilities and pipelines	Commonly combined with EM or GPR techniques to provide additional information on the shallow subsurface over a large survey area for common archaeological and brownfield targets; combined with electrical resistivity for deeper geological variations; combined with EM and gravity for detection of mine workings. Shallow anomalies commonly targeted by trial pits or observation trenches
Microgravity mapping	Cavities (structures, natural voids, mine workings, poorly compacted ground, geological structure and mineralization, depth to bedrock	Most commonly combined with intrusive investigations to determine the true nature of the density variations located by the technique via dynamic probing or boreholes. Combined with EM and magnetic techniques where they have indicated possible voiding or mine workings
Electrical resistivity tomography	Landfill, leachate, groundwater, contamination, bedrock, voids, geological stratigraphy and structure, fractures, mineralization, soil corrosivity assessment	Combined with seismic refraction to provide more robust ground models of depth to bedrock, water table, lateral and vertical variations in geology; combined with EM and TEM to provide depth control on anomalies identified using those techniques; combined with borehole data to provide robust and reliable information about geological made ground variations laterally and between borehole positions
Seismic refraction	Depth to bedrock, geological structure and stratigraphy, water table, landfill depth, rock rippability	Combined with electrical resistivity tomography as described above; combined with MASW to indicate variations in material type (e.g., granular or clay rich) and to constrain layer position in the inversion of MASW data; combined with borehole data to provide robust and reliable information laterally away from borehole positions
Multichannel analysis of surface waves (MASW)	Ground stiffness (Gmax) and seismic response parameters (Vs30), bedrock, geological stratigraphy, poorly consolidated or weak ground, voids, ground improvement verification	Combined with refraction and resistivity surveys as described above; borehole and probe data and downhole geophysical logs including seismic and sonic measurements can be used to provide constrains on the layered ground model produced with MASW data then providing continuity of information away from borehole positions; combined with targeted dynamic probing, plate bearing tests, or borehole to obtain further information about anomalous areas identified by the MASW data
Seismic reflection	Geological mapping of stratigraphy and structure, water table, mine workings, and natural voids	Combined with seismic refraction data especially in determining depth to bedrock and shallow velocity structure; commonly combined with borehole logs and downhole geophysical data to constrain the processing of seismic reflection data and the interpretation of a robust ground model

Cross-References

▶ Aeromagnetic Survey
▶ Aquifer
▶ Artificial Ground
▶ Bedrock
▶ Boreholes
▶ Brownfield Sites
▶ Conductivity
▶ Contamination
▶ Cross Sections
▶ Density
▶ Designing Site Investigations
▶ Engineering Geological Maps

G

▶ Engineering Properties
▶ Excavation
▶ Faults
▶ Geological Structures
▶ Groundwater
▶ Hydrogeology
▶ Karst
▶ Landfill
▶ Mechanical Properties
▶ Mining Hazards
▶ Modulus of Elasticity
▶ Physical Weathering
▶ Pipes/Pipelines
▶ Risk Assessment
▶ Rock Properties
▶ Sinkholes
▶ Site Investigation
▶ Soil Properties
▶ Subsurface Exploration
▶ Surveying
▶ Velocity Ratio
▶ Voids
▶ Waste Management

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Geopolymers

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Synonyms

Alkali-activated material; Inorganic polymer glass; Inorganic polymers; Mineral polymers; Soil cement

Definition

A broad class of materials produced by the reaction of an alkali solution and an aluminosilicate powder which can bind other materials (e.g., aggregate) into a hardened, cohesive mass.

In contrast to traditional cements, the aluminosilicate powder used in geopolymers is not *hydraulic* (i.e., does not harden through the addition of water) and contains little, if any, calcium oxide. Consequently, the aluminosilicate powder must be *activated* by reaction with an alkali solution to form a hardened binder. Geopolymers exhibit technical characteristics which often meet or exceed those of traditional Portland cement, with far lower CO_2 emissions. Consequently, the dominant application for geopolymers is in use as a sustainable construction material. Other applications include use in zeolite synthesis, sol-gel processing, radioactive waste immobilization, biomaterials, fiber composites, and as low-cost refractories.

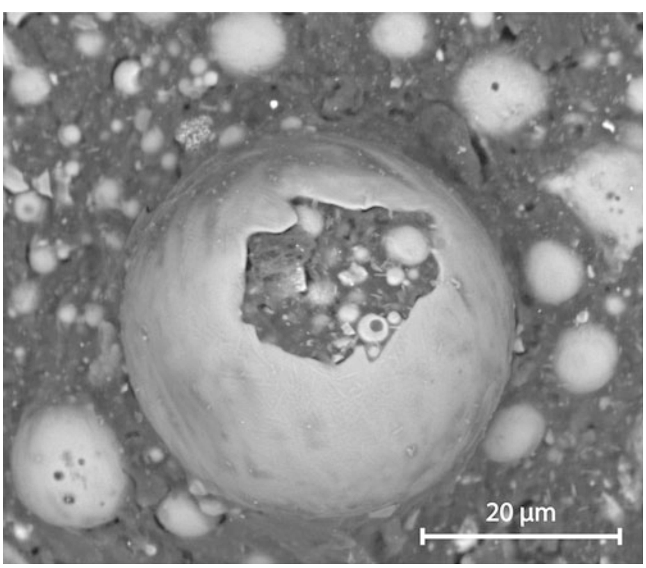
Characteristics

Geopolymers are produced by reaction of an alkali solution and an aluminosilicate powder, typically under ambient conditions. The aluminosilicate powder is most commonly sourced from siliceous industrial by-products (e.g., fly ash) or calcined clay (e.g., metakaolin) and typically contains little, if any, calcium oxide. The alkali source is typically a solution of sodium or potassium silicate or hydroxide. The reaction between the alkali solution and the aluminosilicate powder (commonly referred to as the *alkali activation* process) is exothermic (Palomo et al. 1999).

The primary reaction product is an alkali aluminosilicate hydrate gel framework with a highly cross-linked, long-range disordered (X-ray amorphous), pseudo-zeolitic structure (Provis and van Deventer 2009). This hydrate gel (abbreviated either N-A-S-H or K-A-S-H, as Na and K are the most common alkalis used) is the phase responsible for the binding and strength characteristics of hardened geopolymers. The term geopolymer itself arises from comparisons of the pseudo-zeolitic (i.e., geological; *geo*) yet polymerized (*polymer*) nature of the inorganic hydrate gel with that of organic polymers.

Al and Si are both present in tetrahedral coordination, analogous to their roles in zeolitic aluminosilicate frameworks, with Si bound via oxygen bridges to m Al atoms and $(4 - m)$ Si atoms, where m is a whole integer between 1 and 4 depending on the Si/Al ratio of the gel (typically $1 \leq \mathrm{Si/Al} \leq 4$). Al is predominantly bound to four other Si atoms due to the energetic penalty associated with Al^{IV}–O–Al^{IV} bonding (Provis et al. 2005). The negative charge associated with Al substitution for Si is balanced by the alkali cations. This nanostructure can be significantly affected by kinetic limitations on silica and alumina release from solid precursors used in gel synthesis and consequently evolves over time as the *alkali activation* reaction proceeds. The pH of the pore solution in geopolymers is highly alkaline, typically greater than 13 (Fig. 1).

Due to the absence of the need for calcination of limestone and other differences in production methods, geopolymers exhibit desirable environmental characteristics, with an 80–90% reduction in associated CO_2 emissions when compared with traditional Portland cement (Habert and Ouellet-Plamondon 2016). By control of mix formulation, geopolymers can be designed to exhibit high compressive strength, high durability, low shrinkage, low density, macro- or nano-porosity, acid resistance, fire resistance, freeze-thaw resistance, and controlled setting (Duxson et al. 2007). Consequently, geopolymers can be tailored for use in a wide variety of applications with demanding requirements.



Geopolymers, Fig. 1 Scanning electron micrograph of a sodium hydroxide-activated fly ash geopolymer, showing unreacted fly ash particles (brighter regions) embedded within the geopolymer matrix (darker regions) (Image courtesy of Dr. S.A. Bernal, the University of Sheffield)

Cross-References

- ▶ Aggregate
- ▶ Alkali-Silica Reaction
- ▶ Cement
- ▶ Clay
- ▶ Coal
- ▶ Concrete
- ▶ Geochemistry
- ▶ Infrastructure
- ▶ Limestone
- ▶ Waste Management

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Geostatic Stress

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Synonyms

Lithostatic stress; Overburden pressure

Definition

The weight of Earth materials in an imaginary vertical column acting on an imaginary horizontal surface at the base of the column.

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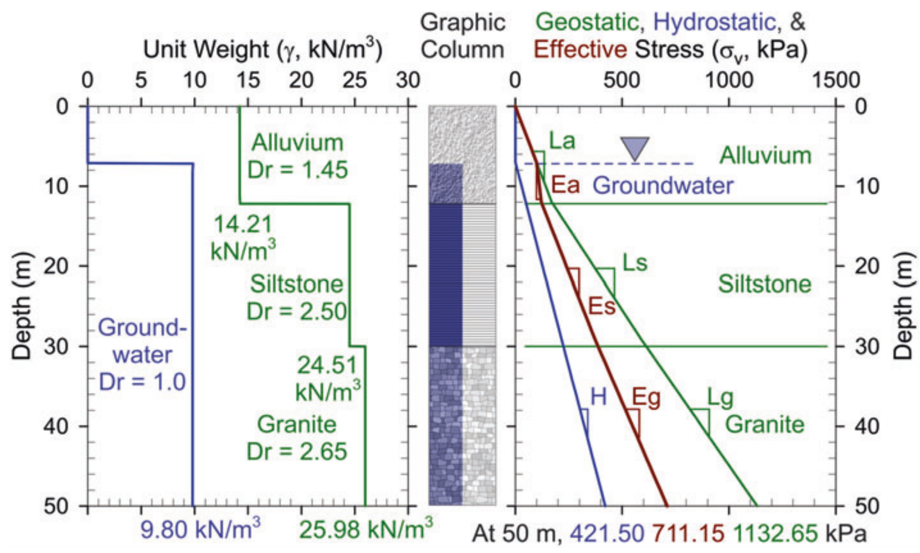

Geostatic Stress, Fig. 1 Unit weight and stress curves for a simple three-layer model of the earth. *Left*: Unit weights and relative densities (Dr) of earth materials and groundwater. *Right*: Geostatic (*green*), hydrostatic (*blue*), and effective stress (*red*) profiles. The slope of the geostatic stress in the earth materials is labeled by lithology (La, Ls, and Lg for alluvium, siltstone, and granite, respectively). The slope of the hydrostatic stress (H) is independent of earth material. The slope of the effective stress is Ea, Es, and Eg. Stresses at 50 m are listed at *bottom* of graph. *Middle*: Graphic column

A simple three-layer model of the Earth (Fig. 1) consists of 12 m of alluvium overlying 18 m of siltstone, overlying granite. An underground structure, such as a tunnel, may be planned at this location at a depth of 50 m, so the geostatic stress at 50 m needs to be computed. Assume that the relative densities (Dr) of alluvium, siltstone, and granite are as listed in Fig. 1; the Dr. of water is 1.0. Mass density is Dr times the mass density of water, which is 1000 kg/m^3. Force is mass times acceleration; weight is mass times the acceleration of gravity; thus, the unit weight of water is 9802.26 N/m^3. Hydrostatic pressure at a depth of 1 m would be 9.802 kPa (the slope of the hydrostatic pressure curve; H in Fig. 1). Therefore, at the base of a 1-m column of alluvium in the example, the unit geostatic stress would be 1.45×9.802 kPa $= 14.21$ kPa (the slope of the geostatic stress curve; La in Fig. 1); similarly, the unit geostatic stress in siltstone would be 24.51 kPa/m (Ls in Fig. 1), and the unit geostatic stress in granite would be 25.98 kPa/m (Lg in Fig. 1).

At a depth of 50 m in the geologic model, the geostatic stress is 1132.65 kPa. Effective stress is the geostatic stress minus the hydrostatic stress, because the hydrostatic stress acts equally in all directions, including opposite of the direction of gravity (Coduto 1999). Thus, the unit effective stress in the alluvium would be 14.21 kPa/m minus 9.80 kPa/m $= 4.41$ kPa/m (Ea in Fig. 1). Similarly the unit effective stresses in the siltstone and granite would be 14.71 (Es) and 16.18 kPa (Eg), respectively. Consequently, at a depth of 50 m in the geologic model, including the effects of groundwater, the effective stress would be 711.15 kPa.

Cross-References

- ▶ Density
- ▶ Effective Stress
- ▶ Engineering Properties
- ▶ Groundwater
- ▶ Normal Stress
- ▶ Pressure
- ▶ Rock Properties
- ▶ Soil Properties
- ▶ Stress
- ▶ Tunnels

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Geotechnical Engineering

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Definition

Geotechnical engineering is a discipline within civil engineering with a focus on the application of soil and rock mechanics. Geotechnical engineering includes the design structures that include or interface with soil and rock, such as excavations, foundations, earth-retaining structures, and structures composed of soil. In addition, geotechnical engineers contribute to the assessment of natural slopes and design the means of stabilizing these slopes.

Cross-References

- ► Engineering Geology
- ► Engineering Properties
- ► Excavation
- ► Mechanical Properties
- ► Rock Mechanics
- ► Soil Mechanics

Geotextiles

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Synonyms

Filter fabric

Definition

Permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain. The term "filter fabric" is more expressive, but it has fallen into comparative disuse. Geotextiles are really fabric materials used in a "geo," that is an Earth-related, engineering context.

The Classification of Geotextiles

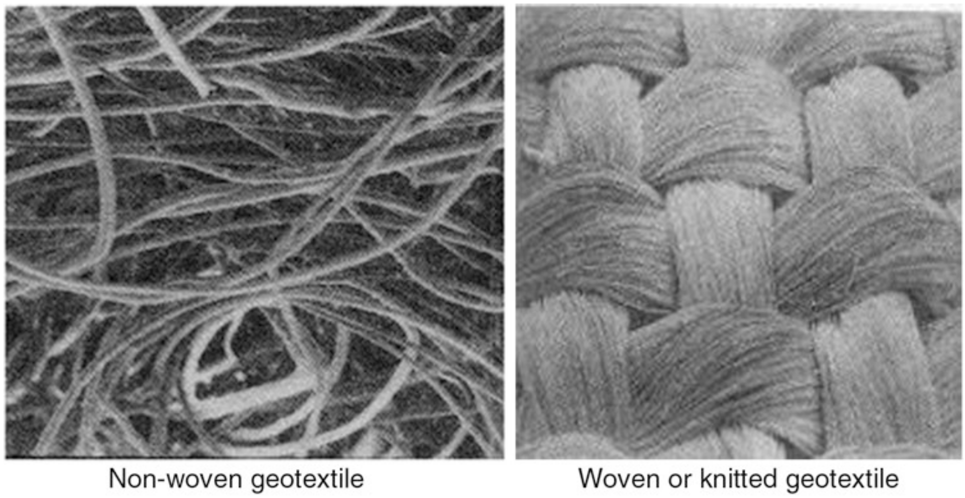
There are two principal geotextile or structure types: woven and nonwoven. Other manufacturing techniques, for example knitting and stitch bonding, are occasionally used in the manufacture of specialty products (The Bombay Textile Research Association 2012) (Fig. 1).

Geotextiles may be based on natural fibers or man-made synthetic fibers. The latter type of geotextiles are referred to as geosynthetics. Geosynthetic materials are generally not biodegradable, whereas natural geotextiles are often biodegradable. Biodegradability can be an advantage or a disadvantage, depending upon the user needs and use contexts. Geosynthetics are made from polymers of various types. Some common polymers used are polyethylene (PE), polypropylene (PP), polyvinylchloride (PVC), nylons, polystyrene (PS), and polyesters (Fig. 2).

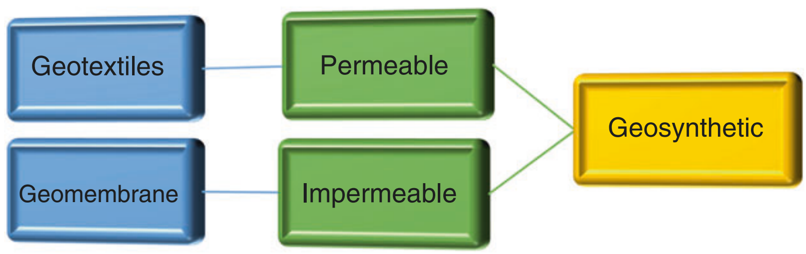
The other classification of geosynthetics is given as follows. Geogrids are plastics formed into open grid-like configuration. Geonets are formed by continuous extrusion of sets of polymeric ribs which when opened give rise to a net-like configuration. Geonets, geosynthetic clay liners (GCL), geogrids, geotextile tubes, and others can yield benefits in geotechnical and environmental engineering design. Geomembranes are very thin sheets of polymeric materials. GCLs are thin layers of bentonite clay sandwiched between geotextiles or geonets. Geopipes are simply buried plastic pipes and represent one of the oldest members of the family of geosynthetics. On the other hand, geocomposites are the newest member of the family and are composed of either geotextiles and geonets, or geotextiles and geogrids, or geogrids and geomembranes, etc. (Mohan and Nair 2005).

Application of Geosynthetics

With regard to geotechnical engineering, the function of geosynthetics is diverse (Müller and Saathoff 2015): (1) Barrier, to prevent the migration of liquids or gases; (2) Containment, to contain soil or sediments to a specific geometry and prevent their loss. The contained fill takes the shape of the inflated at-rest geometry of the geosynthetic container. (3) Drainage layers, to collect and transport fluids; (4) Filter layers, to allow passage of fluids from a soil while preventing the uncontrolled passage of soil particles; (5) Protection layers, to prevent or reduce as a localized stress reduction layer the damage to a given surface or layer. Geotextiles can improve soil strength at a lower cost than conventional soil nailing. In addition, geotextiles allow planting on steep slopes, further securing the slope. (6) Reinforcement, to resist stresses or contain deformations in geotechnical structures; (7) Separation layer, to separate two dissimilar geotechnical materials to prevent intermixing; (8) Surficial erosion controller, to prevent the surface erosion of soil particles due to surface water run-off and/or wind forces; (9) Frictional interlayer, which is a layer introduced within an interface with the purpose of increasing or reducing friction across the interface. These functions can be fulfilled sustainably with respect to production and transportation, easily with respect to handling and installation, as well as cost efficiently by the appropriately designed geosynthetic products. These are the reasons for the already very large and still growing market of geosynthetics (Table 1).



Geotextiles, Fig. 1 Types of geotextiles



Geotextiles, Fig. 2 Types of geosynthetics

Geotextiles, Table 1 Application of different types of geotextiles (The Bombay Textile Research Association 2012)

Type of geosynthetic	Separation	Reinforcement	Filtration	Drainage	Containment
Geotextile	√	√	√	√	
Geogrid		√			
Geonet				√	
Geomembrane					√
Geosynthetic clay liner					√
Geopipe				√	
Geofoam	√				
Geocomposite	√	√	√	√	√

Design Considerations

While many possible design methods or combinations of methods are available to the geotextile designer, the ultimate decision for a particular application usually takes one of three directions: design by cost and availability, design by specification, or design by function. Key design properties of interest include the following: (1) Physical properties – includes specific gravity, mass per unit area of the geotextile, thickness, and stiffness; (2) Mechanical properties – includes compressibility, tensile strength, seam strength, burst strength, and elongation; (3) Hydraulic properties – includes porosity, percent openings in the geotextile, an equivalent opening size, permeability, and soil retention; (4) Endurance properties – includes potential for damage during installation, long-term strength loss, abrasion potential, and clogging potential; and (5) Degradation properties – includes potential for degradation from sunlight, temperature, oxidation, biological action, and hydrolysis (Mohan and Nair 2005).

Geotextiles and the Environment

Increasing environmental concerns regarding the protection of groundwater, atmosphere, waterways, and oceans have led to a massive awareness of the versatility of the available range of geosynthetic types that can provide secure long-term mitigation from harmful human and natural activities. The greatest evidence of geosynthetics assisting in protecting the environment is through the use of geomembranes and geosynthetic clay liners (GCLs) in many landfill applications.

Geosynthetics are a good choice for the environment as:

1. They are less resource intensive since they are lightweight, durable, and easy to install.
2. They safeguard our water as they can act as an impermeable layer designed to contain a wide variety of liquids including water, wastewater, and various chemicals.
3. They prevent soil contamination by acting as a filtration layer between the soil and groundwater.
4. They advance alternative energy production by allowing biogas to be collected, which can then be burned for fuel.

Cross-References

- ▶ Acid Mine Drainage
- ▶ Angle of Repose
- ▶ Characterization of Soils
- ▶ Collapsible Soils
- ▶ Compaction
- ▶ Compression
- ▶ Consolidation
- ▶ Contamination
- ▶ Dewatering
- ▶ Engineering Properties
- ▶ Erosion
- ▶ Expansive Soils
- ▶ Factor of Safety
- ▶ Filtration
- ▶ Foundations
- ▶ Geopolymers
- ▶ Geotechnical Engineering
- ▶ Ground Anchors
- ▶ Grouting
- ▶ Hydraulic Action
- ▶ Hydrocompaction
- ▶ Infiltration
- ▶ Landfill
- ▶ Landslide
- ▶ Lateral Pressure
- ▶ Monitoring
- ▶ Noncohesive Soils
- ▶ Percolation
- ▶ Pore Pressure
- ▶ Retaining Structures
- ▶ Rock Mechanics
- ▶ Saturation
- ▶ Sediments
- ▶ Shear Strength
- ▶ Shear Stress
- ▶ Site Investigation
- ▶ Stabilization
- ▶ Subsidence
- ▶ Waste Management

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Geothermal Energy

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Definition

Heat energy generated from within the Earth.

Introduction

Geothermal energy is heat energy generated from within the confines of the Earth. The word geothermal comes from the Greek word "γ," which means Earth, and the Greek word "θερμός," which means hot. The heat from the Earth can be used in the forms of hydro-geothermal resources (steam or hot water) and petro-thermal resources (hot dry rocks) for heat supply or to produce electricity. Geothermal energy is a renewable "green" source of energy because the heat of the Earth is practically unlimited and, mostly, environmentally friendly. Some benefits of geothermal energy are that it:

1. Provides clean and safe energy using little land
2. Is renewable and sustainable
3. Generates continuous, reliable base load power
4. Conserves fossil fuels and contributes to diversity in energy sources
5. Avoids imports and benefits local economies
6. Offers modular, incremental development and village power in remote sites

Thermal waters are used for many purposes – for development of the electric power, for central heating and cooling, for hot water supply, in agriculture, livestock industries, aquaculture, in the food, chemical and oil-extracting industries, in medicinal springs and spas, and for recreational purposes.

The heat of the Earth has been known since ancient times, mostly in areas with high heat flow due to volcanoes and geysers. The historical significance of geothermal energy use is reflected in myths, legends, and fairy tale stories of many peoples in the world (Svalova 1998, 1999).

For commercial production of electric power from geothermal energy, a geothermal reservoir is necessary that will provide hot water and steam resources that are required for the production of electricity. A geothermal reservoir is a large area where natural geothermal resources occur within the ground. For the commercial production of electricity, high-temperature reservoirs (with temperatures above 150 °C) that are located close to the surface (1–2 km from the surface) are most suitable. These places are usually found in areas near the tectonically active places where there is also high volcanic activity (Fig. 1).

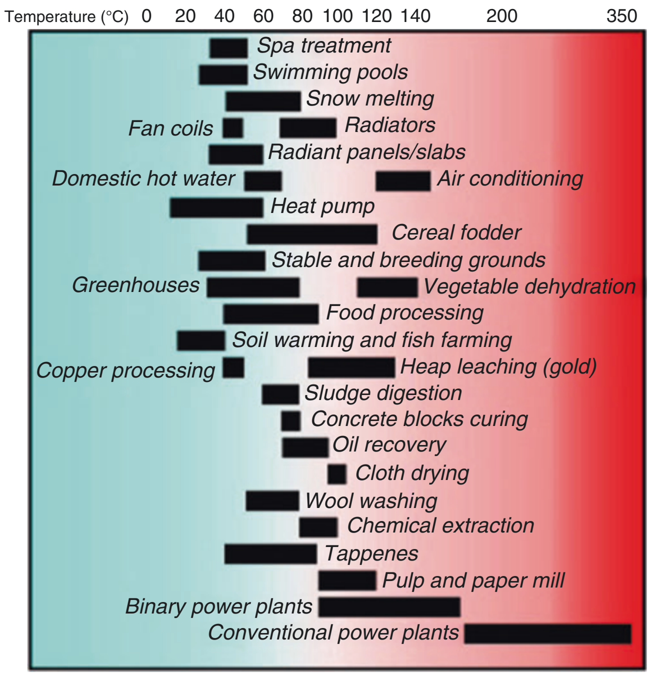
By the end of the nineteenth century, Prince Piero Conti conceived the idea to harness the natural steam of the Larderello, Italy, geothermal field to produce electric energy. He started to develop technical experiments and tests in 1903, and a year later the first geothermal electric energy was produced and used to light five lamps of 5 W. In 1913 Conti put in commercial operation a power plant of 250 kW fed by pure steam and in 1916 two power units of 3.5 MW each. Geothermal power production was born. The first modern geothermal power plants were also built in Larderello, Italy, but were destroyed in World War II and were rebuilt later. Today after 100 years, the Larderello field is still producing energy.

Geothermal Energy Utilization

Electricity generation is the most important form of utilization of high-temperature geothermal resources (>150 °C) (Povarov and Svalova 2010; Svalova 2010; Svalova and Povarov 2015). The medium-to-low temperature resources (<150 °C) are suited to many different types of application. Fluids at temperatures below 20 °C are rarely used, only in very specific conditions such as heat pump applications (Fig. 2) (Lindal 1973).

There are three different types of geothermal power plants commonly used for the commercial generation of electricity.

Geothermal Energy, Fig. 1 The tectonically active areas within which many areas have readily accessible geothermal resources, especially countries in the circum-Pacific "Ring of Fire" as well as plate spreading centers, continental rift zones, and other hot spots



Geothermal Energy, Fig. 2 Diagram of utilization of geothermal fluids

Dry steam systems use the steam from a geothermal reservoir in order to turn the turbines that generate electricity. Flash steam systems are the most common type of geothermal power systems. They use the high pressure hot water from a geothermal reservoir and convert it to steam in order to turn turbines that generate electricity. When the steam used to generate electricity cools down, it turns into water. The water is then sent back into the Earth to be reused. Binary cycle systems transfer the heat from the reservoirs hot water to another liquid. That other liquid is then turned into steam which in turn is used to turn the turbines that generate electricity.

Geothermal power plants work well in many countries (Table 1) (Bertani 2015), most successfully in the USA, the Philippines, Indonesia, Mexico, and New Zealand (Figs. 3 and 4).

Direct heat use is one of the oldest, the most versatile, and also the most common forms of utilization of geothermal energy (Table 2) (Lund and Boyd 2015). Bathing, space and district heating, agricultural applications, aquaculture, and some industrial uses are the best-known forms of utilization, but heat pumps are the most widespread. There are many other types of utilization, on a much smaller scale, some of which are unusual.

Geothermal heat pumps (GHP) are used in buildings for heating and cooling purposes. The heat pump pipes are inserted into the ground taking advantage of the ground's constant temperature throughout the year. The ground temperature below 10–15 m is maintained constant between 10 °C and 20 °C depending on the latitude. Also heat pumps can use heat from the air, water, polluted waters, and other sources.

Unlike the geothermal energy for the production of electricity which uses a geothermal reservoir and can be found in certain areas of Earth, geothermal heat pumps can be used all over the world (Fig. 5).

A special fluid with the property of warming up strongly when compressed (usually water with ethylene glycol) circulates inside the pipes. With the help of a compressor, evaporator, and condenser, the fluid warms up while circulating through the underground pipes and transfers the heat inside the building. The colder liquid inside the building is then transferred back into the ground in order to be circulated and warmed up again. Similarly, when the GHP system is reversed, it can be used to cool a building (Fig. 6).

The benefits of using a GHP system for heating and cooling purposes are:

- Low operational and predictable annual costs for the heating and cooling of a building. A GHP system can save up to 70% of the heating costs of a building.
- Low yearly maintenance costs for maintaining the GHP system.
- Long life expectancy. A typical GHP system's life is estimated to be 25 years for the components inside the building and more than 50 years for the pipes inside the ground.
- No outside exposed equipment.
- No environmental impact. According to the US Environmental Protection Agency, a GHP system is the most energy efficient and environmentally friendly system for a building's heating and cooling requirements.
- GHP systems are aesthetically better than conventional systems as they do not have large and visible components (like roof top equipment). They are also much more quiet and improve humidity control.

Before the installation of a GHP system, a number of design factors need to be taken into consideration, like the soil conditions, building's size, climate conditions, and type of loop of the GHP system that will be used (closed or open loop pipe system). The design and installation of a GHP system must be undertaken by a professional. A typical GHP system has high initial capital cost (CAPEX) but low operational costs (OPEX). The cost of a typical GHP system in the USA can vary from $8,000 up to $15,000 depending on all of the design factors specified above and with a payback period between 3 and 10 years.

Non-electric applications of geothermal energy give the installed capacity 70,329.0 MWt and energy use 587,786.4 TJ/year worldwide for the year 2015 (Table 2, Lund and Boyd 2015).

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Geothermal Energy, Table 1 Installed generating capacity (MWe)

Country	1990	1995	2000	2005	2010	2013	2015	Sources (2013 and 2015)
Total	5.8318	6.8668	7.9741	9.0641	10.7167	11.7720	12.6361	
Argentina	0.7	0.6	0	0	0	0	0	Bertani (2015)
Australia	0	0.2	0.2	0.2	1.1	1.0	1.1	Matek (2013) and Bertani (2015)
Austria	0	0	0	1.0	1.4	1.4	1.2	Antics et al. (2013) and Bertani (2015)
China	19.2	28.8	29.2	28.0	24.0	27.0	27.0	Matek (2013) and Bertani (2015)
Costa Rica	0	55.0	142.5	163.0	166.0	207.1	207.0	Cuéllar (2013) and Bertani (2015)
El Salvador	95.0	105.0	161.0	151.0	204.0	204.4	204.0	Cuéllar (2013) and Bertani (2015)
Ethiopia	0	0	8.5	7.0	7.3	8.0	7.3	Matek (2013) and Bertani (2015)
France (Guadeloupe and Alsace)	4.2	4.2	4.2	15.0	16.0	17.0	16.0	Antics et al. (2013) and Bertani (2015)
Germany	0	0	0	0.2	6.6	11.9	27.0	Antics et al. (2013) and Bertani (2015)
Guatemala	0	33.4	33.4	33.0	52.0	48.0	52.0	Cuéllar (2013) and Bertani (2015)
Iceland	44.6	50.0	170.0	322.0	575.0	664.4	665.0	Antics et al. (2013) and Bertani (2015)
Indonesia	144.8	309.8	589.5	797.0	1,197.0	1,341.0	1,340.0	Suryantini (2013) and Bertani (2015)
Italy	545.0	631.7	785.0	790.0	843.0	875.5	916.0	Antics et al. (2013) and Bertani (2015)
Japan	214.6	413.7	546.9	535.0	536.0	537.0	519.0	Matek (2013) and Bertani (2015)
Kenya	45.0	45.0	45.0	127.0	167.0	248.5	594.0	Mugo (2013) and Bertani (2015)
Mexico	700.0	753.0	755.0	953.0	958.0	1,017.4	1,017.0	Personal data provided by Luis Gutierrez-Negrin and Bertani (2015)
New Zealand	283.2	286.0	437.0	435.0	628.0	842.6	1,005.0	Think Geoenergy Magazine (2013) and Bertani (2015)
Nicaragua	35.0	70.0	70.0	77.0	88.0	149.5	159.0	Cuéllar (2013) and Bertani (2015)
Papua New Guinea	0	0	0	39.0	56.0	56.0	50.0	Matek (2013) and Bertani (2015)
Philippines	891.0	1,227.0	1,909.0	1,931.0	1,904.0	1,848.0	1,870.0	Ogena and Fronda (2013) and Bertani (2015)
Portugal (Azores)	3.0	5.0	16.0	16.0	29.0	28.5	28.0	Antics et al. (2013) and Bertani (2015)
Romania	0	0	0	0	0	0	0.1	Bertani (2015)
Russia	11.0	11.0	23.0	79.0	82.0	81.9	82.0	Antics et al. (2013) and Bertani (2015)
Taiwan	0	0	0	0	0	0	0.1	Bertani (2015)
Thailand	0.3	0.3	0.3	0.3	0.3	0.3	0.3	Matek (2013) and Bertani (2015)
Turkey	20.6	20.4	20.4	20.4	82.0	166.6	397.0	Antics et al. (2013) and Bertani (2015)
USA	2,774.6	2,816.7	2,228.0	2,544.0	3,093.0	3,389.0	3,450.0	Matek (2013) and Bertani (2015)

The most common non-electric use worldwide (in terms of installed capacity) is heat pumps (70.95%), bathing (13.00%), space heating (10.74%), greenhouses (2.60%), aquaculture pond heating (0.99%), industrial processes (0.87%), cooling/snow melting (0.51%), agricultural drying (0.23%), and others (Lund and Boyd 2015).

Environmental Impact

There is no way of producing or transforming energy into a form that can be utilized by humanity without making some direct or indirect impact on the environment, including exploitation of geothermal energy. But there is no doubt that it is one of the least polluting forms of energy. In most cases the degree to which geothermal exploitation affects the environment is proportional to the scale of its exploitation. Electricity generation in binary cycle plants affects the environment in the same way as direct heat uses. The effects are potentially greater in the case of conventional back-pressure or condensing power plants, especially as regards air quality, but can be kept within acceptable limits (Dickson and Fanelli 2005).

The initial environmental effect is that drilling, whether the boreholes are shallow ones for measuring the geothermal gradient in the study phase, or exploratory/producing wells. But the impact on the environment mostly ends once drilling is completed.

Environmental problems also arise during plant operation. Geothermal fluids (steam or hot water) usually contain gases such as carbon dioxide (CO_2), hydrogen sulfide (H_2S), ammonia (NH_3), methane (CH_4), and trace amounts of other gases, as well as other dissolved chemicals, the concentrations of which usually increase with temperature. Some geothermal fluids, such as those utilized for district heating in Iceland, are freshwaters, but this is very rare. Waste waters from

Geothermal Energy, Fig. 3 Mutnovsky GeoPP (geothermal power plant), Kamchatka, Russia. The installed capacity of this installation for the first stage is 50 MW

Geothermal Energy, Fig. 4 Mutnovsky GeoPP (geothermal power plant), Kamchatka, Russia. Primary separators provide MGeoPP with the high-quality steam (Photo by Svalova V.)

geothermal plants also have a higher temperature than the ambient environment and can therefore be a potential thermal pollutant.

Air pollution may also be a problem when generating electricity in conventional power plants. Hydrogen sulfide is one of the main pollutants.

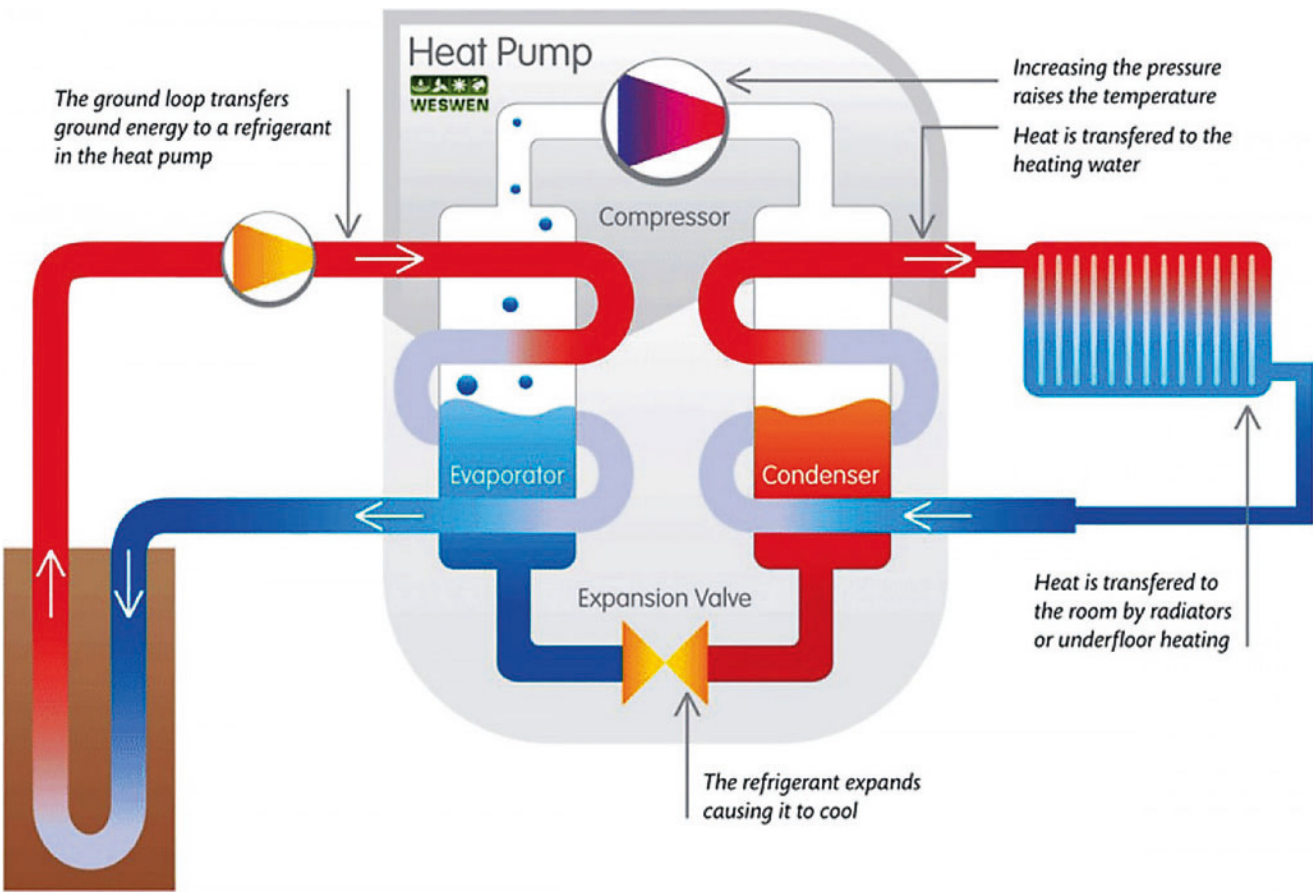
Discharge of waste waters is a potential source of chemical pollution. Spent geothermal fluids with high concentrations of chemicals such as boron, fluoride, or arsenic should be treated, reinjected into the reservoir, or both. However, the low-to-moderate temperature geothermal fluids used in most direct-use applications generally contain low levels of chemicals, and the discharge of spent geothermal fluids is seldom a major problem.

Extraction of large quantities of fluids from geothermal reservoirs may give rise to gradual subsidence of the land surface. This is an irreversible phenomenon but by no means catastrophic, as it is a slow process distributed over large areas.

The withdrawal and/or reinjection of geothermal fluids may trigger or increase the frequency of seismic events in certain areas. However, these are microseismic events that can

Geothermal Energy, Table 2 Top ten countries for highest direct uses, 1995–2015

Country	Power (MWt) 1995	Energy (TJ/year) 1995	Power (MWt) 2000	Energy (TJ/year) 2000	Power (MWt) 2005	Energy (TJ/year) 2005	Power (MWt) 2010	Energy (TJ/year) 2010	Power (MWt) 2015	Energy (TJ/year) 2015
China	1,915.0	16,981.0	2,282.0	37,908.0	3,687.0	45,373.0	8,898.0	75,348.3	17,870.0	174,352.0
USA	1,874.0	13,890.0	3,766.0	20,302.0	7,817.4	31,239.0	12,611.5	56,551.8	17,415.9	75,862.2
Sweden	47.0	960.0	377.0	4,128.0	3,840.0	36,000.0	4,460.0	45,301.0	5,600.0	51,920.0
Turkey	140.0	1,987.0	820.0	15,756.0	1,177.0	19,623.1	2,084.0	36,885.9	2,886.3	45,126.0
Iceland	1,443.0	21,158.0	1,469.0	20,170.0	1,791.0	23,813.0	1,826.0	24,361.0	2,040.0	26,717.0
Japan	319.0	6,942.0	1,167.0	26,933.0	413.4	5,161.1	2,099.5	15.698.0	2,186.2	26,130.1
Germany	32.0	303.0	397.0	1,568.0	504.6	2,909.8	2,485.4	12,764.5	2,848.6	19,531.3
Finland			80.5	484.0	260.0	1,950.0	857.9	8,370.0	1,560.0	18,000.0
France	599.0	7,350.0	326.0	4,895.0	308.0	5,195.7	1,345.0	12,929.0	2,346.9	15,867.0
Switzerland	110.0	3,470.0	547.3	2,386.0	581.6	4,229.3	1,060.9	7,714.6	1,733.1	11,836.8
Total	8,604.0	112,441.0	15,145.0	190,699.0	27,824.8	261,418.0	50,583.0	438,071.0	70,329.0	587,786.4

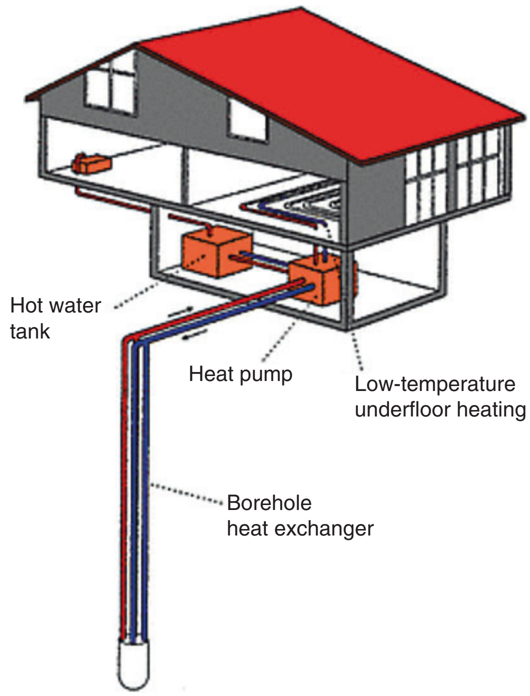


Geothermal Energy, Fig. 5 Scheme of heat pump

only be detected by means of instrumentation. Exploitation of geothermal resources is unlikely to trigger major seismic events.

The noise associated with operating geothermal plants can be a problem where the plant is generating electricity because, during the production phase, there is a high-pitched noise of steam travelling through pipelines and occasional vent discharges. But these are normally acceptable. At the power plant, the main noise pollution comes from the cooling tower fans, the steam ejector, and the turbine “hum.” The noise generated in direct heat applications is usually negligible.

All negative aspects, such as air quality pollution, surface water pollution, underground pollution, land subsidence, high noise levels, well blowouts, conflicts with cultural and



Geothermal Energy, Fig. 6 Typical application of ground-coupled heat pump system (Sanner et al. 2003)

archaeological features, social–economic problems, chemical or thermal pollution, solid waste disposal, and others, must be taken into account when evaluating, designing, and undertaking operations. With good design, operation, and regulation, the environmental problems can be dealt with.

Summary and Conclusions

The thermal energy present in the Earth is enormous. If exploited correctly, geothermal energy can certainly make an important contribution to the energy balance of many countries. In certain circumstances, even small-scale geothermal resources are capable of solving local problems and raising the living standards of small isolated communities. Together with other alternative energy sources (solar, wind, small rivers, tides, biofuel, and others), geothermal energy should contribute to an improved quality of life for many people.

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GIS

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Synonyms

Geographic information system

Definition/Overview

A Geographic Information System (GIS) is an organized set of (i) geolocated (also georeferenced) datasets in different formats, (ii) computer hardware, software, applications, and procedures that allow data capture, storage, and processing (Kennedy 2001), and (iii) people involved in using the system. These components once combined enable effective usage of geolocated data for finding answers and solutions to spatially based challenges. GIS is an integral part of the Geographic information technology that uses computer-based tools to analyze spatial information (Leblon 2013). The key advantage of GIS is that it offers summarized geographically related replies to the end-users' questions related to space such as "Where is it?" "What is it?" "Object's shape and relation to neighbors?" "Object's change through time?" and "What if. . .?"

In a general sense, the term GIS describes any information system that integrates, stores, edits, analyses, shares, and displays geographically defined information (Clarke 1986). The most basic principle characterizing GIS is that the information used is assigned to a specific geographic location.

Introduction

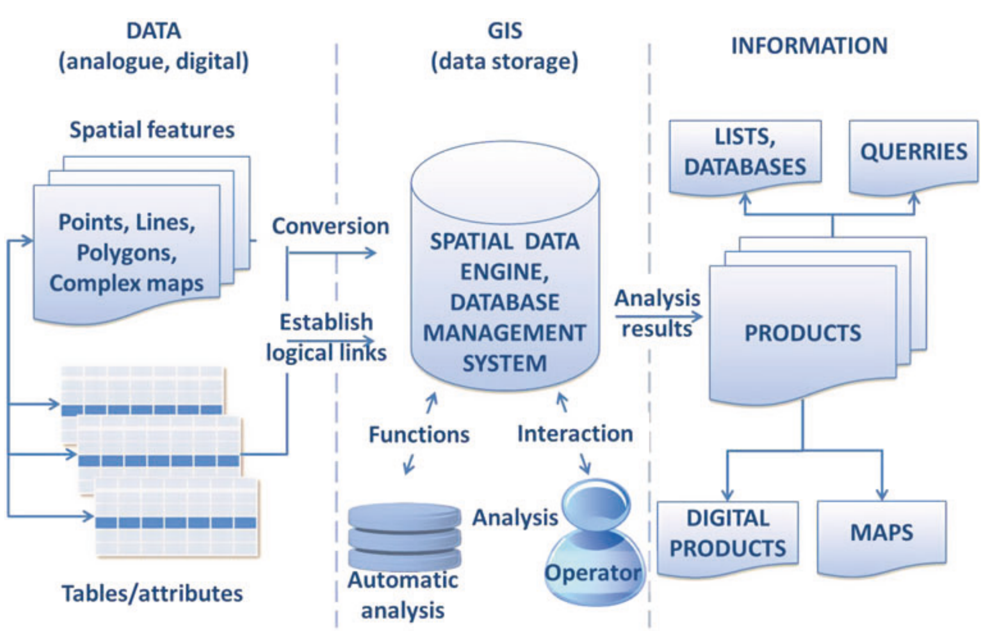
Geology, its domains and geoscience as a whole, like all other sciences dealing with the environment, depends heavily on spatially defined data. Hahmann and Burghardt (2013) estimated that almost 60% of existing data are spatially related (geolocated), yet in the environmental sciences this number would be much higher and closer to 95%. Consequentially it is almost impossible to imagine not using GIS and the advantages it offers in fields like geology, engineering geology, geohazards (Tarolli and Cavalli 2013), and other science branches, based on analyses of spatial relationships between observed occurrences, patterns between them, and potential trends in their behavior. To effectively utilize the GIS, the user has to clearly articulate the geographic question and to break it into manageable parts (Campbell and Shin 2012). The best GIS utilization will be achieved if manageable pieces of enquiry conceptually match with the basic constituents of GIS – entities and their inter-relations.

GIS Concept

GIS is a particular horizontal technology in the sense that it has wide-ranging applications across the industrial and intellectual landscape. Consequentially its best representation is a holistic model (Fig. 1), which shows that GIS stores spatial data with logically linked attribute information in a GIS storage database where analytical processes are controlled interactively by a human operator (or an automated system for generic results) with the aim to generate the required information products (after Tomlinson 2013).

Components of GIS

In addition to (i) georeferenced data that are composed of the graphical and the descriptive (attribute) components, there are also several other essential elements of the GIS. Whereas the data represent the digital content, this content needs to be stored and managed on (ii) hardware (computers, disks, servers, plotters, digital media, etc.) with the use of specific (iii) software packages – tools that enable capturing, viewing, modifying, editing, analyzing, and modeling of the GIS data. Tools operate under repetitive algorithms or (iv) processes that handle the GIS data and can be both predefined and automatic or can be governed/operated by a (v) human expert or nonexpert end user. In any case,



GIS, Fig. 1 GIS model (simplified after Tomlinson Associates Ltd. Consulting Geographers; Tomlinson 2013) processes are standardized to assure traceability and repeatability of the actions performed on or with the GIS data. Finally, the results are presented in different forms of (vi) products that can be either maps, lists, new databases, descriptive answers to one of the questions listed above, or just temporary displays, and these can be in a digital or paper format.

Types of GIS Data and Representation of the Real World

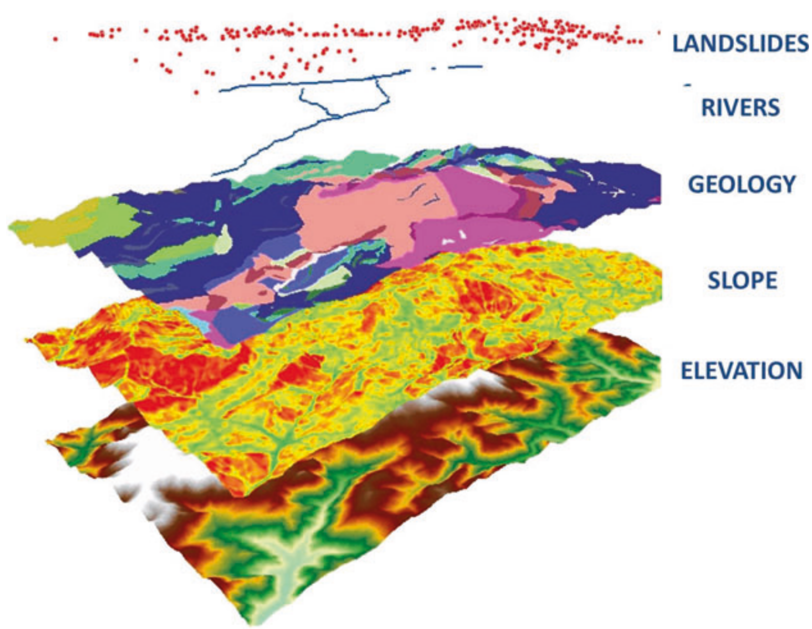
To effectively represent various spatial phenomena (i.e., geological units or parcels, faults or streets, a landslide, a borehole, or a customer's location), the GIS uses entities (objects) of different types that represent the graphical part of the GIS data. There are two types of data that are used in the GIS, vector, and raster data. For the vector-type data, the basic entities in GIS are points with no dimension, lines, or polylines with one and polygons with two dimensions (Tomlinson 2013). Points represent single location phenomena such as a location of a sample (location of a fossil or a soil sample), lines, and polylines represent linear phenomena such as geological boundary, a river, a fault, or measurement profile/cross-section, and polygons represent homogeneous areas (these are a simplified conceptual presentation of a real world) of different dimensions such as an engineering-geological unit, a larger landslide, or a flooding area. There are many other examples of spatial phenomena that could be simplified to this basic GIS object scheme. In addition to the abovementioned entity types, there are also several other entities used in GIS, such as bodies (with three dimensions), arclines (lines defined by curvature/arcs between them), and annotations or text. These entities alone do not yet present the GIS data. Only when they are linked to attributes (descriptions) that are stored in a form of tables or databases, do they become the real GIS data.

For the raster-type data (an image), the basic constituents are pixels that are usually arranged in a regular mesh of single value cells, which implies that each cell is a homogeneous area. A cell/pixel value represents either a continuous physical property (i.e., elevation, slope angle or inclination, a surface temperature, etc.) or a nominal value describing some type of a classified description of a given cell (i.e., land use, satellite image, topographic map, etc.).

Every single type of GIS data that represent a specific theme is called a GIS layer. When a layer is geographically positioned – put into a known coordinate system – it can be overlaid (Fig. 2) with other layers, and different analyses within and between these layers are then possible. In Fig. 2, raster data are represented by the elevation and the slope layers, whereas polygon vector data are represented by the geological units, linear vector data by rivers, and point vector data by landslide locations.

GIS Data Usage

From the analysis aspect, GIS data can be used in several ways. The most simple form of analysis is to perform a query with which the user searches through the spatial database for an attribute (term, phrase, number, etc.) or/and location (dimensions, spatial relations between entities) properties.



GIS, Fig. 2 GIS layers taken from a real case of landslide susceptibility modeling

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Advanced queries include search for dimensional (i.e., distance, vicinity, area, volume) properties and inter-relation between various layers. With a precondition of availability of layers that feature landslides, slope, and lithology, an example of a simple query would be: "Find all dormant landslides that lie on the slope with an angle between 7° and 15°, and that occur in marls." In GIS such a query would be expressed in the following (simplified) syntax: *"Landslides" = "Dormant landslide" AND ("SLOPE" > = "7" OR "SLOPE" < "15") AND "Lithology" = "Marls."*

Advanced GIS data analyses include various mathematic, statistic, logic, geometric, classification, and neighborhood functions (i.e., algebraic, trigonometric, filtering, extracting, etc.). Examples of such analysis would be reclassification of the original layer's values into new classes, application of an averaging filter or a calculation of a spatial density of an entity (i.e., density of faults per square kilometer in an area where a power plant is planned).

The most powerful use of GIS data is modeling that is used to derive interpolated results, either based on predefined functions or potential scenarios based of the basic question "What if...?" Examples of modeling would be calculating Darcy's flow, the flow accumulation, slope aspect, or slope inclination. In addition, experienced GIS users can develop advanced algorithms to model complex natural processes or phenomena occurrence (i.e., debris-flow susceptibility).

The abovementioned actions – queries, analyses, and modeling – have specific purposes. With integrated geographic information, derived results, and spatial models, new and advanced maps can be created in virtually real-time, events can be explained, outcomes predicted and impact estimated, strategies planned, ideas and scenarios visualized, and finally complicated problems could be solved with better potential impact inclusion and higher spatial and temporal accuracy.

Challenges of GIS

Currently more and more geographically related data are collected, and for this bulk of data to be manageable, the data need to be organized and stored in a GIS storage system, if possible, in a standardized and interoperable way. Properly ordered, standardized, and stored GIS data enable analyses, queries, and modeling. In addition to analyses, the properly stored GIS data enable easy and traceable editing and linking with other data, displaying, and serving the data or their quality archiving for the future usage. Maguire (1991) points out that the value of information depends upon timeliness, the context in which it is applied and the cost of collection, storage, manipulation, and presentation. In addition, the accuracy of GIS data is very important for the quality of the further use. Due to its digital nature, relatively easy illegal (re)use of and manipulation of GIS data represents a challenge from the legislative, and especially the intellectual property rights perspective.

Conclusions

GIS is an extremely useful tool for spatial analyses and modeling of phenomena occurrences, their consequences, and potential impacts on natural and anthropogenic environments. Large amounts of data collected on a daily basis, preferably georeferenced and in a standardized format, can effectively be assessed and analyzed only with computers and with specialized tools.

Cross-References

▶ Aerial Photography
▶ Aeromagnetic Survey
▶ Databases
▶ Engineering Geological Maps
▶ Engineering Geomorphological Mapping
▶ Floods
▶ Geohazards
▶ Hazard Assessment
▶ Land Use
▶ Landslide
▶ Lidar
▶ Modelling
▶ Monitoring
▶ Photogrammetry
▶ Remote Sensing
▶ Risk Mapping

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Glacier Environments

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Definition

Glacier environments are landscapes either directly associated with contemporary glaciers or those affected by former ice masses.

Introduction

Engineers require a basic understanding of alpine glaciers given their importance as freshwater sources for humans and ecosystem health. Glaciers and associated terrain also present a unique collection of natural hazards typically not found in other mountain environments. This chapter addresses alpine glacier environments, their associated processes, and the challenges that they pose for engineers and geoscientists.

Glaciers and Glacierized Terrain

A glacier is a perennial body of mostly ice, covered seasonally by snow that undergoes internal deformation (amended from Cogley et al. 2011). Glaciers are found at high elevations and latitudes, and range in size from hundreds of square meters to those that drain large sectors of the Greenland and Antarctic ice sheets (Fig. 1). Mass change of alpine glaciers including past and present-day ice sheets directly affect sea level with major implications for coastal engineering geology globally.

Glacierized terrain refers to landscapes directly affected by present-day glaciers. In the Northern Hemisphere most alpine glaciers reached their Holocene maximum extents about 200–300 years ago during the “Little Ice Age.” Substantial mass loss during the twentieth century exposed considerable areas of terrain within or adjacent to glacier margins. Other regions of glacierized terrain include the ice caps of the Canadian Arctic Archipelago and the ice sheets of Greenland and Antarctica

Glacier Mass Balance

Most alpine glaciers are nourished by winter snowfall and depleted by summer melt. Mass accumulation can also occur through avalanching or wind transport of snow onto their surfaces, whereas calving into lakes and oceans represent additional ways to lose ice mass. In arid environments, particularly with high wind speeds, sublimation can account for substantial ice mass loss (Rabatel et al. 2011). Over time scales of years to decades, changes in mass lead to dimensional changes of the glacier which, under the current climate, has led to widespread glacier retreat.

Glaciers represent unique challenges for engineering geology. Nearby land use may affect the surface albedo of a glacier altering the amount of energy that can be used for melt (Rabatel et al. 2011). If this added debris is thin (millimeter scale) and nonreflective, it can exacerbate surface melting since it absorbs shortwave radiation and adds thermal energy to an underlying ice surface. Excessively thick debris added to a glacier either through landslides or mining or other process (Jamieson et al. 2015) can also cause glaciers to advance since this debris reduces surface melt and increases the total driving stress leading to tertiary creep (Fig. 2).

Ice near or at 0 °C and introduction of free water from surface melting allows many glaciers to slide along their beds. Basal sliding produces coarse and fine-grained sediment respectively through processes of quarrying and abrasion. These processes in addition to sediments originating from proglacial environments can cause bed and suspended sediment loads from glacier-fed streams to be much higher than ice-free ones (Church and Slaymaker 1989; Gurnell et al. 1996). Sediments introduced by such processes can elevate flooding hazards for glacier environments or complicate small-scale hydroelectric projects.

Glacier Hydrology

Runoff from glaciers and adjacent terrain differs from nonglacierized catchments. The thermal properties of glaciers allow snow to accumulate sooner and stay longer delaying runoff from these frozen surfaces. Glaciers can also temporarily store liquid precipitation or surface melt within unsaturated snow and firn (snow older than a year). In contrast, runoff from the lower sections of a glacier can be sudden as there are few opportunities to store runoff. At the catchment scale, ice disappearance can lead to flash runoff and thus more frequent exceedances of critical discharges capable of mobilizing higher volumes of channel bed sediment during extreme runoff events (Moore et al. 2009).

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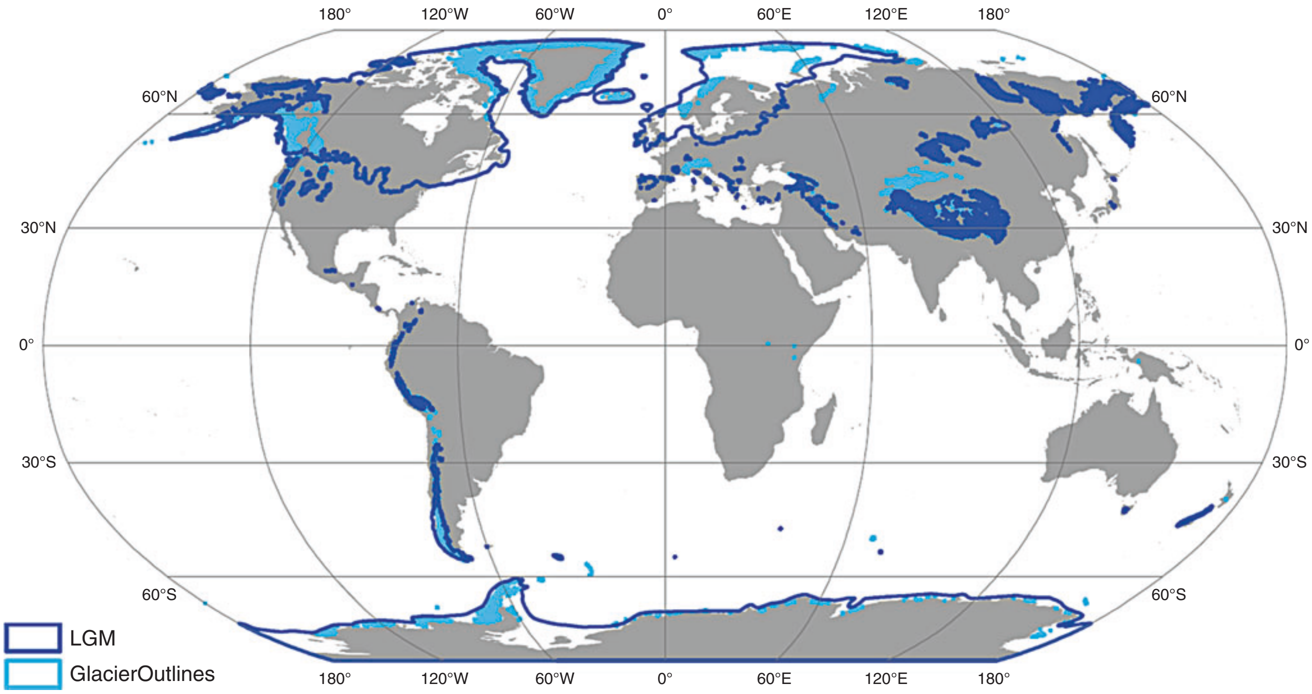
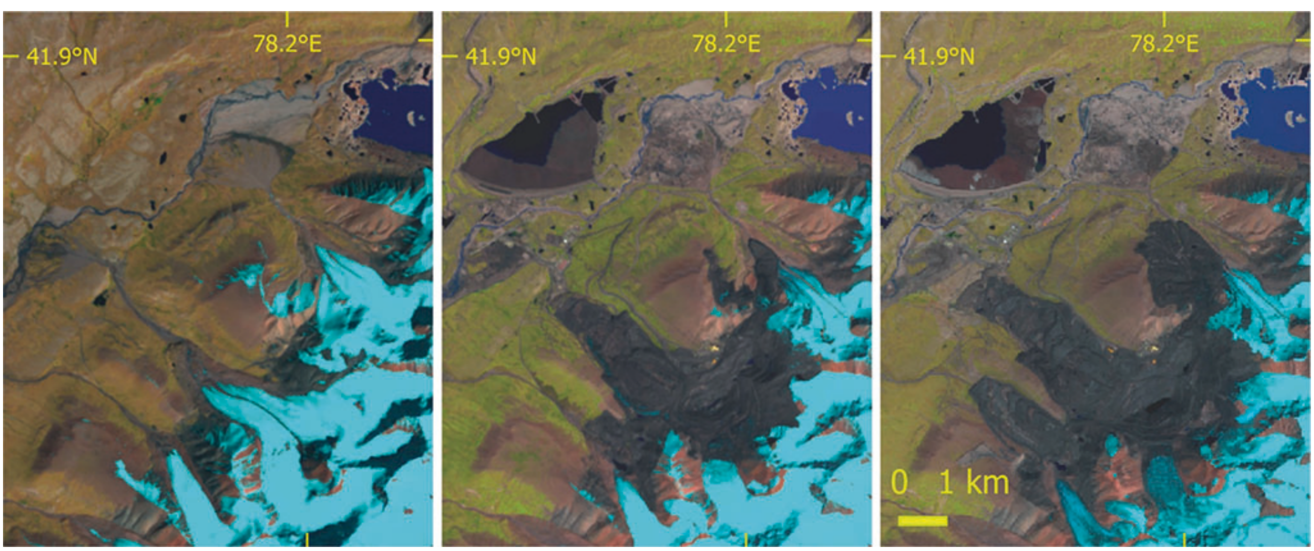
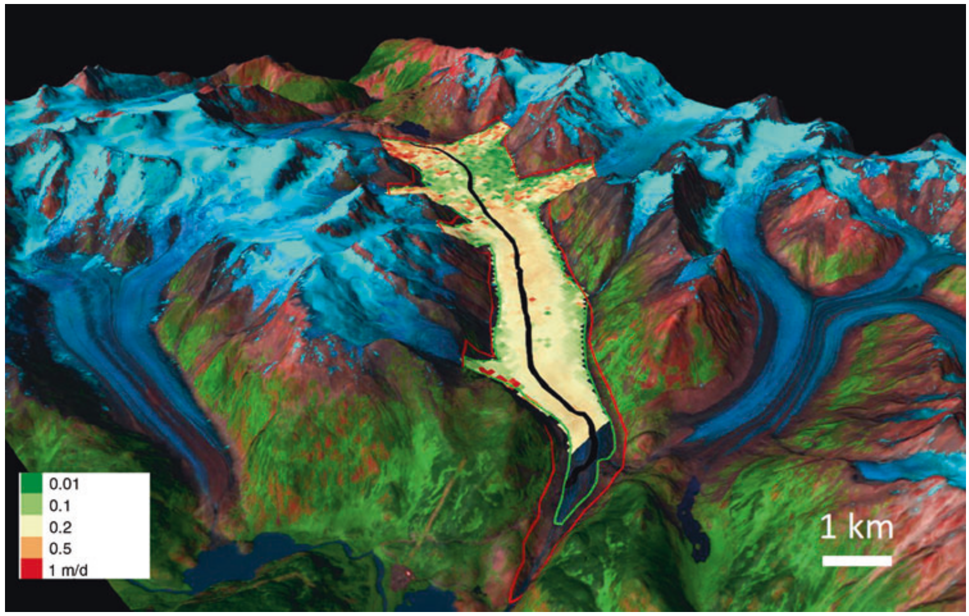


Glacier Environments, Fig. 1 Global distribution of glaciers (www.glims.org) and outline of Last Glacial Maximum



Glacier Environments, Fig. 2 Kumtar gold mine, Kyrgyzstan, showing glacier growth. *Left panel*: Landsat 5 TM 5 September 1993; *Center panel*: Landsat 8 OLI 31 August 2013; *Right panel*: Sentinel 2A MSI 31 August 2017

Glaciers also provide thermal buffering for many snow-fed streams during late summer or years of drought (Moore et al. 2009), so any changes in nearby land use can affect a glacier's surface mass balance and thereby the buffering capacity provided by alpine glaciers. Finally, glacier surfaces may provide one way to access areas of high mineral potential. For instance, the Brucejack Mine in northwestern British Columbia, Canada, has a 12.5 km long access road across the Knipple Glacier. Glacier travel is likely to become more common as glaciers retreat and mineral values are



Glacier Environments, Fig. 3 Glacier Road (*Black line*) to access the Brucejack Mine, British Columbia, Canada. Red line indicates the 1985 glacier outline (Landsat 5 TM) and the *green line* indicates the 2017 glacier outline (Sentinel-2A OLI). Colors indicate glacier velocities (m day^{-1}) derived from Planet Scope images (3 m resolution) from 2 August 2017 to 21 September 2017. Background image: Sentinel-2A September 5, 2017

increasingly exposed (Fig. 3). The nature of surface melt and ice flow present notable challenges for using such routes as permanent travel corridors. Surface melt may channelize and erode pathways vertically into the ice creating mill wells or moulins. Drainage networks beneath the ice can seasonally switch from distributed to channelized networks creating streams beneath the ice that thermally erode the ice towards the surface.

Hazards in Glacierized Terrain

Glaciers cause a number of hazards that need to be considered, especially in steep, mountainous terrain (see ▶ Mountain Environments).

Glaciers can also erode, widen, and deepen valleys. When this happens, stresses are imposed on the valley walls, but the ice still supports the slopes. In many cases, terrain adjacent to contemporary ice masses consists of steep, unvegetated slopes susceptible to landslides (Ballantyne 2002). Landslides can include debris flows triggered by snow or ice melt, or failures caused by intense precipitation events. Deep-seated failures can occur adjacent to glaciers once lateral support of the slope occurs during glacier retreat, a process known as debuttressing (Ballantyne 2002). When glaciers eventually thin and retreat, the valley walls are debuttressed, and the stress fractures can widen, progressively weakening rock masses. The slopes in their weakened states can deform slowly, fall repeatedly with small landslides, or fail catastrophically in the form of large rock slides.

Glacial retreat can create ice contact or proglacial lakes impounded by unstable landforms such as moraines. Moraine-dammed lakes are common in many glacierized mountainous regions of the world, and their failure presents notable hazards to communities living in such areas. The geotechnical characteristics of moraine-dammed lake make them prone to rapid incision and failure (Clague and Evans 2000). Some moraines are ice-cored or lie within permafrost zones characterized by interstitial ice. These landforms and the water that they trap can be especially hazardous since projected climate warming could melt interstitial ice making the ice dam unstable. Rock or icefall or calving of the glacier terminus into moraine-dammed lakes may lead to the generation of a displacement wave that can overtop the moraine dam (one way such dams fail) or through progressive piping within the moraine itself. These potentially destructive waves can extend the footprint of landslides by many kilometers. Remote sensing can be applied to entire mountain ranges to evaluate the hazard potential for potential glacier lake outburst floods (Huggel et al. 2002).

Outburst floods can also occur when water is suddenly released from storage sites adjacent to a glacier, on its surface or confined below its surface. These floods can be triggered by precipitation events, glacier retreat, subglacial eruption (e.g., 2010 Eyjafjallajökull eruption, Iceland) along with seepage erosion or overtopping in the case of moraine-dammed lakes. The pattern of drainage is similar to that of earthen dams, beginning slowly and continuously accelerating to a peak just before exhaustion of the impoundment. Thermal energy of the water and energy released via frictional dissipation leads to widening of the conduit for meltwater to escape.

Similar to landslide and moraine dams, a power-law relation exists between the peak discharges that can be generated from an ice-dammed lake of a given volume (Walder and Costa 1996). However, predictive models (Clarke 1982) using physically based approaches typically require detailed information about the thermal state of the ice, lake water, and conduit location.

Finally, glacier fluctuations may alter the flow directions of creeks and rivers leading to major changes in the delivery of sediment and water within mountain environments. In some cases, advancing glaciers can direct water into previously unoccupied river valleys, whereas marginal retreat of major glaciers along topographic divides can cause river piracy and complete river abandonment.

Glaciated Environments

Glaciated environment areas are those that were formerly covered by alpine glaciers or ice sheets. During the Quaternary, ice sheets repeatedly covered most land mass north of 50° North latitude and, as a response, global sea level fluctuated by over 100 m (see Fig. 1 for extent of Last Glacial Maximum). Although ice may not exist in these landscapes, the legacy of glaciation continues to influence sediment transfers in these settings. Rates of slope and river adjustments scale to the time since deglaciation and size of the drainage basin in question (Ballantyne 2002). Glaciers erode and deepen valleys, but also accumulate thick sediment packages. Deep fills may be deposited in glacial environments, including till, and glaciolacustrine and glaciomarine sediments. Some of the most problematic engineering soils include: (i) pre-sheared glacial lake sediments that have been covered by glaciers and are now overlain by till, and (ii) isostatically uplifted glaciomarine sediments that have developed sensitivity, to the extents that some can be classified as quick clays (see ▶ Quick Clay). These types of sediments may be predisposed to large, low-gradient landslides.

Deposits and Sediments

One of the greatest challenges of dealing with glacierized environments is the heterogeneity in the nature and distribution of deposits. In some cases, for instance for ice contact deposits, sediment variation is exceedingly composition is highly varied covering the entire range of grain size (clay to large boulders); sorting and grading is equally diverse and unpredictable as are the facies and stratigraphic relationships, all of which is impossible to predict during site investigations (see ▶ Sequence Stratigraphy). This lack of predictability poses formidable challenges to engineering geologists dealing with construction and building requirements in these situations that must rely heavily on a sound and confident understanding of the sediment properties and characteristics. In contrast, glacierized environments are key sources for gravel, sand, and silt deposits, essential commodities in building and construction economies (see Aggregates). Landforms associated with this type of terrain are common over many parts of the far Northern Hemisphere.

Conclusion

Glacier environments, including glaciers and glacierized terrain, present unique challenges to engineering geology. The effects of glaciation and deglaciation may be observed downstream from glacier environments and may also be observed years, decades, or centuries following deglaciation. Glacier hazards are of increasing interest because of the growing industrial and recreational interest in glaciated environments. The geotechnical characterization of sediments from glacierized environments remains challenging due to the heterogeneity of the deposits.

Cross-References

- ▶ Aggregate
- ▶ Avalanche
- ▶ Clay
- ▶ Climate Change
- ▶ Geohazards
- ▶ Hazard
- ▶ Hazard Assessment
- ▶ Insar
- ▶ Landforms
- ▶ Landslide
- ▶ Lidar
- ▶ Loess
- ▶ Mass Movement
- ▶ Mountain Environments
- ▶ Permafrost
- ▶ Quick Clay
- ▶ Risk Assessment
- ▶ Risk Mapping
- ▶ Sand
- ▶ Sequence Stratigraphy
- ▶ Silt

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Gradation/Grading

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Definition

Gradation in geology refers to a landscape-scale concept and a characteristic of clastic sediments and rocks.

Grading in geology refers to a characteristic of clastic sediments and rocks.

Gradation in engineering refers to a calculated parameter of coarse-grained soils.

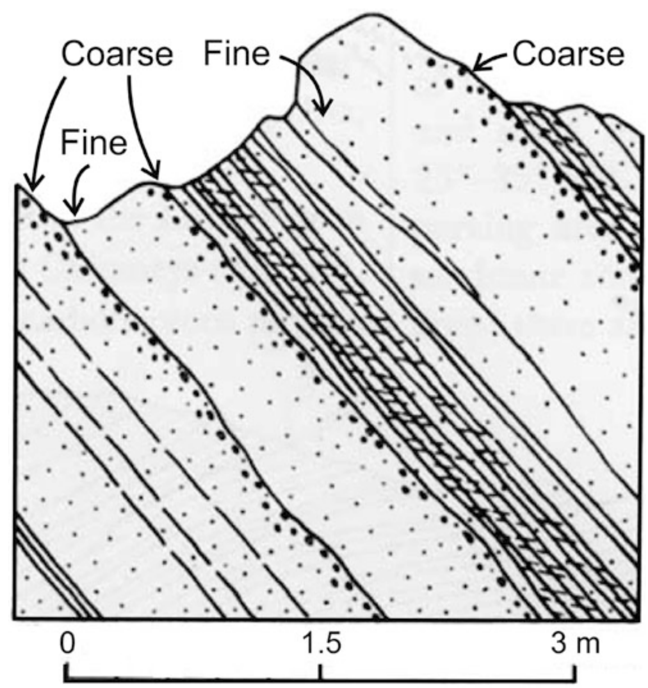
Grading in engineering is also a general term used to refer to earthwork activities and the result of those activities that involve excavation of soil and rock and placement of excavated material in a controlled manner as fill (e.g., cut and fill).

Gradation at a landscape scale is a general term for the continuous processes that operate at the Earth's surface that contribute to long-term, stable landscapes that are in equilibrium.

The equilibrium condition in landscapes is one in which Earth materials at some elevation tend to be broken into fragments or dissolved and transported to lower elevations where they are deposited or precipitated. Thus, high ground is worn away and low ground is buried in a landscape that is graded, meaning that "the mountains are shaped or graded in such a way that the products of decay of the bedrock can be moved across the ground surface and carried off in the channelways out of the area" (Hack and Goodlett 1960, pp. 57–58). All of the processes of weathering, erosion, transportation, and deposition are involved in the landscape gradation.

Gradation as a characteristic of clastic sediments and rocks refers to deposits that are interpreted to have happened under processes of normal Newtonian physics; in other words, under the influence of gravity. Clastic sediment particles carried by flowing water become deposited as the flow velocity slows, such that the coarser particles settle first and the fine particles settle last. Consequently in a normal alluvial (fluvial) environment, sandy and silty sediment is deposited with a graded, fining upward character. One aspect of field geology involving sedimentary rocks uses this graded bedding feature to determine facies (Fig. 1) for interpreting structural geologic relationships.

Grain size distribution in soils is a fundamental test performed for engineering classification. The shape of the



Gradation/Grading, Fig. 1 Sketch of the Thunderhead Sandstone in a sedimentary rock sequence in the Great Smoky Mountains National Park, eastern Tennessee, USA, showing inverted graded bedding leading to an interpretation that the sequence was overturned (Modified from a sketch by H.W. Ferguson and P.B. King in King et al. (1968, Fig. 5b))

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grain size distribution curve is characterized by the grain sizes of specific percentages of the soil deposit or soil sample. The term "Coefficient of Uniformity" describes the specific percentages for calculating that coefficient and for a companion coefficient of gradation (ASTM 2009). A poorly graded soil in engineering has a concentration of particles in a small range of grain sizes and is considered to be uniform. Conversely, a well-graded soil in engineering has a large range of grain sizes with no major concentration of grain sizes. In geology, however, the terms poorly graded and well graded are reversed and a dimensionless sorting coefficient is used to characterize the deposit or formation.

The term grading is used in engineering to refer to general earthwork activities of excavation of soil and rock and placement of excavated material as compacted fill. The results of earthwork activities are also referred to as grading (i.e., cut and fill).

Cross-References

▶ Aggregate Tests
▶ Characterization of Soils
▶ Classification of Soils
▶ Coefficient of Uniformity
▶ Compaction
▶ Cut and Fill
▶ Density
▶ Erosion
▶ Excavation
▶ Facies
▶ Filtration
▶ Fluvial Environments
▶ Infiltration
▶ Sedimentary Rocks
▶ Soil Laboratory Tests
▶ Soil Properties

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Ground Anchors

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Synonyms

Rock anchors; Soil anchors

Definition

Prestressed tension elements installed in drill holes to resist tension forces in the ground.

Ground anchors are installed in drill holes, usually 50–100 mm in diameter and grouted. They are actively prestressed to a properly designed tension force (Fig. 1).

Nails are similar to ground anchors, but are *passive*, that is, they are not prestressed. They do not have an unbonded length. Nails only mobilize tension when the ground tends to deform.

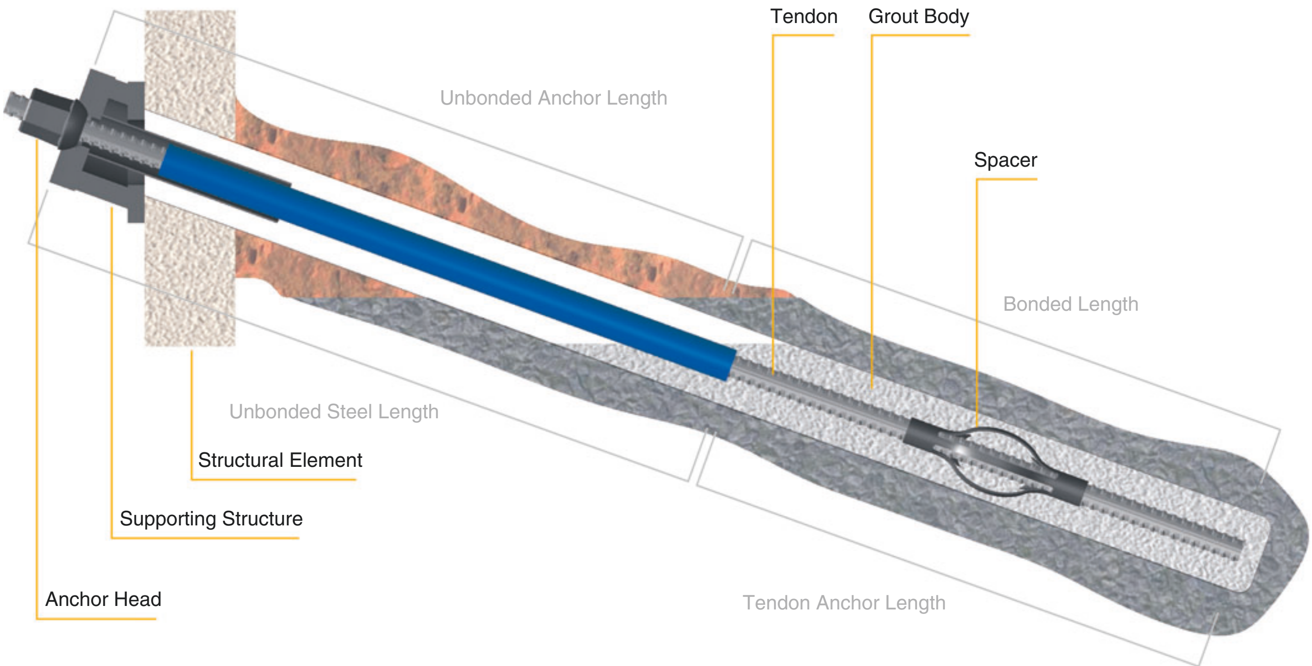
Characteristics

Anchors have the following elements (Ortigao and Brito 2004):

- *Anchor head* is the anchor-end outside the ground, comprising a bearing plate and a nut.
- *Bonded length* is the length that transmits tension forces to the ground.
- *Unbonded length* is the frictionless length between the head and the bonded length that transmits tension load from the bonded length to the head.
- *Spacers* are plastic devices to keep the tendon centered in the drill hole.

This entry addresses single bar inclusions only. Experience in using multistrand anchors has been unsuccessful due to inadequate corrosion protection and many reported failures. This has led to many standards organizations to ban multistrand anchors for permanent structures.

Nails differ from anchors in two ways: there is no free length and they are not prestressed. Ortigao and Palmeira (1997) discuss the design of soil-nailed walls and slopes. Ortigao and Brito (2004) present a comprehensive review of anchors and nails, as well as installation methods and corrosion protection for permanent ground anchors.



Ground Anchors, Fig. 1 Ground anchor (By courtesy of Dywidag)

Cross-References

▶ Rock Bolts

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Ground Motion Amplification

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Synonyms

Local ground response; Site amplification; Site effects; Site response

Definition

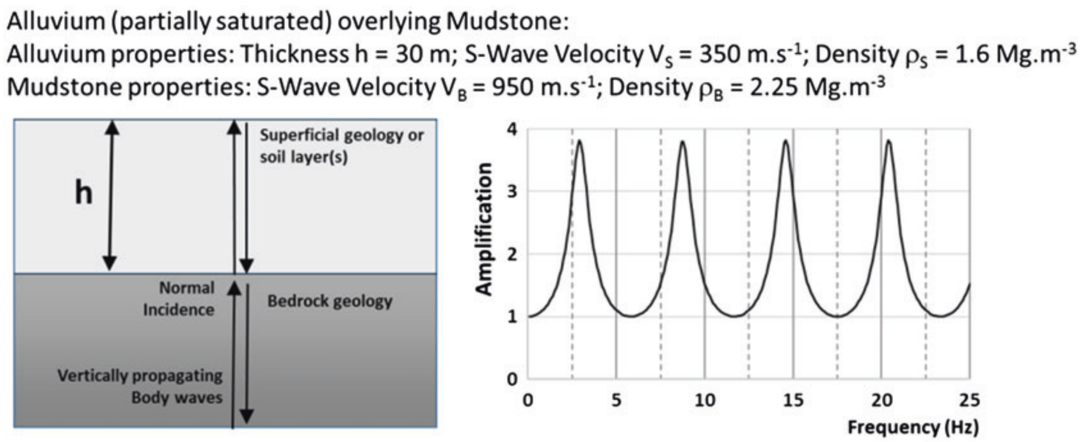
The increase in amplitude of seismic waves as they pass from faster to slower geological units, plus increases in amplitude due to interactions within more complex structures containing soft sediments.

Ground motion amplification (GMA) is defined as the ratio of the ground motion at the surface of the surficial (or soil) outcrop with respect to the ground motion at bedrock outcrop at the same site. Quantitative GMA factors can be defined using any ground motion parameter, but GMA is most commonly expressed as the ratio of the spectral accelerations at the specific time period of these motions (Kramer 1996; Bowden and Tsai 2017).

Amplification Processes

A horizontally layered sequence with an abrupt increase of shear wave velocity and/or density will cause ground motion amplification. Thus, alluvial basins, deltas, and lake deposits are places where one would expect GMA during earthquake induced ground motion. In this general case, GMA relates to upward propagating, body waves passing into a horizontal layer of lower velocity than the region below.

Seismic body waves (P and S) from a distant earthquake refract into shallow geological structures and reflect between the ground surface and the top of bedrock (Fig. 1). Most surficial materials behave elastically at shear strains $<10^{-5}$, and in the case of normal wave incidence, the fundamental spectral GMA ratio is given by Bolt (1970) as a function of frequency, f:



Ground Motion Amplification, Fig. 1 Amplification of body shear waves at surface of superficial geological layer relative to bedrock. Example of thick alluvial deposit over mudstone

$$\frac{A_S(f)}{A_B(f)} = \left[\left(\frac{\rho_s.v_S}{\rho_B.V_B}\right)^2 + \left\{1 - \left(\frac{\rho_s.v_S}{\rho_B.V_B}\right)^2\right\}\cos^2(k.h)\right]^{-0.5} \tag{1}$$

where ρ_S and ρ_B are the densities, and V_S and V_B the seismic velocities of the surficial and bedrock geology respectively, and $k =$ the wave number for the surficial layer, given, $k = \frac{2\pi f}{V_S}$.

The minimum value of the GMA ratio is 1 and the maximum value is $[(\rho_B.V_B)/(\rho_S.V_S)]$, where when the surficial layer behaves elastically, the maximum occurs at a series of time periods, T_m, satisfying,

$$T_m = \frac{4.h}{[(2.m+1).V_S]} \ldots\ldots \text{for the Eigen factor values}, m = 0, 1, 2, 3.. \tag{2}$$

Equation 1 can provide a useful estimate of the site amplification effects, but further factors such as increased damping of motion at greater strains (Abrahamson and Silva 2008) and higher frequencies (Murphy et al. 1971) generally lead to reduced amplification with increasing Eigen factor values.

GMA is also caused by lateral interference of body waves obliquely incident on the horizontal layer (Aki and Richards 2002), and also, of surface waves (Bowden and Tsai 2017) propagating within basin structures bounded by abrupt lateral velocity changes. Amplification of surface waves becomes increasingly important with closer proximity to shallower hypocenters, such as the Los Angeles Basin – San Andreas Fault System in California, where greater amplification of surface waves can occur at lower time periods.

The effect of local topography on GMA is quite complex, and the amplification factor is largely influenced by the lateral scale of the topographic feature, that is, valley or ridge width in relation to seismic wavelength. Generally, the amplification due to a ridge is greatest at wavelengths corresponding to the half-width of the ridge, whereas maximum amplification occurs at valley edges for wavelengths around twice the valley width (Geli et al. 1988).

GMA is affected by the local geological stratigraphy and structure, and the local topography. The effects of local stratigraphy are most well-known, and GMA can be expected wherever poorly consolidated sediments overlie crystalline or cemented bedrock. This effect is usually modeled using the nonlinear elastic wave propagation through the surficial sequence, for example, using modelling packages such as SHAKE91 (Schnabel et al. 1972).

Further amplification occurs due to the complex interference of body and surface waves propagating within shallow structures into which surficial materials are deposited. As the interference varies laterally across the structure, then so too does the amplification. Ridges and valleys also cause interference within the seismic propagation field, further affecting the local distribution of amplification. The effects of shallow structures and local topography require specific modelling, using more detailed analytical or numerical analysis methods (Vai et al. 1999).

Cross-References

▶ Bedrock
▶ Characterization of Soils
▶ Clay
▶ Collapsible Soils
▶ Compaction

▶ Compression
▶ Consolidation
▶ Earthquake
▶ Earthquake Intensity
▶ Earthquake Magnitude
▶ Effective Stress
▶ Elasticity
▶ Environments
▶ Factor of Safety
▶ Failure Criteria
▶ Faults
▶ Geohazards
▶ Ground Shaking
▶ Hazard
▶ Hazard Assessment
▶ Hazard Mapping
▶ Induced Seismicity
▶ Liquefaction
▶ Probabilistic Hazard Assessment
▶ Quick Clay
▶ Risk Assessment
▶ Risk Mapping
▶ Sand
▶ Shear Strength
▶ Soil Properties
▶ Stress
▶ Surface Rupture
▶ Tsunamis

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Ground Preparation

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Definition

A general term used to describe activities needed at a site that will undergo earthwork, grading, fill placement, or construction of foundations for a structure, including a building, dam, pipeline, tank, or other facility.

Site preparation includes ground preparation, as well as development of access roads, laydown and staging areas, water supply, power supply, waste handling and disposal, and other necessary utilities.

Ground preparation for site grading intended to prepare a level area, or a series of level terraces separated by intermediate slopes or walls, for construction of a building begins with clearing and grubbing (Coduto et al. 2011), which consists of removal of vegetation, trash, buried objects, construction debris, highly organic soils, large rocks, and other undesirable or deleterious materials (Figs. 1 and 2). Once the initial clearing has been accomplished, additional ground preparation activities include scarifying the ground surface to a depth of 150–200 mm, applying water for dust control and moisture conditioning, and proof-rolling with a heavy, self-propelled vehicle or trailer on pneumatic tires (e.g., a loaded dump truck (Fig. 2) or a water-filled tank on inflated rubber tires). Areas of soft or loose soil revealed as depressions or ruts caused by the rubber tires are excavated, scarified to a depth of 150–200 mm, conditioned with water, and proof-rolled again. When a base surface shows consistent firm conditions without depressions or ruts, the site surface is considered to be prepared for fill placement and compaction.

In areas where sinkholes, or Earth fissures, or shallow mines are known or suspected to exist, additional measures are needed for ground preparation. A geotechnical site investigation is needed to determine soil deposit characteristics, including grain size distribution so that filter material may be specified for placement in the sinkholes or along the Earth fissures. Geotextiles may be used to minimize the potential for migration of soil material into subsurface voids. Where macrosized voids are encountered, large fragments of crushed rock may be placed into the voids with additional crushed rock fragments decreasing in particle size placed in successive layers. Geotextiles would be placed over gravel-size fragments with random fill on top of the geotextile to bring the site to a workable level grade. The locations of the repaired sinkholes, Earth fissures, or shallow mines would be determined by Global Positioning

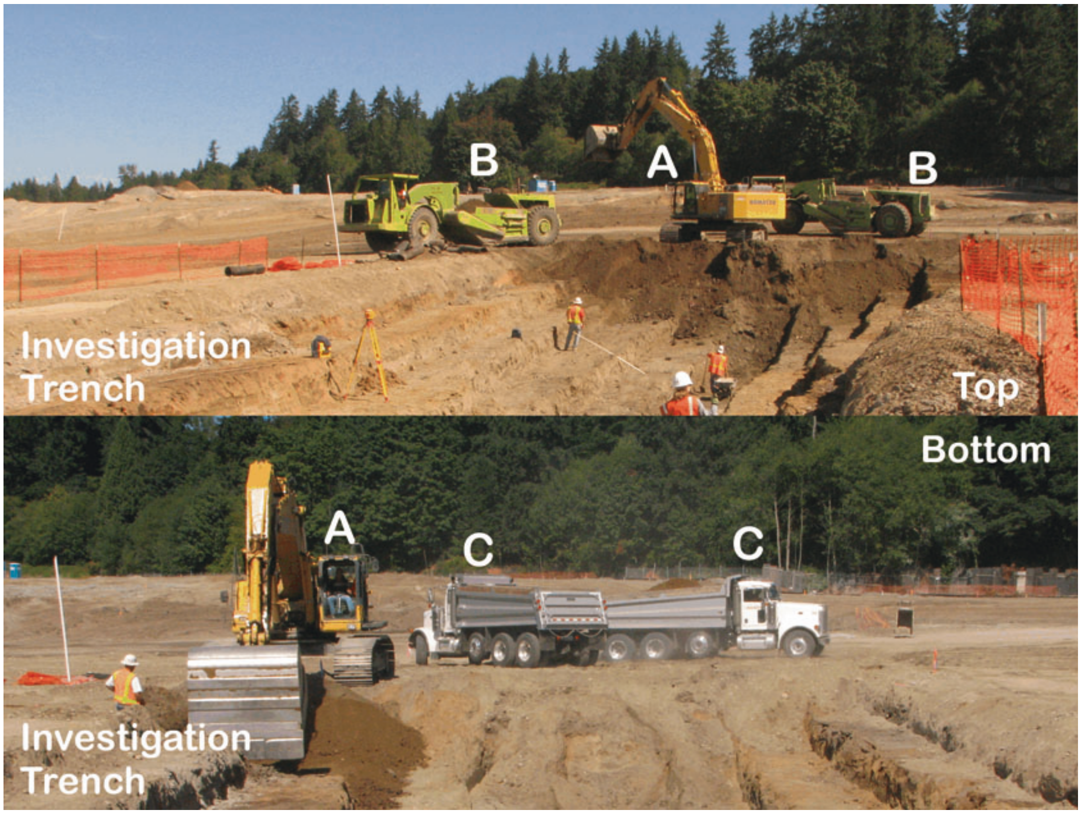
Ground Preparation, Fig. 1 Ground preparation to remove oversize rock and bring sloping ground into a level configuration. *A*. Application of water from a water truck that is out of view for controlling dust and initial moisture conditioning. *B*. Track-mounted excavator. *C*. Rubber-tire-mounted loader. *D*. Track-mounted dozer (Photo by Jeffrey Keaton, August 29, 2007)

Satellite (GPS) receivers and noted, so that repeated survey observations could be made periodically to allow early detection of future subsidence development and subsequent repair.

Sites with liquefiable or collapsible soil deposits would have ground preparation consisting of densifying the deposits by dynamic compaction, if feasible, or over-excavation, scarifying, moisture conditioning, proof-rolling, and soil compaction to a suitable site elevation. Sites with expansive soils may need little or no ground preparation prior to fill placement, depending on the degree of expansion potential in the on-site soils and the thickness of fill to be placed. Sites with highly expansive clay soils that need little site grading fill should overexcavate the expansive soils and bring the site back to a ground-preparation grade with nonexpansive clay soils; sandy soil should not be used to replace the overexcavated expansive clay soil because it would allow excess moisture to come in contact with the expansive soils with little thickness of site grading fill above it.

Sites with erosion gullies or stream channels would have ground preparation consisting of modification to the gullies or channels, which could require permits from agencies that regulate surface water resources. Sites with karst conditions that include rock pinnacles would need to have the rock pinnacles removed to a reasonable grade below the subgrade where the top of the ground preparation would be completed. Sites with occasional boulders or rock blocks that will need substantial fill placement could have the boulders or rock blocks buried at the base of the fill provided that the boulders or rock blocks are isolated with sufficient space for compaction equipment to move around them. The top of the ground preparation would be the elevation where the isolated boulders or rock blocks are surrounded and covered by compacted fill soils.

Sites with environmentally contaminated soil deposits would have careful excavation of the contaminated soils, placement in haul trucks or vehicles that could be covered with tarp or other containment, and be transported to a suitable hazardous waste disposal facility. The top of ground preparation would coincide with the moisture-conditioned and proof-rolled site surface, ready to receive soil for compacted fill.



Ground Preparation, Fig. 2 Late-stage ground preparation for a site in a tectonically active area that was required to demonstrate absence of faulting with a geologic investigation using a large trench exposure; the trench was excavated in a manner that allowed it to be backfilled with a fill placement and compaction process using heavy equipment. Top: Excavator (*A*) and scrapers (*B*). Bottom: Excavator (*A*) and heavy-duty 10-wheel dump trucks (*C*) with sleeper wheels for extra-heavy loads. Proof-rolling could be accomplished effectively with loaded dump trucks, but probably not with loaded scrapers because of the large contact area of the scraper tires (Photos by Jeffrey Keaton; top photo, August 20, 2006; bottom photo, August 19, 2006)

The success of a ground preparation program depends on the quality of the site investigation to identify the nature and extent of deleterious material or condition that is unsuitable to receive compacted fill that require mitigation.

Cross-References

▶ Artificial Ground
▶ Bearing Capacity
▶ Boulders
▶ Characterization of Soils
▶ Collapsible Soils
▶ Compaction
▶ Compression
▶ Consolidation
▶ Cut and Fill
▶ Dissolution
▶ Dynamic Compaction/Compression
▶ Environmental Assessment
▶ Erosion
▶ Evaporites
▶ Geotechnical Engineering
▶ Ground Motion Amplification
▶ Infiltration
▶ Loess
▶ Organic Soils and Peats
▶ Run-Off
▶ Sinkholes
▶ Site Investigation
▶ Stabilization
▶ Subsidence

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Ground Pressure

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Definition

Pressure exerted vertically or laterally on, and within, the ground.

Overburden pressure is the load caused by overlying materials on the ground causing an effective vertical stress in the ground. Effective lateral ground pressure is the horizontal effective pressure in the ground. The lateral ground pressure can be neutral, active, or passive. Virgin ground pressure is the ground pressure before any activity in a new project is undertaken also called "virgin pressure," "virgin stress field," or "virgin stress configuration." "Active ground pressure" is the minimum possible pressure exerted on an object by the ground, whereas "passive ground pressure" is the maximum possible pressure caused by an object on the surrounding ground.

Ground materials are diverse and may be gases, fluids, solids (i.e., minerals, grains, and aggregates of grains or minerals), and any mixture of these and also include man-made ground, such as fills and waste dump materials. Ground is commonly differentiated into soil and rock; soil being an aggregate of loose or weakly bonded particles, and rock consisting of particles cemented or locked together, giving rock a tensile strength. Soil and rock are, by some, differentiated based on a compressive strength difference mostly with soil being weaker than 1 MPa and rock being stronger. A differentiation is made between "intact" and "discontinuous" ground, that is, ground without and with, respectively, distinct planes of mechanical weakness (discontinuities) such as faults, joints, bedding planes, fractures, schistosity. A groundmass consists of (blocks of) intact ground with discontinuities, if present. The stresses in the ground should be considered in terms of total and effective stresses.

Terminology and Definitions

The terminology and definitions above are given as used in this chapter. However, in the literature there is little uniformity on terminology and definitions used for ground pressures in engineering geology and geotechnical engineering. Agreement, more-or-less, exists for overburden pressure which is in virtually all literature the load caused by overlying materials on the ground, but whether this is in terms of effective or total stress and whether this pressure is specifically vertical has mostly to be found out from the context. Lateral ground pressure expressions and terminology suffer even more from many different definitions or inaccuracies in descriptions. Generally lateral ground pressure, for which also the term Earth pressure is used, is the horizontal pressure in the ground. It may have been defined in terms of effective or total stresses. Also the orientation may not be horizontal. Neutral ground pressure may be the ground pressure before any activity in a new project is undertaken or it may be the pressure in ground that is assumed not to have moved at all since starting the project. Sometimes the geological history and man-made stress influences are included in neutral ground pressure.

Theoretical Ground Pressure – K_0-*Factor*

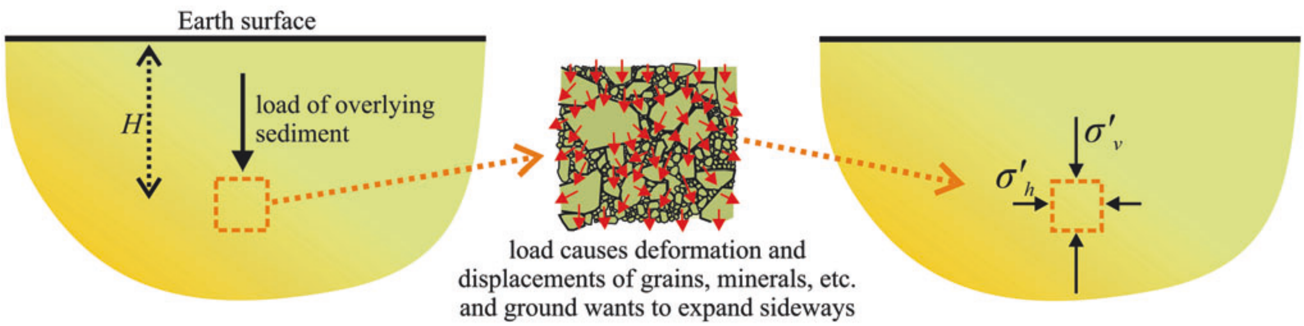
Assume an infinitely large area where gradual sedimentation has taken place (Fig. 1). The vertical effective stress (σ'_v) at depth (H) is then the sum of the weights per surface area of all the sediment materials accumulated above minus the pore pressure at depth H:

$$\begin{gathered}\sigma'_v(H) = \int\limits_{h=0}^{H} UW(h)dh - p_H \\ UW(h) = \text{unit weight of sediment } [\text{in kN/m}^3] \text{ at depth } h \\ p_H = \text{pore pressure at depth } H\end{gathered} \tag{1}$$

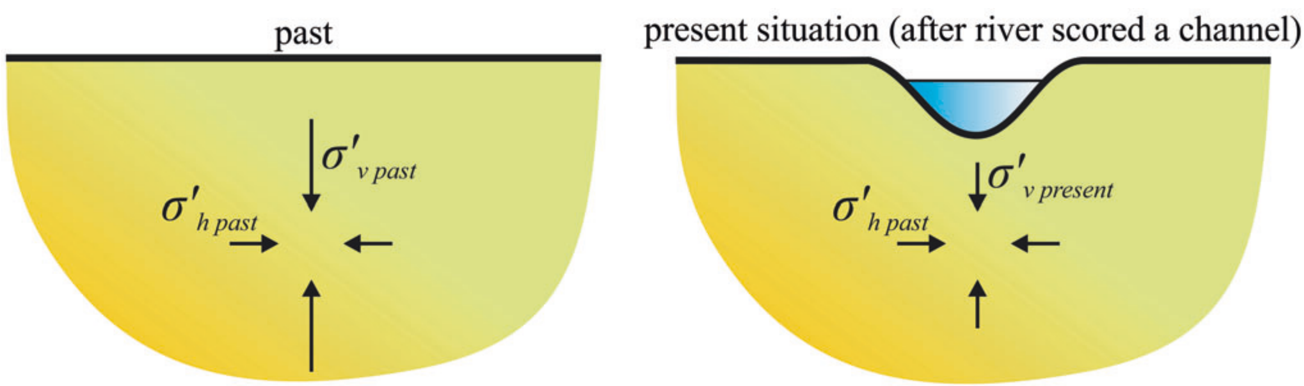
The load of the sediments causes compression of underlying materials; these deform vertically and want to expand (deform) sideways. However, the deformation sideways is impossible because there is already surrounding ground under the same load also trying to expand sideways; hence, the horizontal deformation is 0. This results in a horizontal stress induced in the ground. Mathematically, in terms of strain (ε) (i.e., deformation over the length of the body deforming), this is formulated as:

$$\begin{aligned}\varepsilon_{xx} &= 0 = -\frac{\sigma'_{xx}}{E} + \frac{v\sigma'_{yy}}{E} + \frac{v\sigma'_{zz}}{E} \\ &\frac{\sigma'_{xx}}{E}, \frac{v\sigma'_{yy}}{E}, \frac{v\sigma'_{zz}}{E} = \text{strain in the } x \text{ direction due to} \\ &\text{effective stress } \sigma'_{xx}, \sigma'_{yy}, \text{respectiviely, } \sigma'_{zz} \text{ in } x, y \\ &\text{and } z \text{ direction} \\ &E = \text{Young's modulus} \\ &v = \text{Poison's ratio} \\ &x \text{ and } y \text{ are horizontal and } z \text{ is vertical}\end{aligned} \tag{2}$$

Similarly, in terms of the strain in the y-direction:



Ground Pressure, Fig. 1 In situ ground pressure after sedimentation



Ground Pressure, Fig. 2 Frozen stress condition; the horizontal stresses reflect the stress situation from the past while the vertical stress is reduced due to scoring of a channel by a river

$$\varepsilon_{yy} = 0 = -\frac{\sigma'_{yy}}{E} + \frac{\nu\sigma'_{xx}}{E} + \frac{\nu\sigma'_{zz}}{E} \tag{3}$$

As no external stresses work on the ground and the ground is unlimited extending, isotropic, homogeneous, and intact, the shear stresses in the ground will be 0, and the stress in x and y direction will be equal: $\sigma'_{xx} = \sigma'_{yy}$. Rewriting Eqs. 2 and 3 gives:

$$K_0 = \frac{\sigma'_{xx}}{\sigma'_{zz}} = \frac{\sigma'_{yy}}{\sigma'_{zz}} = \frac{\upsilon}{1-\upsilon} \tag{4}$$

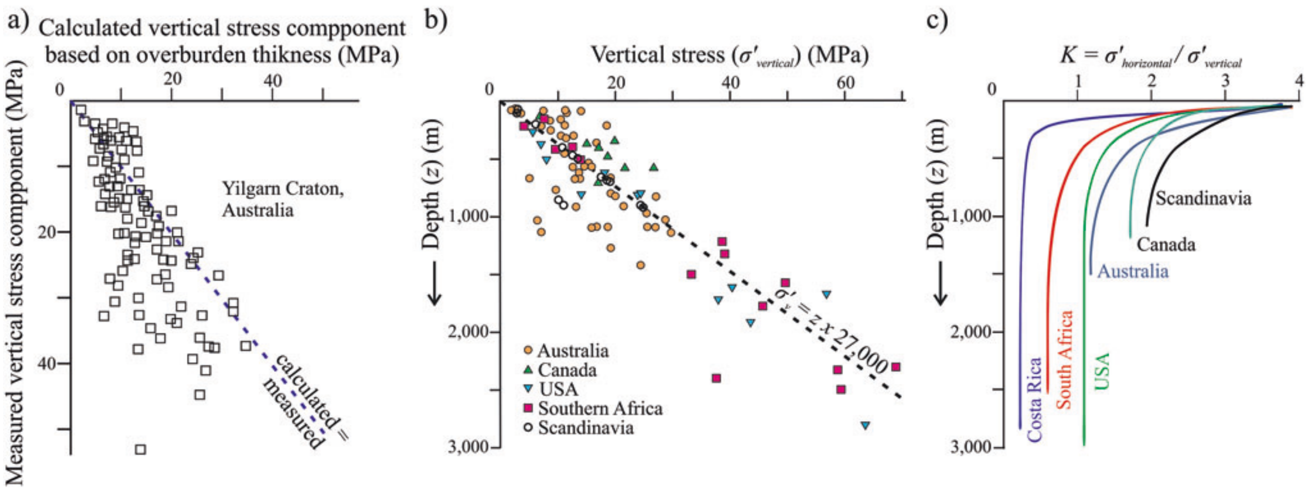
K_0 = factor for neutral ground pressure under conditions (see text)

For many types of ground, the Poisson's ratio (υ) is about 0.25 and thus K_0 is about 1/3 and the horizontal stresses are both about equal to 1/3 of the vertical stress. This ratio is seen often used in calculations, but in many cases likely incorrect (see below). If the ground is anisotropic, inhomogeneous, contains discontinuities, or is not ideal-elastic, the mathematical derivation of K_0 is more difficult or the equations do not have a unique solution (Verruijt 2012).

Real *In Situ* Virgin Stress Field – *K-Factor*

Groundmasses deform with time such that the shear stresses in the mass are minimized. The rate of time-dependent deformation is governed by factors such as material, discontinuities, temperature, and magnitude of confining stresses. Some masses may deform in geological time spans in the order of millions of years, whereas others deform within seconds. This time delay in adjusting to an imposed stress field is the reason for many *in situ* stress conditions to be representative for the geological past rather than for the present-day situation. Notably, the overburden stress in the past has been higher than at present because since then the surface has been eroded, or areas may have been glaciated and thus were loaded with ice masses (Fig. 2). The lateral stress configuration may still partly reflect high overburden stress from the past. This only applies to the horizontal stresses as the vertical stress reduces directly after erosion of surface material or withdrawal of the glaciers.

However, vertical stresses may also be diverging from what theoretically these should be based on overburden thickness (Fig. 3a). Geological structures, such as folds and faults, topography undulations, such as mountains, tectonic stresses,



Ground Pressure, Fig. 3 (**a**) Measured versus theoretical overburden pressure (vertical stress component); (**b**) depth versus vertical stress; (**c**) depth versus K-factor. Not all publications report whether the stresses are effective or total, however, the measurement techniques likely measured in terms of effective stresses (**a**: modified from Lee et al. (2006); **b** data in Brown and Hoek (1978); **c**: modified from Lopez (1999) with data from Bieniawski (1984) except Costa Rica, which is from Lopez (1999))

and man-made influences (see below) may cause vertical and horizontal stresses to be smaller or larger (Fig. 3b,c). In Fig. 3c the average values of σ'_{xx} and σ'_{yy} are used, but also considerable differences may exist between the two horizontal stresses, particularly, near tectonic active areas. Last but certainly not least, the orientation of the major, intermediate, and minor principal stresses do not need to be aligned vertical and horizontal, but may be rotated (Heidbach et al. 2016). The K_0-factor does not take these influences into account and therefore often a K-factor is used rather than K_0. The K-factor is just the ratio of effective horizontal over effective vertical existing stress, whatever the origin of the stresses:

$$K_{xz} = \frac{\sigma'_{xx}}{\sigma'_{zz}} \; or \; K_{yz} = \frac{\sigma'_{yy}}{\sigma'_{zz}} \qquad (5)$$

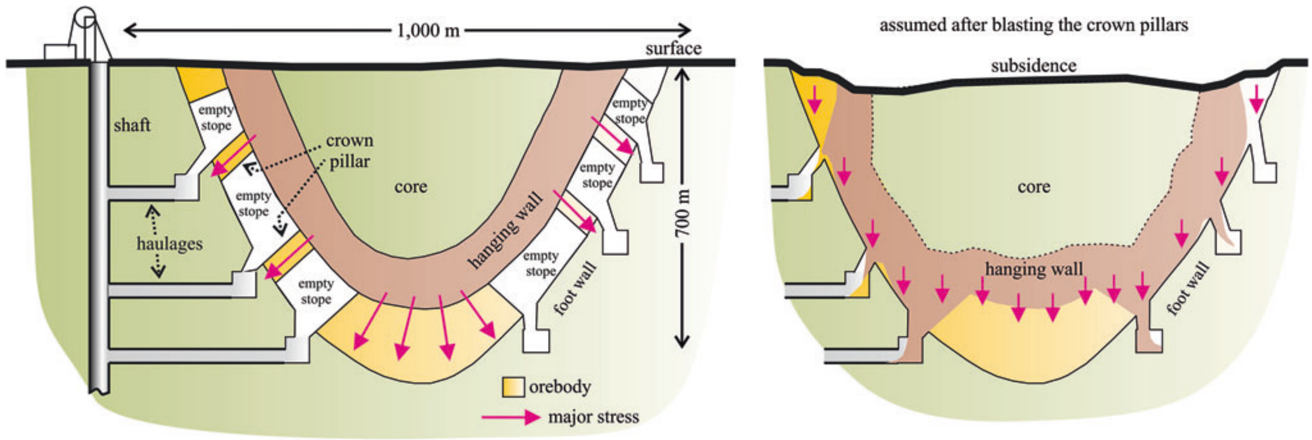
Measuring and Guessing or Estimating the Virgin Ground Pressures

The effective virgin ground pressures can be measured with many methods, such as pressure and dilatometers (Jang et al. 2003), hydraulic fracturing, overcoring, and jacking methods. However, measuring is cumbersome, generally expensive (borehole drilling), and measurements are sensitive to local influences such as the presence of discontinuities and multiple measurements are needed to get reliable values. If no other information is available, rules of the thumb for the *in situ* virgin stress configuration are given in Table 1. These should be used with extreme care as, on many occasions, the stress field may be completely different. For a new project, it is advisable to investigate whether any other surface or underground workings (slopes, mines, tunnels, etc.) have been made nearby. The staff and labor may have measurement data at hand or may give personal accounts of problems during construction due to the virgin stress field. It may also be worthwhile to go underground in other projects and see whether any signs are visible that indicate problems with the stress field, such as heavier support or more deformation and fracturing depending on tunnel direction that cannot be explained by differences in the groundmass.

Ground Pressure, Table 1 Virgin effective stress field (not very reliable rules of the thumb)

	Depth (m)	
Vertical effective stress	<50	Stress due to weight overburden
	50–2,000	Stress due to overburden weight, but large scatter has been measured with values between 0.5 and 2 times the overburden weight
	>2,000	Stress due to overburden weight
Horizontal effective stress	<50	$K = 0.4$–0.6 for cohesive material, e.g. clay; $K = 0$–0.4 for discontinuous rock masses; $K = 0.25$–0.45 for loose, non-cemented material
	50–500	$K = 0.4$–3.5
	500–1,500	$K = 0.5$–1.5
	1,500–2,500	$K = 0.7$–1
	>2,500	$K = 1$

Notes: the above are the values as used by the author if no other information is available; the values should be handled with great care ($K = \sigma'_h/\sigma'_v$; σ'_h is the average horizontal effective stress)



Ground Pressure, Fig. 4 Man-made induced stresses due to poor mine planning; left: crown pillars in place; right: after blasting the pillars (the number of stopes has been reduced for clarity of the figure)

G

Human-Induced Stresses

In many situations, the site for a proposed excavation is to be made in a stress field disturbed by other excavations. Consider Fig. 4, which shows a synclinal ore body that has been mined from surface downwards by open stopes (mining expression for a "mining room" from which the ore is extracted). The "crown pillars" made to allow mining of the stopes without collapse of the hanging wall were left in place after the stope was mined. These were expected to fall apart or to fail under the increased stress concentrations, and the hanging wall was supposed to collapse onto the footwall. Then the weight of the central core of the syncline would have spread over the whole footwall. However, the crown pillars did not collapse, but kept transferring the weight from the central core to the footwall. This resulted in a strongly anisotropic stress field at the locations of the crown pillars and created problems with the stability of the haulages (mine tunnels), and the stresses on the bottom part of the syncline prohibited further mining. A solution for the problem was found in blasting the crown pillars causing the collapse of the hanging wall and thus the more evenly spreading of the weight of the core over the whole footwall. This example is extreme, but human-induced modified stress fields can also be caused by an already existing tunnel or by surface structures in case of a shallow underground excavation. Notably foundations of high-rise buildings may influence the magnitude and orientation of the stress field underground.

Effective Active (K'_a) and Passive (K'_p) Ground Pressures

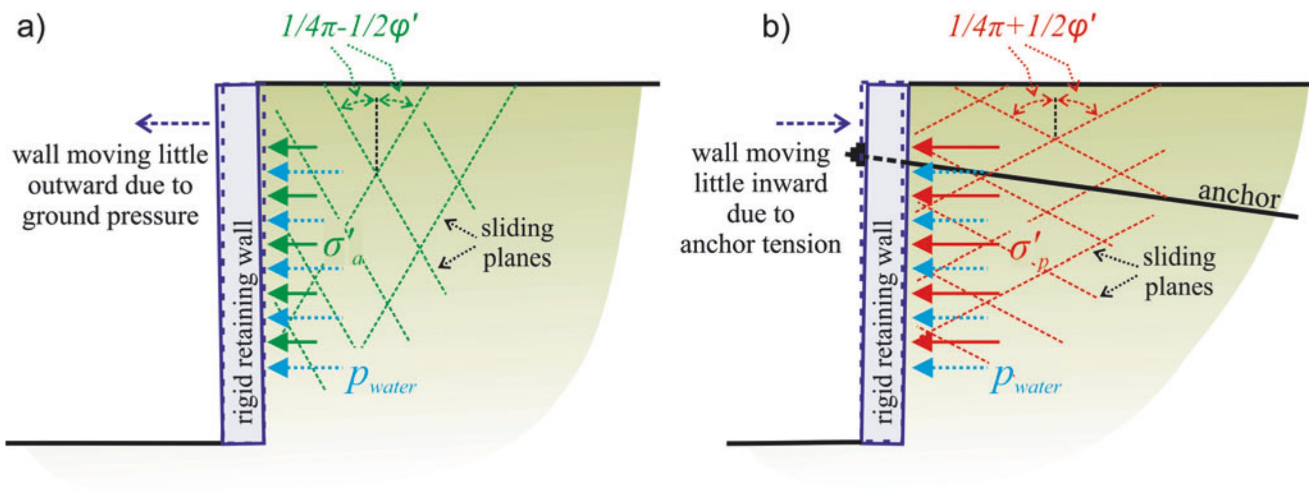
Often the virgin stress field is not important and the ground stresses exerted on a structure are only dependent on the local situation, for example, a retaining wall with back-fill. The stress working on the structure is then due to the back-fill only and stresses can be calculated. The effective active ground pressure is the minimum possible pressure exerted on an object by the ground, whereas the effective passive ground pressure is the maximum possible pressure caused by an object on the surrounding ground. The "active" and "passive" denotation should thus be considered from the action of the ground and not from the object. Consider the rigid retaining wall in Fig. 5a. The ground behind the wall can exert a pressure by sliding on sliding planes in the ground. The sliding planes make an angle of $1/4\,\pi - 1/2\,\varphi'$ with the vertical. The effective active lateral ground pressure (σ'_a) equals:

$$\begin{aligned}
&\sigma'_a = \sigma'_3 = K_a\sigma'_1 - 2c'\sqrt{K_a}\\
&K_a = \frac{1-\sin\varphi'}{1+\sin\varphi'} = \tan^2\left(\frac{1}{4}\pi - \frac{1}{2}\varphi'\right)\\
&K_a = \text{coefficient of active ground pressure}\\
&c', \varphi' = (\text{effective})\text{ parameters Mohr} - \text{Coulomb}\\
&\qquad\quad \text{failure envelope}\\
&\sigma'_1, \sigma'_3 = \text{major respectivily minor effective stress}
\end{aligned} \tag{6}$$

Analogue, consider Fig. 5b, in which a rigid retaining wall is anchored, and the anchor pulls the retaining wall against the ground. The ground behind the wall is pushed upward over the sliding planes in the ground that make an angle of $1/4\,\pi + 1/2\,\varphi'$ with the vertical. The effective passive ground pressure (σ'_p) equals:

$$\begin{aligned}
&\sigma'_p = \sigma'_1 = K_p\sigma'_3 + 2c'\sqrt{K_p}\\
&K_p = \frac{1+\sin\varphi'}{1-\sin\varphi'} = \tan^2\left(\frac{1}{4}\pi + \frac{1}{2}\varphi'\right)\\
&K_p = \text{coefficient of passive ground pressure}
\end{aligned} \tag{7}$$

The derivation of the equations is referred to text books on soil mechanics such as Verruijt (2012). Note that pulling a



Ground Pressure, Fig. 5 (**a**) Active and (**b**) passive effective ground pressure

retaining wall by anchors such that the ground actually moves, and the passive ground pressure is fully mobilized, requires very large forces on the anchors. This is in real situations virtually never achieved, and the passive ground pressure will be (considerably) less than the maximum possible.

Total or Effective Pressures, Water Pressure

In Eqs. 6 and 7, the stresses are in terms of effective stresses and to get the total stresses on the wall the water pressure has to be added in case of water behind impermeable walls. Some authors use accented factors of active (K'_a) and passive (K'_p) ground pressure if in terms of effective stresses; however, this is not widespread practice.

Effective Neutral Ground Pressure

Some engineers and literature use logical reasoning to obtain an effective neutral ground pressure, for instance, a ground pressure for ground that has not "moved." For example, a rigid retaining wall is constructed with a gradually installed back-fill by sand. In this case, it is unlikely that the retaining wall will exert a pressure on the sand and vice versa; it is unlikely that the ground will be allowed to move the retaining wall for any distance. Following this reasoning, the pressure on the wall is somewhere between K_a and K_p. Jaky (1948) formulated Eq. 8 which is reported to be often quite accurate for normally consolidated soils.

$$K_0 = 1 - \sin\varphi'$$
K_0 = coefficient of effective neutral ground pressure
φ' = effective angle of internal friction following Mohr –Coulomb failure envelope
(8)

Summary

Many projects have underestimated the influence of the virgin stress field on the structures in or on the subsurface and consequently suffered large financial losses. Therefore, a good estimation or preferably measurement of the virgin stress field should always be made by somebody knowledgeable about the local and regional geology. The often-applied assumption that the vertical stress equals the overburden weight per area and the horizontal stresses are about 1/3 of the vertical stress may be the start of a disaster. Only when for certain the virgin stress field will not influence the project, a relative simple calculation of stresses based on active and passive ground pressures or using a neutral pressure calculation will be appropriate.

Cross-References

▶ Ground Anchors
▶ Mechanical Properties
▶ Mohr Circle
▶ Mohr-Coulomb Failure Envelope
▶ Poisson's Ratio
▶ Rock Field Tests
▶ Shear Stress
▶ Soil Mechanics

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Ground Shaking

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Definition

Ground shaking at a site depends on the earthquake source and mostly on local geological conditions (site amplification effect), which increase or decrease the level of shaking compared with standard energy attenuation, in laterally homogeneous media. To deal properly with local geological conditions, the use of standard attenuation relations may be misleading (Molchan et al. 2011).

Using specific knowledge about geological structures and geotechnical engineering properties described in the published material available for any area of interest, earthquake ground motion scenarios can be computed varying the earthquake source position, mechanism, and magnitude based on advanced physical modeling and minimize the above-mentioned drawback (NDSHA method or Neo-Deterministic Seismic Hazard Assessment), see Panza et al. (2011). The three-component synthetic seismograms, computed, with a broadband content and in laterally inelastic models in the domains of displacement, velocity, and accelerations, can be processed to estimate the site effects and to extract parameters significant from an engineering point of view (Panza et al. 2012).

Shear wave (V_S) and surface waves (Rayleigh and Love) are the most sensitive waves to local soil conditions. The investigation of local soil properties can thus be made by using *FTAN analysis* (http://www.mitp.ru/en/soft/Ftan_Introduction.html) to record both local small earthquakes (Costanzo et al. 2013), and active and passive seismic experiments (Nunziata et al. 2012).

In fact, the most powerful methods to make measurements of V_S are based on the dispersion of phase and group velocities of surface waves. Phase velocity measurements suffer intrinsically for the undetermined number of cycles of the phase spectrum and, when signals are contaminated by higher modes, for the difficulty of isolating the fundamental mode. Seismic spreadings need several and very close receivers because of spatial aliasing. FTAN, based on group velocity dispersion, is an implemented multifilter technique, able to separate the different modes. It needs just one receiver and works well even in noisy areas, like urban areas. Then the nonlinear inversion of dispersion curves gives detailed V_S profiles with depth of shallow structures (Nunziata et al. 2012), overwhelming standard V_{s30} estimates.

Cross-References

- ► Earthquake
- ► Earthquake Magnitude
- ► Geological Structures
- ► Geotechnical Engineering
- ► Ground Motion Amplification
- ► Hazard
- ► Soil Properties

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Groundwater

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Definition

Subsurface waters can be divided into two categories (Fig. 1): (i) water in the unsaturated or vadose zone and (ii) water below the water table, occurring in the saturated or phreatic zone. The term "*groundwater*" traditionally refers to the latter.

Water in the vadose zone includes "hygroscopic water" and "capillarity water." Hygroscopic water (n. 1 in Fig. 1) forms a thin film of molecules that adhere to soil particle surfaces. These molecules are "glued" to soil particles by electrostatic forces, so there is no movement. Generally, hygroscopic water can be removed from soil only if heated ($T > 105$ °C). Capillarity water (n. 2, Fig. 1) can move slightly into very narrow pores (capillary); movement of water is omnidirectional (upward, downward, lateral), depending on the narrowness of soil pores. Generally the wideness of the capillary fringe increases as the average of the particle size decreases. Capillary water can be removed by evaporation processes in natural conditions (also by vegetation), but it is not subjected to gravity gradient (it cannot be removed by pumping).

Groundwater completely saturates soil pores (n. 3 in Fig. 1). Water migration is driven by gravity and more precisely by hydraulic gradient. Groundwater can be removed by pumping. (Michael 1985).

Groundwater Origin

Most groundwater abstracted for domestic, agricultural, or industrial use is meteoric water, that is derived from the infiltration of rainfall within the normal hydrological cycle (see ► Hydrogeology) at the recharge areas. Very deep groundwater in sedimentary basins may have originated as sea water trapped in marine sediments at the time of their deposition; this groundwater is referred to as connate waters. Finally, a relatively tiny amount of groundwater can originate by igneous processes within the Earth (juvenile waters). Sometimes mixing among two (or three) origins could occur.

Groundwater Circulation

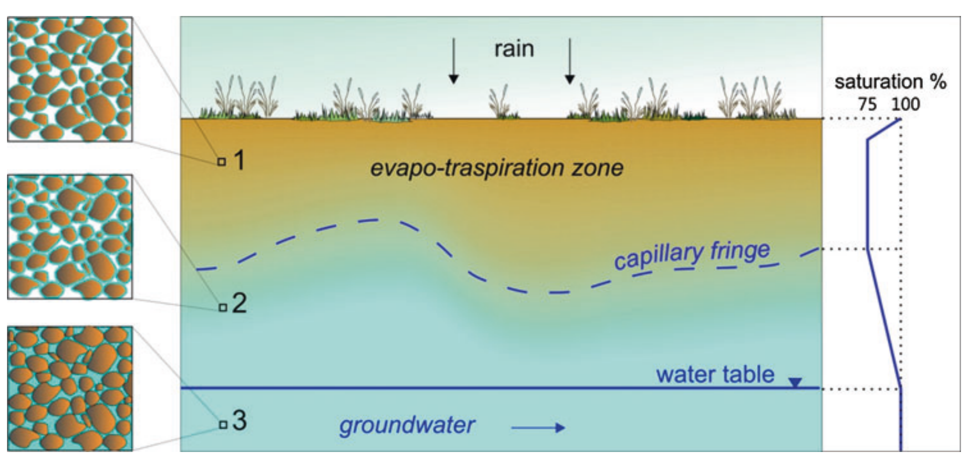
Groundwater that flows in the subsoil inside the pores and fractures of the reservoir rock formations is called an *aquifer*. From the recharge zone, groundwater moves by gravity towards the discharge zones, where it can feed rivers, springs, lakes, and wetlands or directly to the sea.

The groundwater circulation path represents a portion of the global water cycle (Fig. 2). Groundwater is a renewable resource since it is replenished by annual rainfall, but it is exhaustible. The assessment of actual groundwater availability for anthropogenic purposes and environmental needs (terrestrial dependent ecosystems) is the basis for sustainable water resource management.

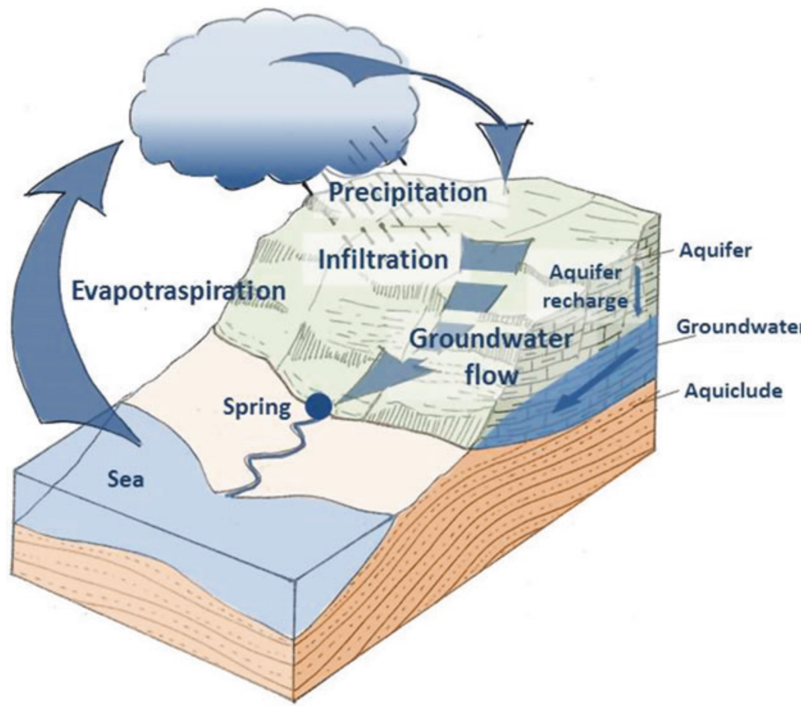
Groundwater represents a very small percentage (less than 1%) of the waters that make up the hydrosphere (Table 1), but it makes up about 30% of the Earth's freshwaters, which includes ice, snow, rivers, and lakes.

Groundwater and Surface Water

Groundwater and surface water are in hydraulic, chemical-physical, and biological continuity, representing a single



Groundwater, Fig. 1 Subsurface water and groundwater. All water beneath the ground



Groundwater, Fig. 2 Global cycle of water. Groundwater originates from rainfall then circulates into the aquifer from the recharge areas to the discharge zones (springs, lakes, rivers, or sea)

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Groundwater, Table 1 Global distribution of freshwater (Maidment 1993)

Reservoir		Percent of all water	Percent of fresh water
Oceans		96.5	–
Ice and snow		1.8	69.6
Groundwater	Fresh water	**0.76**	**30.1**
	Saline water	0.93	
Surface water	Lakes	0.013	0.26
	Marshes	0.008	0.03
	River	0.0002	0.006
Soil moisture		0.0012	0.05
Atmosphere		0.001	0.04
Biosphere		0.0001	0.003
Total		100 (1.4×10^9 Km^3)	100

resource (Rosenderry and LaBaugh 2008; Winter et al. 1998). Groundwater feeds and sustains rivers, lakes, wetlands, and dependent terrestrial ecosystems.

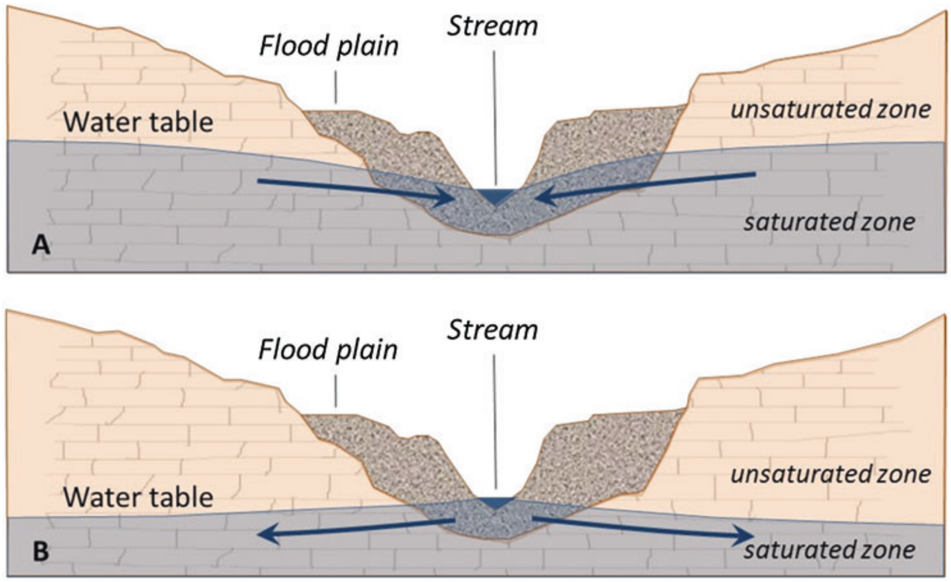
The interaction between groundwater and surface water occurs in different ways and varies over time and space. During dry periods, when rainfall and surface runoff are lacking, groundwater feeds streams (Fig. 3a), lakes, and wetlands, ensuring water supply because of the regulation role of the aquifer. During wet periods, streams can feed groundwater (Fig. 3b), modifying their chemical characteristics; it may happen, for example, that such surface waters contain pollutants and contaminate groundwater.

Understanding the interaction between groundwater and surface water is crucial in water management. This interaction has both qualitative implications (e.g., contamination and exchange of pollutants, enrichment of dissolved substances) and quantitative. Water diversions from rivers can deplete groundwater resources. In turn, water abstraction from an aquifer can exhaust discharge of streams and springs producing deterioration of connected ecosystems.

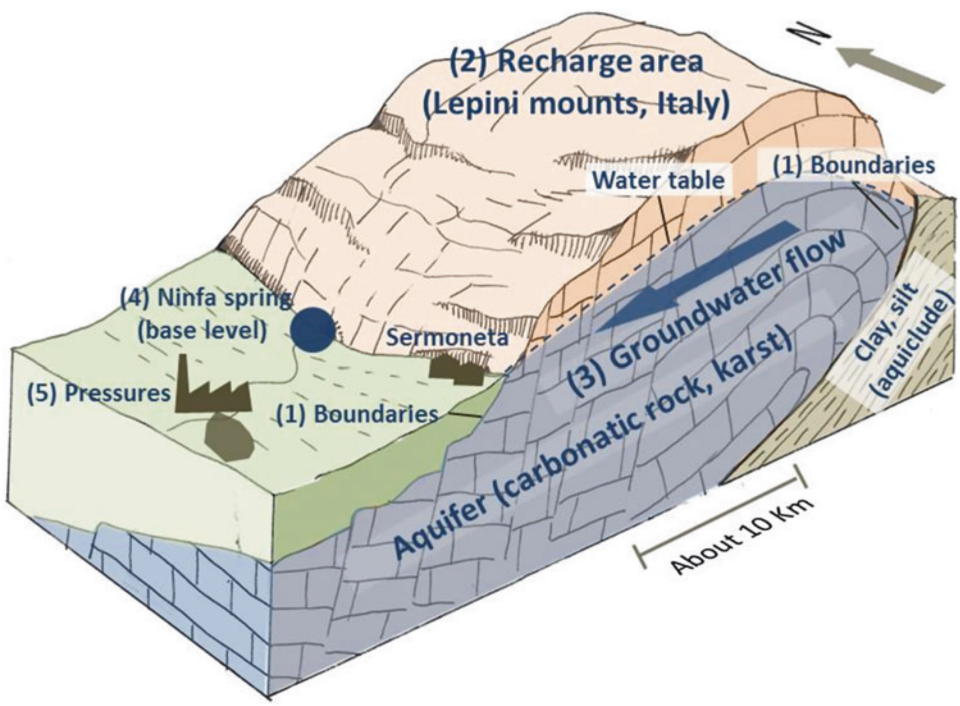
Hydrogeological System: The Conceptual Model

The hydrogeological system is the space unit where the groundwater cycle takes place in the unit of time. It is physically defined by the permeability limits of the aquifer, and is geologically defined by the features of the reservoir rock. Over time, it is defined by the reference period (e.g., long-term annual average) arbitrarily defined in order to assess water balance.

The hydrogeological system can be exemplified and described through a conceptual model. This model is constructed using the following basic elements of groundwater hydrology: (1) the **boundaries of permeability** that represent the physical boundaries of the aquifer and define its dimensions and volume; (2) the **recharge areas**, where rainfall feeds net infiltration and aquifer recharge, through pathways in the unsaturated zone of the aquifer; (3) the **pathways in the saturated zone** that connect the recharge areas to the



Groundwater, Fig. 3 Interaction of groundwater and stream. (**a**) *gaining stream*: groundwater feeds river during "base flow" conditions, when the discharge of rivers is mainly borne by groundwater inputs; (**b**) *losing river*: stream loses water to groundwater through the *hyporheic zone*



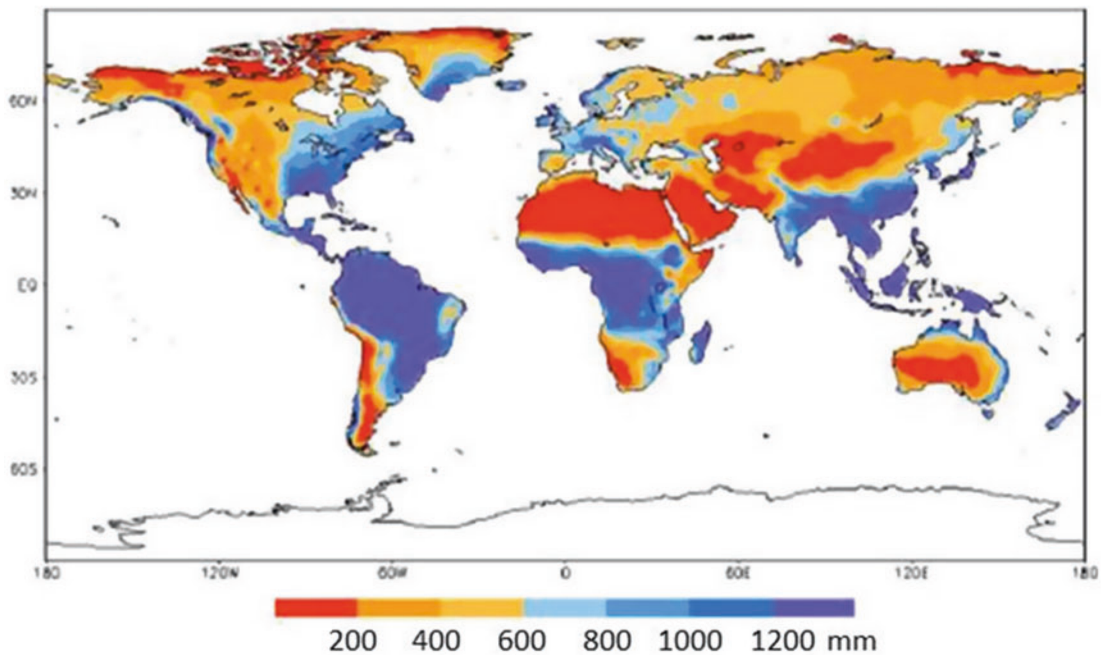
Groundwater, Fig. 4 Conceptual model of the hydrogeological system of Lepini Mounts, Central Italy

discharge zones; (4) the **discharge zones**, where localized springs, rivers, lakes, wetlands, or sea receive flowing groundwater; and (5) **pressures** threatening the hydrogeological basin. Figure 4 shows the conceptual model of an Italian carbonate geological structure.

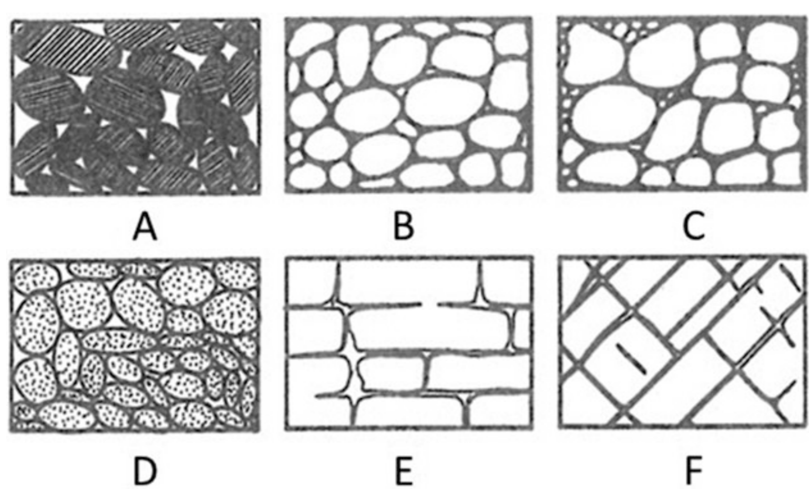
Quantitative Hydrogeology

Quantitative hydrogeology deals with the assessment of the availability of groundwater resources. Quantitative assessments must necessarily refer to the "*hydrogeological balance unit*" defined as the dynamic system "unit of space and time," part of the continental water cycle (Castany 1982); the hydrogeological balance unit coincides with the hydrogeological basin.

The availability of groundwater resources depends mainly on two factors: (i) the climate that controls the volume and the distribution in space/time of the inflows and (ii) the geology of the aquifer, which determines the hydrodynamics and the response of the hydrogeological system (discharge of springs) to external inputs (rainfall).



Groundwater, Fig. 5 Global distribution of annual precipitation (1901–2009 long-term annual average; mm/yr) (Huang et al. 2012)



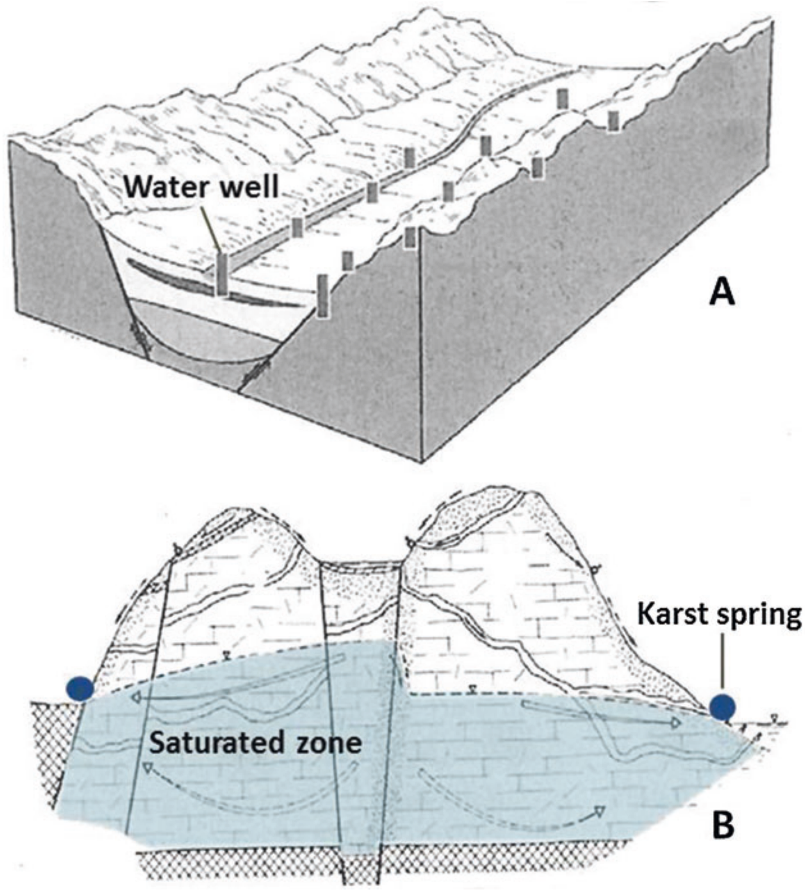
Groundwater, Fig. 6 Different kind of aquifer: (**a**, **b**, **c**, **d**) rocks permeable by porosity (e.g., sand, gravel), (**e**, **f**) fissured and karst rocks (e.g., carbonate karst reservoir) (Meinzer 1923)

The global distribution of precipitation (and therefore of climatic zones) determines the availability surface water and groundwater in the different areas of the world (Fig. 5).

Groundwater availability also depends on the geology of the aquifer. In fact, the hydrogeological features of rock formations such as permeability, storage capacity, and transmissivity, represent indicators of the potential use of water.

The geological features of the aquifer (such as morphology, structure, tectonics) define the groundwater circulation (e.g., Meinzer 1923, 1942). There are two main types of permeable rocks: permeable formations by porosity (sand, gravel, alluvial formations, fan, etc.; Fig. 6a, b, c, d) as well as fissured and karst rocks (Fig. 6e, f).

In these two kinds of aquifer, groundwater circulation occurs in very different ways. The porous aquifers can show a quasi-isotropic hydrodynamic behavior; groundwater circulates through the rock pores, saturating the aquifer homogeneously. In general, in these aquifers exploitation takes place by means of water withdrawal through vertical wells (Fig. 7a). Conversely, fissured and karst aquifers have a marked anisotropic behavior; the network of fissures and fractures imposes, at a local scale, preferential circulation pathways. For this reason, in such aquifers, it can happen that over a short distance a water well is very productive and another completely dry; in fact, water exploitation generally takes place capturing springs and rivers rather than abstraction by wells (Fig. 7b).



Groundwater, Fig. 7 Different kind of water exploitation: (**a**) in the porous aquifers water exploitation takes place generally by means of vertical wells; (**b**) in the fissured and karst rocks water exploitation takes place capturing the large spring located near the base level

Groundwater can be stored in confined (Fig. 8, Aquifer A) or unconfined aquifers (Fig. 8, Aquifer B). In confined aquifers, the water circulation occurs under pressure of the hydraulic head that depends on geometry and features of the geological reservoir

Renewable Groundwater Resources

Quantitative hydrogeology, by means of water balance analysis, answers the question: "*how much water can I use?*" Although water balance of hydrogeological systems is conceptually quite easy (assessment of inflows–outflows and analysis of water needs), the actual calculation and integration of required data is very complex. In addition, some basic indicators of the water balance equation have a certain degree of indeterminacy (e.g., actual evapotranspiration).

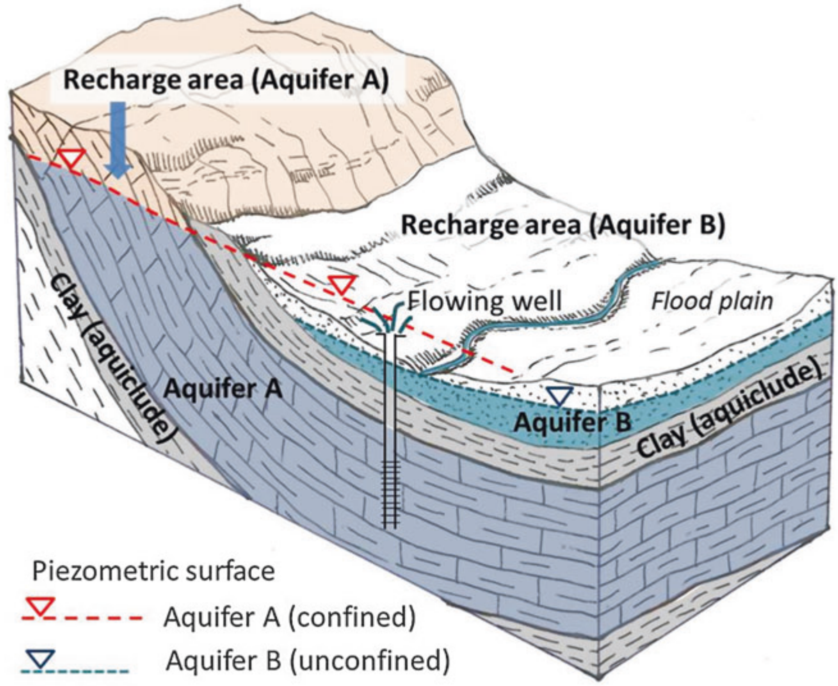
The knowledge of renewable groundwater resources strictly depends on the knowledge of two hydrological indicators that are difficult to determine: (1) the *aquifer recharge* and (2) the *portion of it* that can be exploited without compromising the equilibrium (physical, chemical, biological, ecological) of the natural environment.

Water overexploitation generates quantitative and chemical deterioration of water resources, limiting their use over time and damaging the environment, creating an unfavorable feedback on the ecosystems. The choice of groundwater flow rates that can be exploited without generating such a negative consequences must take into account the water requirements of both surface waters and dependent terrestrial ecosystems, as well as the groundwater ecosystem living up to hundreds of meters deep within the aquifer (Hancock et al. 2005).

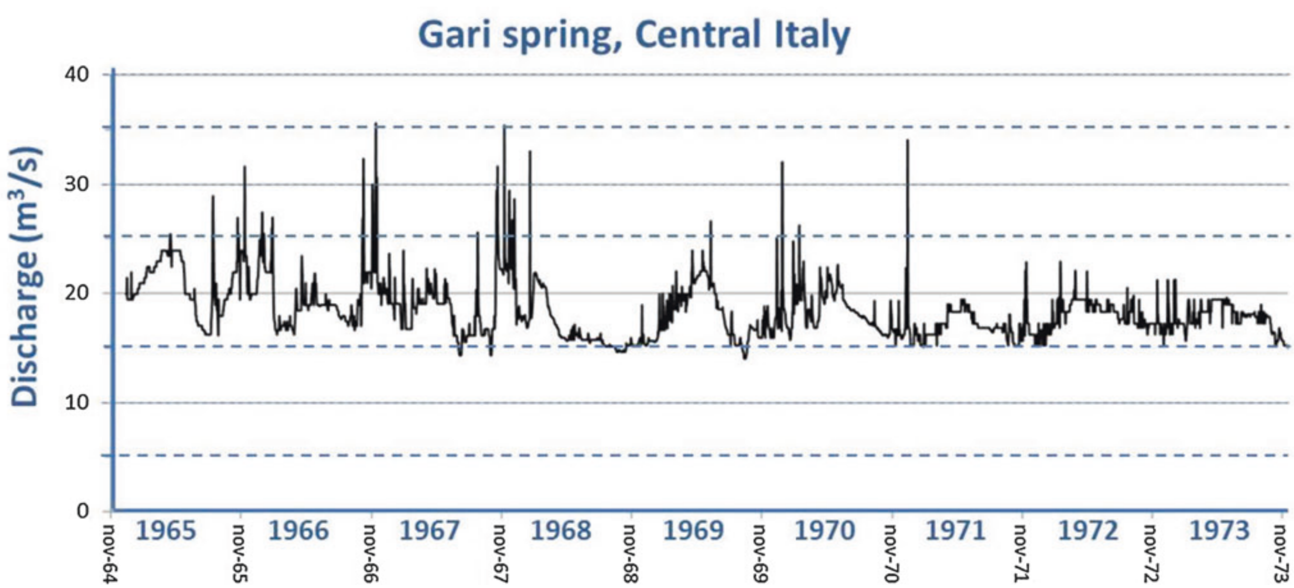
Therefore, it is not possible to use the maximum amount of groundwater resources. It would be a very short-sighted water management policy.

The hydrogeological reservoir works as a regulating system that transforms liquid or solid precipitation into perennial flows leaving the system (Castany 1982; Fitts 2002). The outputs from the system (through springs, rivers, etc.) depend on the recharge and are renewable; they represent the maximum amount of groundwater that the system can provide for the environment and human uses. In terms of water balance; they correspond – per balance unit and in the medium-long term period – to the average recharge of the aquifer. The outputs from the system can be monitored and measured in discharge (Fig. 9).

The aquifer recharge is evaluated by means of direct methods of measurement, such as the field measurement of the discharges that flow out of the hydrogeological system, at the base level. The recharge can also be inferred with



Groundwater, Fig. 8 Hydrogeological conceptual model of two overlapped aquifers. **Aquifer A**: confined aquifer of carbonate rocks; **Aquifer B**: alluvial, unconfined aquifer. The *red dashed line* represents the piezometric surface of the confined **Aquifer A**. The altitude of this piezometric surface, in the flood plain, is located above the ground, thus generating condition of *artesian* water table (flowing well) (From a sketch by Castany 1982, modified)



Groundwater, Fig. 9 Discharge hydrograph of a karst spring (Gari spring, Central Italy), representing the available volumes of groundwater resources, from 1965 to 1973

the indirect estimate of the net infiltration, starting from precipitation data, air temperature, and relative humidity (Thornthwaite 1948; Thornthwaite and Mather 1957; Penman 1948; Hargreaves and Samani 1982, 1985) by applying the *hydrological balance equation*:

Precipitation = Surface runoff + Net infiltration + Actual evapotranspiration

In addition to such water naturally emerging from springs, rivers, and lakes, additional water resources can be withdrawn using the proper methods of water abstractions from wells. In this case, the hydrogeologist must evaluate that the withdrawn volumes of water are consistent with the actual water availability.

There are several management choices related to water resources, such as the proper allocation for various needs (agricultural, civil, industrial, environmental) or the water services integrated management (water supply and sanitation, wastewater management, wastewater reuse). Mathematical simulation models provide help in these complex decision-making processes, involving multiple actors and specialists. However, the proper use of these decision support systems is a challenge, since the necessary information is diverse and their integration is complex (e.g., information about geology, aquifer characterization, quantitative and qualitative monitoring, water withdrawal, estimates of needs, existing pressures, and impacts on environment, interactions between components).

Permanent Water Reserves

In addition to water resources, which are renewable and exploitable, the aquifer hosts nonrenewable water volumes called "*Permanent water reserves*." They represent the groundwater stocks located below the hydrogeological base level stored in a given average period (Fig. 10). The volume of permanent water reserves depends on the geometry of the aquifer and its storage capacity. These reserves constitute, by definition, water volumes not available for uses (as nonrenewable). They represent "stocks" whose use constitutes a risk and requires knowledge of the conditions and times for restoring these "borrowed" stocks.

Groundwater Chemistry

Groundwater chemistry is related to regional geology and to the mineral phases occurring in the aquifer.

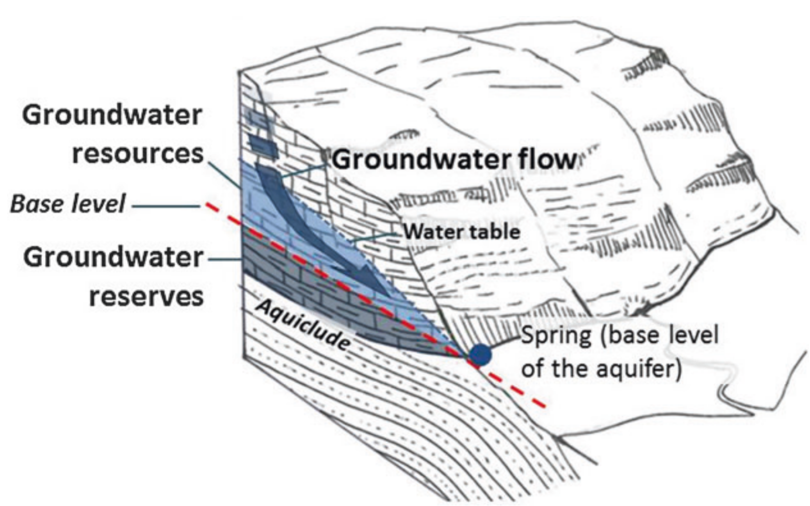
During the flowpath from recharge to discharge areas, the chemistry of groundwater changes during its passage through rocks, the changes depending on such factors as the minerals with which it comes into contact, temperature, pressure, redox conditions, pH, gas-water interaction, and the time available for the interaction.

An ideal complete sequence of the chemical evolution of groundwater starts with water in which the main anions are bicarbonate. As the water moves deeper, sulfate ions increase in importance and becomes dominant, finally, if the system is deep enough, chloride becomes dominant. Accordingly, with further complications due to ion-exchange, as the groundwater becomes older and deeper the dominant cations change from calcium and magnesium to sodium (Fig. 11). The TDS (total dissolved salts) also increases with time and depth.

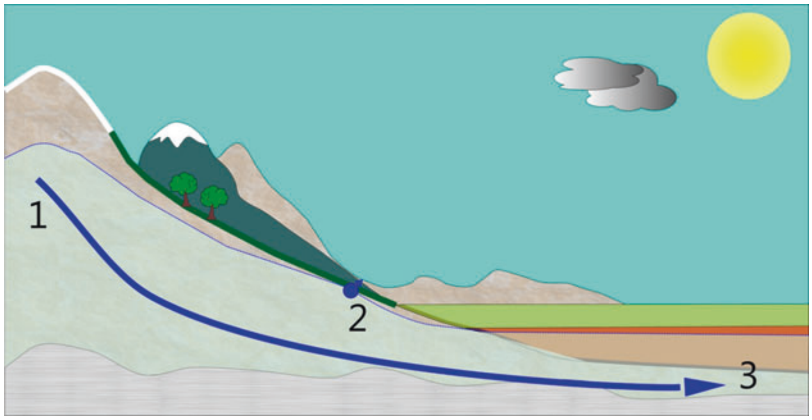
Geochemical properties of groundwater may even control the behavior of contaminants, so in environmental assessment, groundwater investigations should identify the hydrochemical facies (i.e., groundwater falling into well-defined compositional categories based on ratios between main cations and anions). This categorization is often based on graphical methods (e.g., Stiff diagrams, Piper or Chebotarev diagrams, etc. Fig. 12).

Groundwater Remediation

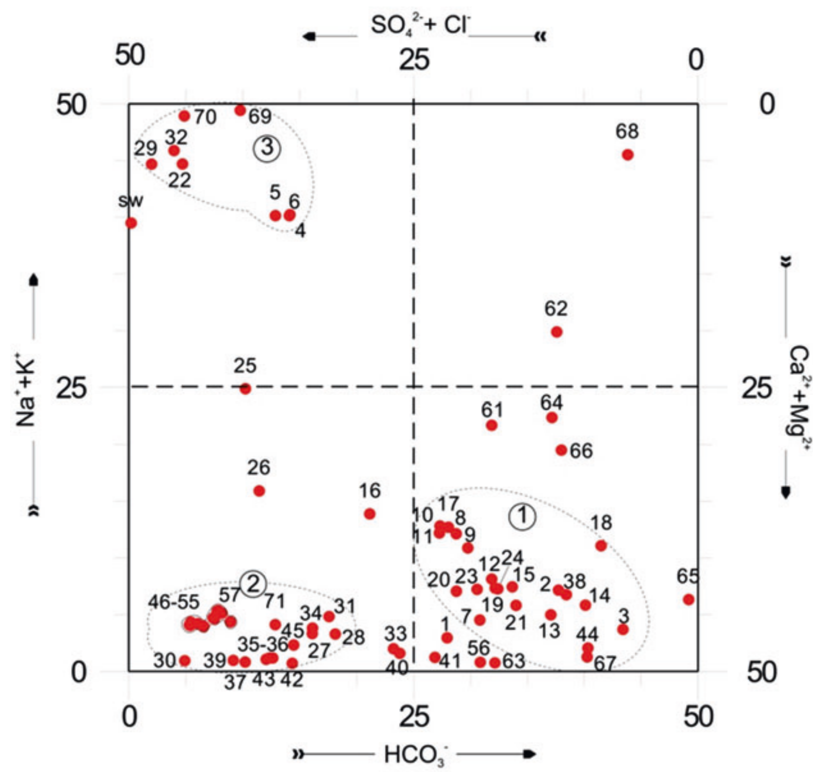
Anthropic pressures (e.g., agriculture, industry, civil sewage) can result in groundwater pollution. Environmental legislation in many countries made groundwater investigation and cleanup vital issues since the 1980s. Groundwater remediation includes technologies designed for groundwater pollution control and/or to make groundwater drinkable as coagulation, sedimentation, filtration, lime softening, ion exchange, oxidation-disinfection, chlorination, aeration, reverse osmosis, and ultrafiltration. The first step in selecting the appropriate groundwater remediation process is to evaluate and characterize the contaminant source and the hydrogeology of the area.



Groundwater, Fig. 10 Groundwater resources and reserves. Groundwater reserves are not renewables and then they should not be available for water exploitation



Groundwater, Fig. 11 This "typical" sequence is regarded as an ideal and very general process. The actual groundwater evolution depends on the site-specific scenario that includes: climate, aquifer mineralogy, organic matter in sediment, permeability and hydraulic gradient (that rules the time of rock-water interaction), heat gradient, interaction with deep seated fluid (e.g., gases moving thought fault). (**1**) Zone of recharge/active circulation: prevailing dissolution, soluble minerals are flushed out; rich in bicarbonate, nitrate, sulfate ions; TDS are low. (**2**) Bicarbonate and sulfate increase, possibly the latter prevailing. Ion exchange softens the water replacing calcium and magnesium are replaced with sodium. (**3**) Low hydraulic gradient zone. Sodium and chloride ions are dominant. TDS are high



Groundwater, Fig. 12 Chebotarev diagram representing the main chemistry of groundwater in Siena graben (central Italy). Group 1 represent the Ca(Mg) bicarbonate water from recharge areas and geothermal reservoir (the latter having very high TDS); group 2 are Ca-sulfate water occurring in deeper formation of Tuscan nappe (anhydrite at the bottom). In group 3, a component from a Pliocene marine clastic sequence is emphasized

Conclusions

Rapid human development, which took place since the 1950s, has led to the over allocation of water resources in almost all developed countries. Today this induces conflicts between different water needs (agricultural, industrial, hydropower, environmental needs, drinking water) leading to further overexploitation of water resources and reserves.

Overexploitation of resources has caused a chain reaction, leading to general degradation of the physical and biological environment and ecosystems loss.

Furthermore, the overexploitation and pollution of groundwater affects the entire hydrographic system, worsening the hydrogeological disturbance, the loss of soil, as well as the vegetation cover.

A growing awareness of the importance of protecting groundwater and surface water resources grew up since the 1990s. As a consequence, in recent years, environmental recovery and water reform projects have been implemented, also under the influence of national regulations and state agreements, such as the Water Framework Directive in Europe (European Commission 2000).

Nevertheless, the hazard of water degradation associated with the climate change scenarios imposes further investments and greater effort and resources. In this perspective, the spread of public awareness, the development of research and of monitoring networks, the improvement of decision-support systems, and communication policies must be promoted.

Cross-References

► Aquifer
► Aquitard
► Artesian
► Capillarity
► Catchment
► Climate Change
► Contamination
► Desiccation
► Dewatering
► Dissolution
► Drilling Hazards
► Facies
► Floods
► Fluid Withdrawal
► Fluidization
► Fluvial Environments
► Geochemistry
► Groundwater Rebound
► Hydraulic Action
► Hydraulic Fracturing
► Hydrocompaction
► Hydrogeology
► Hydrology
► Hydrothermal Alteration
► Infiltration
► International Association of Hydrogeologists (IAH)
► Percolation
► Piezometer
► Pollution
► Saturation
► Voids
► Water
► Wells

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Groundwater Rebound

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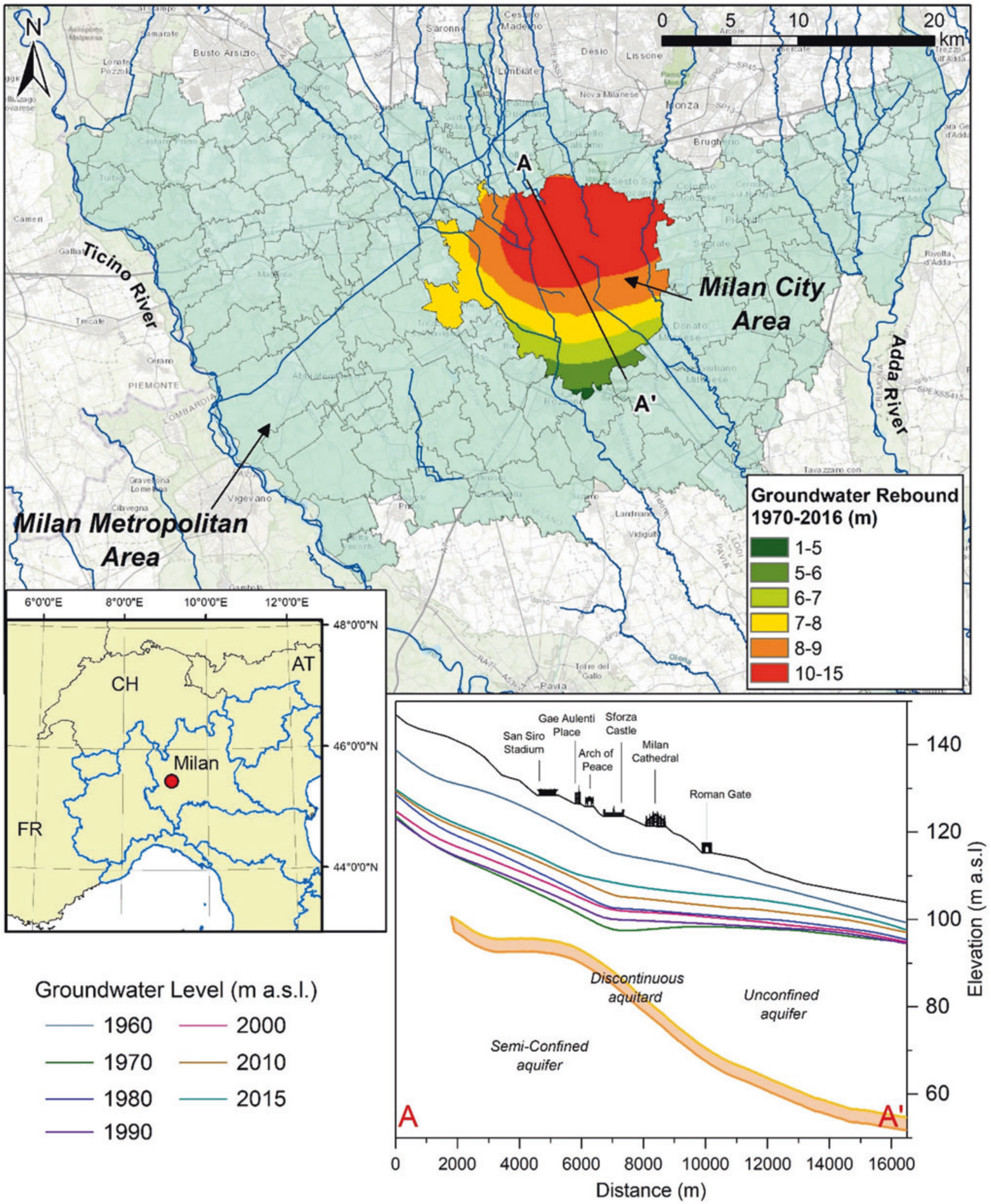
Definition

The gradual recovery of the water table after a prolonged period of artificially controlled drawdown such as high rates of abstraction.

The water table is the limiting surface between the unsaturated zone above (not fully saturated with water) and the saturated zone below it (where interconnected pores are filled with water). Groundwater level changes occur because of both natural phenomena and human activities. These changes



Groundwater Rebound, Fig. 1 Groundwater rebound in the Milan city area (Lombardy Region, Italy) represented for the 1970–2016 time interval. The cross section A-A′ shows the groundwater level oscillation since 1960, with a rapid decrease and the successive rebound involve either the injection (i.e., recharge) or the extraction (i.e., withdrawals or discharge) of water from an aquifer both through natural processes and human involvement.

Groundwater rebound generally occurs over decades after a prolonged period of drawdown (10–30 years, Heathcote and Crompton 1997; Morrison and Taylor 1994; Wilkinson 1985; Wilkinson and Brassington 1991) and within shallow aquifers. It is mainly caused by human activities including increased recharge from leaking water supply networks or septic tanks, overirrigation of parks and gardens within urban areas, intensely irrigated crop areas, import of additional water from surrounding rural areas or through desalination plants in coastal areas, changes in tap water supply sources, and changes in groundwater abstraction patterns due to decommissioning of industrial plants and mining sites (open pits and deep mines) or to the implementation of unsuitable pumping regulation plans.

Among these issues, the decrease of abstraction rates during the postindustrial stage development (i.e., 1970s for most industrial areas) is the principal cause of groundwater rebound for many cities (e.g., Barcelona, Buenos Aires, Liverpool, Birmingham, London, Tokyo, Paris, Milan – Fig. 1) (Brassington 1990; Brassington and Rushton 1987; Hayashi et al. 2009; Hernández et al. 1997; Knipe 1993; de Caro et al. 2017; Kreibich et al. 2009 Wang et al. 2017). In Middle Eastern cities (e.g., Kuwait, Doha, Cairo, Riyadh, Jizan) (Al-Sefry and Şen 2006; Rushton and Al-Othman 1994) the groundwater rise is mainly related to increased anthropogenic recharge. Therefore, the groundwater rebound phenomenon is strictly related with the socioeconomic development of a city. Urbanization results in important changes to the groundwater budget (i.e., balance between recharge and discharge). For example, changes in withdrawals and in land use replace and modify groundwater flow by introducing new discharge and recharge patterns. In addition, aquifers located within densely populated areas may undergo depletion and deterioration in water quality due to reactivation of quiescent pollution sources nested in the unsaturated zone before the occurrence of groundwater rebound.

At present, groundwater rebound is a major concern for many local and regional stakeholders charged with planning the water abstraction and defining strategies at local and national scale (Vázquez-Suñé et al. 2005). Many events have caused concern about groundwater rebound phenomenon and interactions between rising groundwater, underground structures, and the surrounding environments. These include damage to subsurface engineering structures, flooding of underground facilities, exfiltration of groundwater to sewers, chemical attack on concrete foundations, positive (i.e., upward) ground surface displacement, the mobilization of contaminants, and reappearance of surface springs.

Groundwater rise in shallow aquifers also increases the risk of groundwater flooding. In that case, groundwater can reach the ground surface or can facilitate flooding during extreme events because of the decreased water storage capacity within the unsaturated soil. Groundwater rebound in mining areas (Burke and Younger 2000) can involve the outflow of acidic waters with high concentrations in heavy metals and the possible contamination of surficial waters, soil and shallow aquifers. Because of all this, there is a need to understand the interaction between system components for the effective management and mitigation of groundwater rebound.

Cross-References

- ▶ Aquifer
- ▶ Aquitard
- ▶ Capillarity
- ▶ Catchment
- ▶ Climate Change
- ▶ Contamination
- ▶ Desiccation
- ▶ Dewatering
- ▶ Dissolution
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- ▶ Percolation
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- ▶ Pollution
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- ▶ Voids
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Grouting

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Definition

A ground treatment process performed to accomplish one of two things, either to reduce the flow of water or to improve the ground characteristics by drilling boreholes into the foundation and injecting materials under pressure into the subsurface foundation.

Each hole on a grouting project is an extension of the earlier exploration holes and the data gathered are utilized to increase the understanding of subsurface conditions. The injected materials used in grouting range from cementitious grout (particulate) materials to a variety of chemical grouts.

Grouting to Reduce the Flow of Water

Grouting foundations at dams and canals to reduce water flows is among the oldest grouting applications, dating to the early 1800s (Houlsby 1990) with grout curtains used since the 1890s (Bruce et al. 2010). The historical practice was to install vertically oriented grout holes, but it is now common to use two closely spaced curtain rows aligned along the dam axis that are oppositely inclined (Fig. 1). This procedure is designed to intercept the most fractures or joints that exist in the near subsurface.

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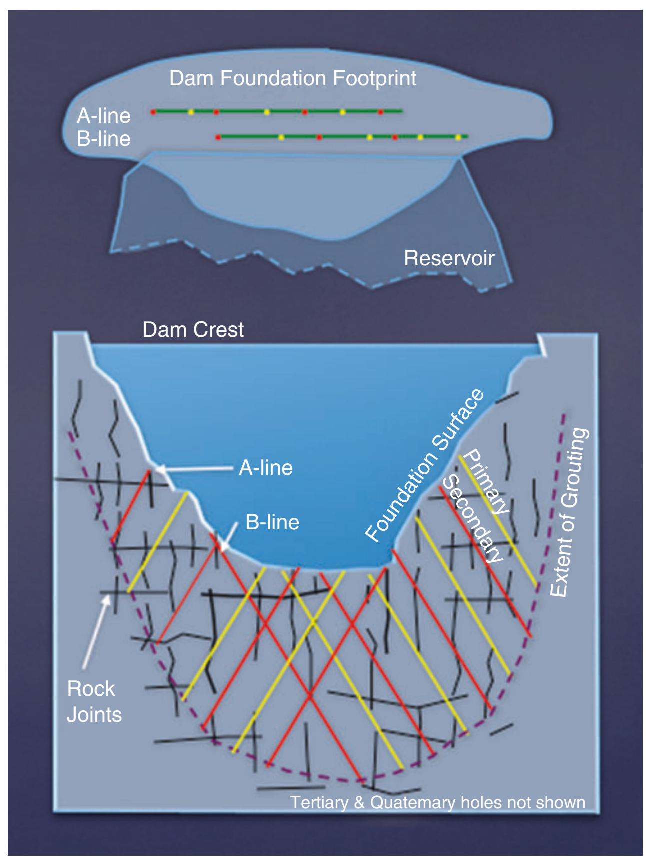

Grouting, Fig. 1 Diagrammatic view of grout hole layout at a dam

Grouting for Ground Improvement

Ground improvement methods include compaction or densification, soil replacement, dewatering, and admixture stabilization. Selection of the most suitable grouting method and type of grout for stabilization will depend on the type of soil, degree of improvement, and depth and extent of treatment required.

A common method used is jet grouting, which discharges cement grout sideways into the borehole where the high-pressure jets replace most types of soils. The soil is eroded and grout is mixed with the soil during the process. This type of grouting has been used to consolidate soft foundation soils, tunnel cutoff walls, and underpinning of structures. With this method, there is no material disposal, and less working room is required.

Mechanics of Grouting and Types of Grouting Materials

Prior to grouting, during the geotechnical site investigation, water takes are assessed in the subsurface foundation by performing water pressure tests in exploration boreholes. Grout monitoring and evaluation utilize fully integrated systems where all field instruments are monitored in real time through a computer interface. The calculations are performed automatically, grouting quantity information is tabulated and summarized electronically, and multiple, custom as-built grouting profiles are automatically generated and maintained real-time.

Grouting materials have evolved over time, with earlier grouts being cement-based (particulate) and chemical-based grouts being developed in the past 20 years (Johnsen et al. 2003; USACE 2017). The current state of the practice in North America is to design the grout curtain as an engineered element and to utilize additives and admixtures to create balanced stable grouts (Weaver and Bruce 2007). The desired grout is one that will penetrate the fractures a sufficient distance to provide overlap of grouted zones but will have sufficient cohesion to not travel too far while having a viscosity that permits the grout to be injected in a reasonable amount of time.

Chemical grouting has become a major activity in remediation and repair work under and around damaged or deteriorated structures. Chemical grouting is the process of injecting a chemically reactive solution that behaves as a fluid but reacts after a predetermined time to form a solid, semisolid, or gel.

Cross-References

► Cement
► Dams
► Ground Preparation

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H

Hazard

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Synonyms

Danger; Susceptibility; Threat

Definition

Hazard is a widely used term to describe threats to people and what they value including life, well-being, material goods, and the environment (Perry 1981) or more generally, a condition or process with the potential for causing an undesirable consequence (Bobrowsky and Couture 2014; IUGS Working Group on Landslides, Committee on Risk Assessment 1997). Ambiguity in the use of the term "hazard" is derived in part from the simple reality that it is used as a colloquialism and as a specialist term with different meanings or levels of precision for different disciplines (Nadim 2013).

In engineering geology, the term *hazard* is differentiated from the term *danger* or *peril* by including the temporal probability of occurrence of the event under consideration (Nadim 2013; ISSMGE 2004; Bobrowsky and Couture 2014).

Confusion also arises by the tendency, even in engineering geology, to use the word *hazard* to describe *susceptibility.*

Susceptibility (H_S) is the spatial likelihood (qualitative or quantitative) that a particular threat or danger will occur in an area. The term susceptibility is frequently used to characterize ground that has the potential to produce a hazard, and consequently may include spatial probabilities or details about magnitude, velocity, or intensity. Thus, susceptibility maps are often communicated to be hazard maps. The single distinguishing characteristic is that formal use of "hazard" explicitly includes a time-frame and a size factor.

Formally then, in engineering geology, Hazard (H_T) is the probability of occurrence within a specific interval of time, at or greater than a specific size (magnitude or intensity), that an event or events (geotechnical, geological, or geomorphological processes) will occur. Implied in most uses of the word is that such an occurrence would adversely affect people or the things people value; however, the term is independent of the actual presence of people. In other words, an earthquake of magnitude 7 with a focal depth of 15 km is the same hazard whether it occurs on a fault within the boundaries of a city or on a fault in an area remote from urban areas. The consequences of the occurrence of such an earthquake would differ depending on proximity to people, of course, but ideally the hazard would be the same.

Hazard represents the first half of the risk equation (the other half generally referred to as consequence). Notation for hazard and susceptibility can be combined as $H_{T,S}$ which refers to the probability of occurrence of a hazard at a specific location. This terminology is more than semantic. Consider the example of a channelized debris-flow hazard at the apex of an alluvial/colluvial fan. The fan may be inundated by a debris flow of a certain size, on average, once every 10 years. The Hazard (H_T) for the fan in this example is, therefore, 10^{-1}per year for a debris flow of a specific magnitude. Two houses on the fan, however, may not be subject to the same level of hazard. House A rests in a central location on the fan (because the views were better!) and house B closer to the fan margin. The average spatial likelihood that the debris flow will reach house A is higher (let's imagine that it is 40% of the time) than the likelihood that it will reach house B (let's imagine that it is 8% of the time) for a given magnitude of event. Put another way, H_T of a debris flow reaching the fan is 0.1. The annual $H_{T,S}$ for House A is $0.1 \times 0.4 = 0.04$, whereas the $H_{T,S}$ for House B is $0.1 \times 0.08 = 0.008$. While over-simplified, this example shows how hazard may be different depending both spatially and temporally.

P. T. Bobrowsky, B. Marker (eds.), *Encyclopedia of Engineering Geology*,
https://doi.org/10.1007/978-3-319-73568-9

Components of Hazard

Engineering geologists may use the informal definition of hazard in discussions with planners and those responsible for zoning ordinances. In such cases, its components depend on the nature of the process that contributes to what makes it hazardous such as slope steepness. Formally however, probability is universally required to define Hazard, as is some description of size. The main components are detailed below.

Probability

Probability is the likelihood of occurrence of an event or series of events over a given time or over a fixed set of observations (10 tosses of a coin for instance).

Analysis of hazard is more useful by considering the probability of occurrence of events of different magnitudes. Typically, for instance, large magnitude events occur less frequently than small magnitude events. The probability that hazards of the same type but different magnitudes will occur over a given area is described by a magnitude-frequency (M-F) distribution. Such distributions are used to quantify hazards such as landslides and earthquakes (Malamud et al. 2004; Guthrie and Evans 2004; 2005; Gutenberg and Richter 1956).

Magnitude

Magnitude is a measure of the size of the hazard. It may be measured as volume (m^3), area (m^2), or displacement (m) in the case of landslides or subsidence, discharge (m^3/s) or stage height (m) in the case of rivers or coastal floods, explosivity including cloud height and ejected bulk volume for volcanoes, energy release, accelerations, velocities, and displacements for earthquakes, wave amplitude for tsunamis and runout in the case of landslides. Velocity and intensity are sometimes used to describe magnitude.

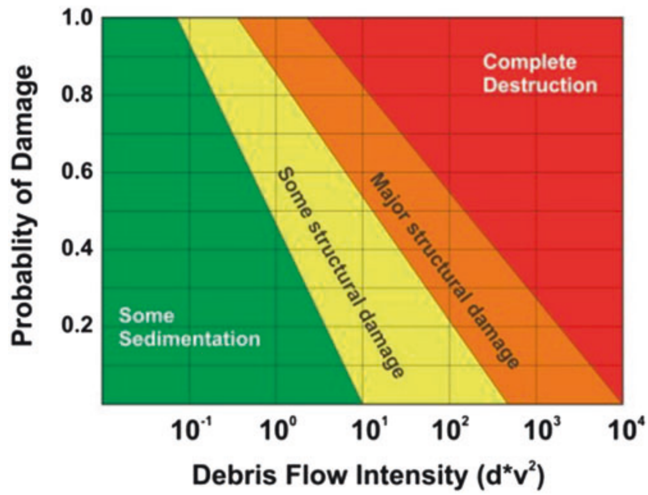
Hazard scenarios must normally consider the range of magnitudes and their associated probabilities.

Intensity

Intensity is a measure of the destructive force of a hazard. Intensity is often estimated using empirical scales that describe potential or observed damage from a hazard. The Mercalli scale for earthquakes and the Saffir-Simpson's scale for Hurricanes are both well known, but intensity can also be described for landslides (Fig. 1), tsunami's, volcanoes, and other hazards.

Extent

Most geological or geomorphological hazards are spatial in nature. Extent is the total area affected by a hazard or by a portion of the hazard with a given intensity classification or range. Flood inundation hazards are regularly described by the probability that an event will reach a certain extent, whereas extent for earthquake hazards is related to the expected zones of intensity or shaking.



Hazard, Fig. 1 Debris flow intensity and the probability of structural damage to a building (Jakob et al. 2012)

Duration

Duration refers to the period of time over which an element at risk is subject to a hazard. Some hazards are relatively instantaneous (landslides), whereas others such as floods and earthquakes may occur over a lengthy time period. The potential for damage and the need to engineer solutions to mitigate against the potential damage relates to the combinations of magnitude, intensity, and duration.

Flood protection works, for example, routinely use Intensity-Duration-Frequency curves to estimate the likelihood (often expressed as design return periods) of volume of water flowing for a given amount of time (time is related to run-off characteristics and the duration of storms supplying the water).

Velocity

Velocity is the speed at which a hazard travels. For most hazards such as landslides, tsunamis, and river floods, velocity is measured in m/s and is directly related to the release of energy and the potential to cause damage. In the case of rock fall, the damage potential is related to kinetic energy, which is equal to one half of the product of the rock mass and the velocity squared. Consequently, the damage potential changes along the path of the rock as its velocity increases during its movement to a maximum value and then decreases to zero in its final roll-out position.

Main Types of Geological Engineering Hazards

Geological hazards are derived from primary events (earthquake shaking and volcanic eruptions for example) or secondary events (tsunamis, debris flows, erosion) that are dependent on the primary event occurring, interacting with the primary and even secondary hazards. Some hazards, such as landslides, can fit into either category whereby they may occur as a result of time-dependent strain, or due to a specific triggering mechanism such as an earthquake or precipitation event. Some of the main types of Engineering Geological Hazards follow.

Seismic

Seismic hazard is the probability/unit time that an earthquake of a given magnitude will occur in a particular area. Seismic hazards are those primarily associated with the motions of crustal plates resulting in seismic waves transmitted through a medium (water, rock, ice, or sediment). Seismic hazards include shaking of infrastructure, sudden displacement of land along an axis (spatial and vertical) or combination of axes, and secondary effects that may result (fault ruptures, landslides, tsunamis, building collapse).

Volcanic

Volcanic hazard is the probability/unit time that an eruption of a given type and magnitude will occur over a particular area. Volcanic hazards can be grouped into explosive or effusive types and may include eruptions, lava flows, pyroclastic flows, lahars (volcanic debris flows), volcanic gas releases and flows, projectiles, and collapses.

Flooding

Flood hazard is the probability/unit time that a normally dry area of land will be inundated by water. Flood hazards can be coastal or riverine in nature.

Coastal floods may be related to high tides, winds, and storm surges, or long term changes in water elevation, or long-term changes in ground elevation related to glacial unloading or climate change. In rare instances, coastal flooding can also be related to the long-term subsidence of ground (such as Venice, Italy, along the Po River where it enters the Mediterranean Sea).

River floods are common, occurring by definition every 1–2 years on average for conditions in which the river channel reaches the capacity of its banks, known as "bankfull discharge" (Wolman and Miller 1960), and engineering design therefore typically considers only less frequent events such as the 1:100 year flood. In wet mountainous regions, landslides and floods are often correlated; however, landslides (debris flows, mud flows, debris floods) are typically more destructive by unit volume.

Tsunami

Tsunami hazard is the probability/unit time that a seismically generated wave of a specified height or run-up will occur in a particular area. Tsunamis are normally generated by earthquakes; however, some of the most famous tsunamis are caused by landslides, including the biggest on record in Lituya Bay Alaska where a landslide into the bay created a tsunami that ran up to a height of almost 525 m. Tsunamis can impact coastlines thousands of km from their source.

Landslide

Landslide hazard is the probability/unit time that a landslide will occur in a particular area at a specified intensity. Landslides are caused by specific triggers including rainfall, floods, earthquakes or other shaking, excavation, or loading of a slope, or they are caused by the time-dependent nature of strain known as progressive failure. Landslide hazards interact with other hazards herein, particularly tsunamis (as a causal force), earthquakes (as an outcome) flooding, and volcanoes. Landslides hazards vary enormously by magnitude, velocity, intensity, depth, and length of runout.

Subsidence

Subsidence hazard is the probability/unit time that the ground will collapse, either slowly or quickly, a specified amount over a given area. Subsidence has several causes, but is mainly a response to the removal of ground water and the subsequent collapse of overlying materials or structures.

Interaction of Hazards

Hazards can occur in isolation, but frequently interact with one another. Floods, for example, can create landslides by eroding support at the base of slopes, whereas landslides can generate floods by damming rivers, creating a lake, and then collapsing under the hydrostatic load after weakening the mass by seepage through the mass.

In another example, earthquakes commonly drive other hazards. The 2015 Himalayan earthquake, for instance, killed 9,000 people and injured thousands more. In addition, the shaking caused several deadly landslides including one that killed 21 on Mount Everest, making it the deadliest day historically on the mountain (Wikipedia 2017).

Hazards may occur simultaneously as a result of the same triggering event, interact with one another, and initiate a cascade of events and compounding hazards.

Summary

Hazard (HT) is the probability of occurrence within a specific time and within a given area that an event or events

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(geotechnical, geological or geomorphological processes) will adversely affect people or the things society values. Engineering geology seeks to understand and predict the type, nature, and magnitude of hazards in order to mitigate against them and reduce the threat accordingly.

Cross-References

- ▶ Earthquake
- ▶ Geohazards
- ▶ Hazard Assessment
- ▶ Hazard Mapping
- ▶ Landslide
- ▶ Probabilistic Hazard Assessment
- ▶ Risk Assessment
- ▶ Risk Mapping
- ▶ Subsidence
- ▶ Tsunamis

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Hazard Assessment

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Synonyms

Danger assessment; Threat assessment

Definition

Hazard assessment is the procedure to characterize and map the location, magnitude, intensity, geometry, and frequency or probability of occurrence, and other characteristics of a given threat, event, phenomenon, process, situation, or activity that may potentially be harmful to the affected population and damaging the society and the environment.

Definition of frequency or probability is a complex endeavor that includes the causes of threats (state of territory, predisposing factors, triggering mechanisms), long-term evolution, and present-day trends (monitoring and geo-indicators), resulting in the frequency/probability of occurrence of certain phenomenon over time (or the susceptibility that an area may be affected by specific perils).

Definitional Uncertainties

Risk assessments require hazard assessment, exposure identification, and vulnerability analysis data at the appropriate scale as well as models with the proper resolution to address the problem of interest. Hazard assessment is generally represented by an indicator, such as the intensity/magnitude of a natural phenomenon or human activities and the probability of occurrence. Exposure is the quantification of the receptors (physical, social, economic, systemic, and functional) that may be influenced by a hazard, for example, number of people and their demographics, number and type of properties, etc. Vulnerability analysis refers to the fragilities of an exposed element (physical, social, economic, systemic, and functional) that can lead to disruption and harm when facing the stress provoked by a given hazard. Hazard assessment is then essential in defining the entire chain, since it is the factor that both enters into risk assessments and that constraints the definition of parameters of physical vulnerability of the built environment, as the latter need to be established according to specific levels and features of the expected stress.

Some misunderstanding is often present in terminology. There are a variety of definitions of "hazard," even though most of them deal with the attempt to characterize, in discrete categories, the different return period of energy (or magnitude), and impact's spatial distribution of natural phenomena and human activities. Hazard is characterized by the intensity, that is, the severity at a given place and the magnitude or the global energy of the event. Having in mind a seismic hazard, the level of hazard (assessment) is governed by the magnitude of the hazard, its frequency or recurrence, and intensity at the impact point.

This definition, when applied in practice, encompasses a broad range of both natural phenomena and human activities which have the potential to cause damage and disruption, with territories being exposed to such hazards in an extremely heterogeneous fashion. Hazards can be of either slow (e.g., drought) or rapid (e.g., earthquake) onset and have the potential to have effects which can be felt across a range of scales. More in detail, any phenomenon has its own "size" in terms of the involved area (see, for instance, landslide vs. earthquake), physical processes, and availability of long term data. The result is a variety of indicators and approaches to hazard assessment, each one dependent on the input phenomenon. Every parameter, and its measurement, has inherent uncertainty sometimes related to the ambiguity in the definition; this is known as definitional uncertainty. In the case of hazard assessment (Nadim 2013), one of the reasons for the ambiguity in the definition of hazard is that the term is used both to describe the temporal probability of occurrence of the event and the situation in question. Whether hazard refers to the event or to its probability of occurrence within a given period of time depends on the context of its usage. As a scientific concept, this ambiguity in the meaning of hazard is not desirable. To avoid this problem, Technical Committee 32 of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), for instance, suggested using the term "danger" or "threat" to refer to the phenomenon that could lead to damage, and the term "hazard" to address the probability that a particular danger (threat) occurs within a given period of time. In this definition, the characterization of danger or threat does not include any forecasting, whereas the characterization of hazard includes the assessment of frequency or temporal probability of occurrence. The ISSMGE definition of hazard is very suitable for scientific discussions. It is, however, not entirely consistent with the common usage of the term. For example, to be consistent with the ISSMGE terminology, one should refer to earthquakes (earthquake) and floods (flood) as natural threats, and not natural hazards (Nadim 2013). The hazard assessment is then, mainly, the procedure to characterize and map the location, magnitude, and frequency of occurrence of a given natural threat.

Another definitional issue relates to the fact that in American literature often the terms "hazard assessment or hazard management" (see, for example, White et al. 2001; and the bibliography provided by Tiefenbacher 2014) actually refer to what in Europe and other regions of the world is referred to as "risk assessment or risk management." The actual meaning in that case is easy to grasp from the text, however it is important to note this difference.

Language of Hazard: Purposes and Scales

From a geological standpoint, the Earth is a "product" of geological processes that define the distribution of land and sea, of mountains and plains, rivers, and lakes, of meteorological processes that constitute our atmosphere. All of these processes are steadily changing the face of our Earth and are thus a phenomenon of which humankind is integral. But these changes do not in every case run smoothly, gently, and peacefully, rather sometimes they are very sudden, eruptive, and incidental (Ranke 2016).

But not only nature can be hazardous to humankind; human activity also often intersects with the natural system that may again interfere with the human environment. Human-induced or anthropogenic hazards originate from technological or industrial accidents, dangerous procedures, infrastructure failures, or other activities, which may cause the loss of life or injury, property damage, social and economic disruption, or environmental degradation (e.g., industrial pollution, nuclear activities and radioactivity, toxic wastes, dam failures, transport, industrial or technological), all resulting from modifications of natural processes in the Earth's system caused by human activities which accelerate and aggravate the damage potential (e.g., land degradation, landslides, forest fires). Anthropogenic hazards also often interfere with climate and have already led to sea-level increase and a warming of the global temperature (Ranke 2016).

Natural processes or phenomena and human activities can be considered "hazards" when they have the potential to adversely impact on human life and infrastructures, even if they occur in uninhabited areas. This is the main difference to risk, which clearly requires the impact of a hazard on a vulnerable and exposed element. Hazard is indeed independent from a real impact on human life and infrastructures.

Methods of hazard assessment include different approaches such as deterministic, probabilistic, scenarios, indicators, process modelling, time-dependent, or time-independent. In any case, they fall under two main categories: forecast (or projection) and prediction.

Forecast is a comparatively imprecise statement of the time, place, and nature of expected activity, contingent on a scenario; whereas a prediction is a comparatively precise statement of the time, place, and, ideally, the nature and size of impending activity. A prediction usually covers a shorter time period than a forecast and is generally based dominantly on interpretations and measurements of ongoing processes and secondarily on a projection of past history (Banks et al. 1989). Forecasts can be further subdivided into long and short-term forecasts. Long-term forecasts are those that pertain to the coming years, decades, or longer. Most hazard assessments are essentially long-term hazard forecasts. Short-term forecasts are those that pertain to the coming hours, days, and weeks, and are usually issued when unrest is escalating sharply, or a hazardous phenomenon has just begun.

Hazard assessment is strongly dependent on the representative indicator (or intensity in a more general way), selected to rank a given natural or human phenomenon. The indicators are related to many factors, such as the spatial "size" affected by a phenomenon, but also the final-user demand.

The spatial dimension of phenomena is clearly a main topic, constraining the methodology to various scales of analysis and approach. As an example, landslide hazard mainly reflects the physical attributes of a potentially damaging mass movement in terms of mechanism, magnitude, and frequency. Since the geographical dimension of a landslide is typically local, any methodology has to reflect such a large scale. In the case of an earthquake, the hazard is given by a sudden release of energy in the Earth's crust or upper mantle as a result of an abrupt shift of rock along a fracture. Small-scale approaches are then adopted when assessing the seismic hazard (microzoning is required when dealing with hazard at a local site). In other words, any natural hazard must be assessed and mapped (preferably) at the scale at which it occurs.

Specific methodologies and spatial resolutions have been identified in hazard assessment (Armonia 2005a), relating to three different strategic approach, corresponding to four different scales. The main conceptual differences between these approaches, apart from the spatial resolution, are as follows:

- The regional strategic approach is mainly developed for regional planning, that is the task of settling the spatial or physical structure and development by drawing up regional plans as an integrated part of a formalized planning system of a state. Regional planning is required to specify aims of spatial planning, which are drawn up for an upper, overall level. The regional level represents the vital link between a state-wide perspective for development and the concrete decisions on land use taken at a local level within the land-use planning of the municipalities. Its textual and cartographic determinations and information normally range across the scale of 1:50,000 to 1:100,000. Smaller scales can be produced as a general view, at scales from 1:100,000 to 1:1,000,000. The regional approach mainly uses qualitative/susceptibility indicators and areas prone to natural events, in discrete categories (e.g., high, medium, low).
- The local general approach is mainly developed for land-use planning and management at the municipality level. The main outcome is the probability that a specific event may occur in a given area within a given time window. Creation of policies at a local/municipal level that guide land and resource uses (inside the administrative borders of a municipality, in charge of this task) fall under this category. Sometimes "urban planning" is used as a synonym. The main instrument of land-use planning is zoning or zoning ordinances, respectively. Land-use planning is situated below the regional planning level and consists normally of two stages: first a general or preparatory land-use plan (scale 1:5,000–1:50,000) for the whole municipality and second a detailed land-use plan for a small part of it, which is mostly legally binding (scale 1:500–1:5,000).
- For a point shape location, rather in depth analyses can be carried out, integrating geotechnical investigation if needed and wherever the relevance or the hazardousness of what needs to be built is high. This is the so-called site engineering approach (site specific) that is generally developed at a site scale, ranging from 1:100 to 1:1000.

In fact, hazard analysis is considered often as the first step or one component of more comprehensive risk assessment; however, it also provides relevant information per se that can be of use to support mitigation measures. The first is certainly early warning systems. Knowledge on the specific characteristics of the hazard(s) in a given area is essential to build reliable monitoring systems and for those phenomena that allow a delay between initial premonitory signals and the occurrence of the extreme event to develop early warning systems. For the latter to be developed, one must know rather precisely the dynamic development of the feared phenomenon, its triggering conditions, and the areas that may be affected. Monitoring systems must be put in place so as to provide enough time for model elaboration and for making and enacting cautionary decisions that are primarily aimed at saving the lives of people but may also to some extent reduce the damage.

Hazard Assessment to Guide Planning

Locational decisions are a set of mitigation measures that can take advantage of hazard assessments, thus defining where an artifact, an infrastructure, or a settlement can be built. Decisions about future land-use have long-term effects, for example on physical structures such as buildings or infrastructure, and are often irreversible. Furthermore, whenever such decisions are taken, risks are unavoidable. Therefore, planning always has to anticipate the consequences of the planned actions. This is particularly relevant in the case of decisions about dangerous industrial structures or managing natural hazards (Fleischhauer et al. 2005).

Land-use and spatial planning come into play when multiple complex decisions regarding not only where to locate buildings, roads, networks, open spaces, and services have to be made but also what is the concentration of people to be expected in different zones and how the latter may be developed and what uses and functions are allowed. In this regard, hazard analysis has been widely used to support this type of decision and for a long time it has been the only basis for decisions, as risk assessments have been lagging behind because of the difficulties in operationalizing the different necessary steps, including in particular vulnerability appraisals. However, for hazard analysis to be of real use in land-use and spatial planning some important requirements need to be fulfilled, regarding the spatial and temporal scales at which phenomena are considered and represented, and the descriptions of the hazardous aspects in the maps' legends.

As for the spatial scale of analysis, it was already noted that the spatial dimension of phenomenon constrains the methodology to a specific scale of analysis and approach. In other words, any natural hazard must be assessed and mapped (preferably) at the scale at which it occurs. However, for planning purposes it is crucial to size the spatial scale of analysis and the consequent level of detail of maps and other representation of the hazard(s) to the scale at which plans are designed (Armonia 2005a).

This distinction between hazard assessments for different planning scales already exists in some good practices. Large-scale hazard analysis is prepared for example by river basin authorities, such as the one codified by the Flood Directive in Europe; local-scale analysis is provided in geological reports that often accompany urban plans. A relevant example is offered by the Risk Prevention Plan in France (Plan de Prévention des Risques) that must provide detail for each municipality area with different levels of hazards.

As for the time scale, hazard factors are not easy to represent for planning purposes because of the different dynamics that characterize hazardous phenomena. Some can be inactive for long periods and then suddenly reactivate thereby changing an apparently stable landscape very abruptly and over a short time, particularly in the case of sudden onset hazards. "While maps generally convey a rather static picture taken in a generally ordinary time or condition, what would be needed is a trustworthy representation of the dynamic processes that shape hazards overtime and may have dramatic accelerations when extreme events occur" (Mejri et al. 2017).

Thirdly, to be effectively used for land-use spatial planners need legends in maps to be comprehensible and convey the information that is relevant for locational and zoning decisions, in particular: the probability of a given event's severity; the maximum severity (expressed as magnitude, intensity, etc.); and the areas that may be affected. In regard to the latter fine-tuned zoning is necessary for local scale plans, whereas coarse grain delimitation is sufficient for regional or county level plans. As for the recurrence overtime and the related severity, both the probabilistic and deterministic approach may be of use for spatial and land-use planning. The first is important to characterize the risk over the lifetime of the plan (that may be prepared for the subsequent 5–10 years, but often remain as the legal reference for much longer) and to link planning to other types of policies and measures, in particular insurance policies. Presently, the latter are often linked to planning policies in that insurance premiums are tailored to the level of risk. Deterministic assessments can be of use to visualize what may be the consequence of a given predetermined event in an area that has to be developed or redeveloped.

Emergency plans are another field of practice that need hazard assessment as key input. Here locational decisions are more temporary than in the case of land-use and spatial plans, since they address areas for evacuation, areas for gathering rescue forces and their means, usability of roads and access ways. Also emergency plans are developed at different spatial scales: at the regional and county level strategic choices regarding deployment of civil protection forces and accessibility issues need to be considered. At the local scale, access internal to settlements must be considered as well as the exact location of areas for emergency uses.

Hazard Analysis Stage: Parameters and Methodologies to Assess Various Hazards

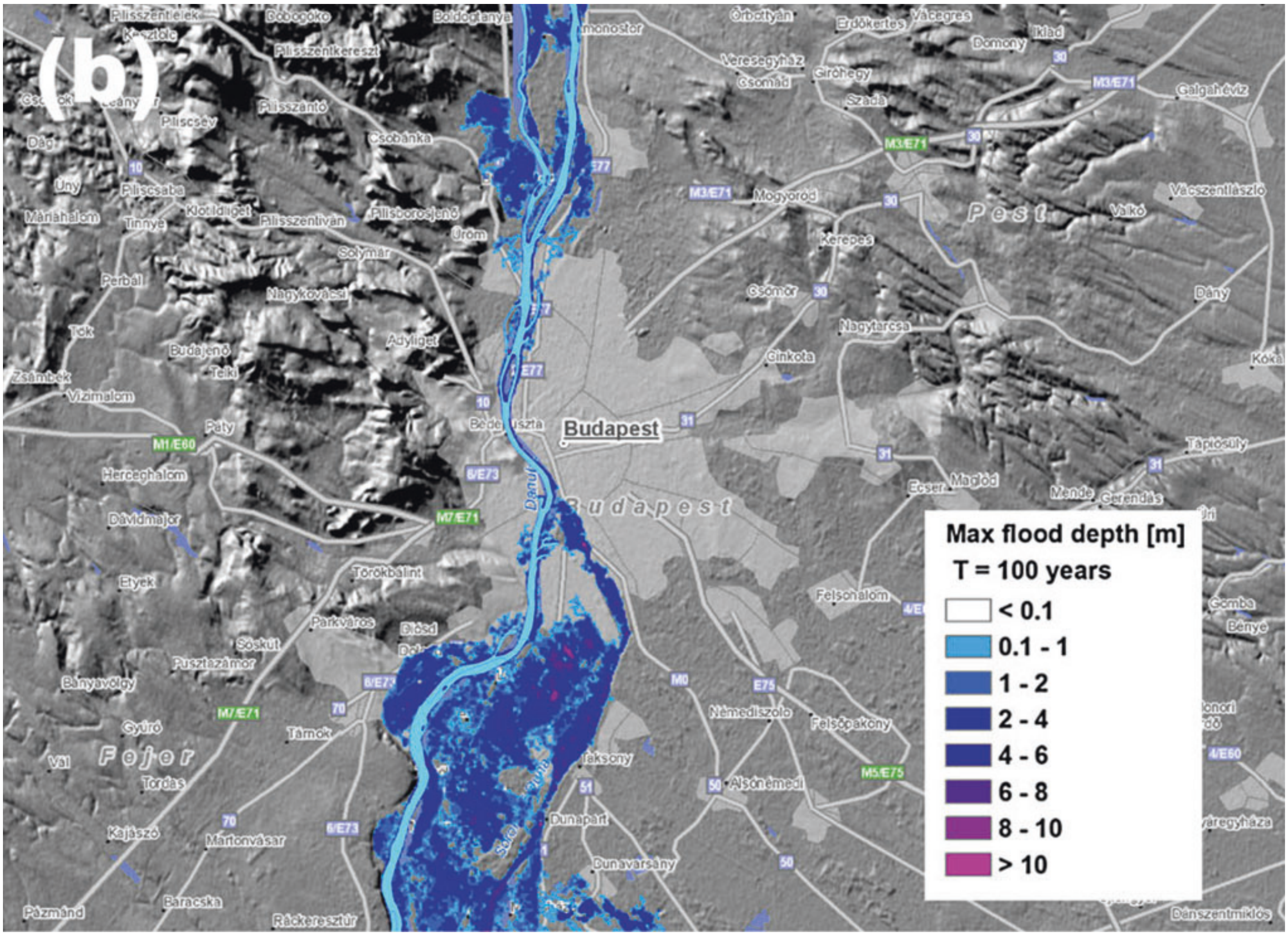
The different typology of natural phenomenon, the physical processes, and indicators used to describe events are reported. Also, for a given phenomenon and for the different scale of hazard assessment, characteristics and use of outcome, data acquisition and mapping procedure, hazard methodology, and suggested legends are explained.

Flood Hazard

Flood investigations need a systemic approach for effectively understanding their dynamic properties including both external/internal factors to derive a rational assessment of their potential impacts. Such information is of basic relevance for planning a set of structural or nonstructural measures for coping with floods. An overview of those key points related to the concept of a systemic approach and the consequent engineering geological issues are required.

Floods may be classified by their various causes, speed of onset, and potential damage. The following broad categories of flood causes are often recognized (Armonia 2005b):

- Flash floods that build up rapidly in mountain areas lowland or plains floods which have a slower and more predictable onset
- Floods from the precipitation during a rainfall event (pluvial floods) or from stored water as snowmelt



Hazard Assessment, Fig. 1 Depth of the flood water for the town of Budapest (in meters) likely to occur in the defined on average once in every one hundred years (1:200). Or a 1% chance of happening in any 1 year (Alfieri et al. 2014)

- Floods from natural events and those from the failure of flood defense infrastructure or "dam breaks"
- Flooding directly from rising groundwater and the contributing antecedent moisture conditions to increase runoff rates
- Flooding from inadequate surface water drainage in urban areas (urban floods) and from minor watercourses and roadside ditches
- Tidal surges and other marine conditions leading to coastal and estuarine flooding (coastal floods)

To evaluate flood hazard fully, the following is needed:

- The areal extent of the floodwater
- The depth of the floodwater (Fig. 1)
- Speed of onset of the flooding
- The velocity of the flow in the floodplain
- How often the floodplain will be covered by water
- The length of time the floodplain will be covered by water
- At what time of year flooding can be expected

In many cases, flood hazard maps only show the flood extent for a particular annual probability of flooding or return period. The size of a flood is generally measured in terms of its flow or discharge (Table 1). The common unit can be expressed in terms of depth and velocity (dynamic) or only the depth (static). Extent of lateral erosion can also provide a measurement of the energy of flood. The method used to estimate the flood hazard is linked to the manner in which flood risk is to be assessed. For example, if the flood risk to people is to be assessed then a flood hazard may need to be assessed in terms of flood depths and velocities. However, if flood risk is to be estimated in terms of economic damage to buildings then the flood hazard is usually established in terms of the floodwater depth and the duration of the inundation.

At different spatial scales, the following approaches can be envisaged:

Hazard Assessment, Table 1 Catchment modelling methods (Armonia 2005b)

Method	Method for flood flow prediction	Method for flood level prediction
Simple hydrological routing	Rainfall-runoff modelling and flow routing	Rating curves derived from a hydrological routing model
Enhanced hydrological routing	Rainfall-runoff modelling and flow routing	Rating curves derived from detailed hydraulic models
Sparse hydrodynamic modelling	Rainfall-runoff modelling	Hydrodynamic model with minimal number of nodes or cross sections

Hazard Map at a Regional/National Scale (>1:50,000)

Provide maps and information that are often used at a catchment level for catchment planning. They usually cover the whole of the river catchment. Such maps usually show the extent and sometimes the depth of the flooding. Data required includes topographic data covering the whole of the country, often such a digital terrain model has a low vertical resolution but at a national level this is generally acceptable. Flood water levels are estimated using simple methods of converting flood flows to water levels (for example, Manning's equation). Flood extent is mapped using the digital terrain model of the catchment.

The hazard methodology is dependent upon the how the risk is to be assessed. It should be possible to map the flood extent and depth for a range of design flood return periods at this scale. An automated process is usually used to convert modelled flood depths to extent and depth within a GIS environment across the entire country. Hazard is expressed in terms of flood extent for a number of annual probabilities or "return periods." The most frequently used return period tends to be 1 in 100 years (i.e., the flood with a 1% annual probability of occurrence). Other return periods that are sometimes used include 1 in 5, 1 in 25, 1 in 50, 1 in 100, 1 in 200, and 1 in 1000 years.

Hazard Map at a Local General Scale (1:5,000–1:50,000)

Provide maps and information that are often used at a catchment level for catchment planning. They usually cover the whole of the river catchment. Such maps usually show the extent and sometimes the depth of the flooding. Data required includes topographic data covering the whole of the catchment. Cross-sections of the watercourse are also required. Flood water levels are estimated using broad-scale hydraulic models that cover the whole of the catchment. Flood extent is mapped using the digital terrain model of the catchment.

The hazard methodology is following the same approach as in the national/regional scale (see above).

Hazard Map at Local Detailed Scale (1:500–1:5,000)

Provide maps and information that are used at a detailed level. They may cover only a few kilometers of the river. The characteristics of such maps are dependent on the nature of the risk to be assessed. Such maps usually show the extent of flooding but they can also show flood depths and velocities. Data required includes detailed hydrological data (including information on historical floods); detailed topographic data of the floodplain and also the watercourse, survey data for relevant hydraulic structures such as weirs and bridges, observed water levels and flow with which to calibrate and verify hydrological and hydraulic modelling. Flood water levels are estimated using a calibrated hydraulic model and flood extent mapped using an accurate digital terrain model of the floodplain.

The hazard methodology is following the same approach as in the national/regional scale (see above).

Hazard Map at Site-Specific Scale (1:100–1:1,000)

Provide maps and information that are used at a very detailed level. They may cover only a few hundred meters of the river. The characteristics of such maps are dependent on the nature of the risk to be assessed. Such maps usually show the extent of flooding but they can also show flood depths and velocities.

Data required includes detailed hydrological data (including information on historical floods), detailed topographic data of the floodplain and also the watercourse, survey data for relevant hydraulic structures such as weirs and bridges, observed water levels and flow with which to calibrate and verify hydrological and hydraulic modelling. Flood water levels are estimated using a calibrated hydraulic model and flood extents mapped using an accurate digital terrain model of the floodplain.

The hazard methodology is following the same approach as in the national/regional scale (see above).

Seismic Hazard

Engineering geology is a basic science needed for seismic hazard assessment, especially when dealing with ground amplification. Indeed, seismic ground motion often generates collateral geotechnical hazards such as slope instabilities, fault rupture, ground deformation, and liquefaction.

Earthquakes are caused by a sudden release of energy in the Earth's crust or upper mantle as a result of an abrupt shift of rock along a fracture. More than 90% of earthquakes are related to plate tectonics and are caused along plate boundaries. The "destructive power" of earthquakes can be described by several indicators, depending on the information available at each singular event. Indicators of earthquake hazard can be collected under two categories, correlated with two different approaches:

Damage effect. The first method is based on the description of damage effects on the built environment, people, buildings, and on the natural environment (macroseismical effects). To this end, several scales have been introduced that allow one to associate the entirety of the effects with an "intensity" level.

The most recent is EMS-98 (Grünthal 1998) and the most famous is the Mercalli scale, both with many modification and adaptations. It is clear that "intensity," as it is defined, provide an evaluation of earthquakes's effects in a given place, and therefore, for the same event, there are different intensity values for different places. Studying earthquakes "intensity" and "severity" are often used as synonyms, because they are an expression of the same category of effects.

Accelerometric registrations. The second method to evaluate earthquakes is through soil motion registration obtained through accelerograms, instruments capable of supplying registration in proportion to earthquake accelerations. The simplest parameter that can be used for this measure is the amount of the maximum recorded acceleration. The *magnitude* (i.e., Richter scale) is, instead, a value that represents the level of energy released by the singular considered event. This value is recorded in the "Historical recorded events catalogue" that associates the magnitude value with the singular earthquake taken into consideration.

Hazard can be assessed in two different ways: considering it is a concept linked to the probability that in a specific site a certain severity event may occur in a predefined time-window ("probability" is the term used), because an exact prevision can't be given for the time being. Statistical analysis of the past events that occurred in a specific site must be done in order to obtain a correct value of the hazard, or considering an individual event (in this case a deterministic approach is followed).

A variety of methods can be classified according to different initial hypotheses, objectives, and detail levels. They are divided into:

A. ***Probabilistic approach:*** permits one to obtain foresight about future events in a specific site, in particular it defines the probability of having an event stronger than an established severity in a given time period, thanks to probabilistic analysis of past events. The result is a distribution function for the site and the determination of possible hazard indicators (Fig. 2).
   A1. *Source zone method.* It is one of the most commonly used methods for hazard assessment. It is based on two hypotheses:
      – Uniform spatial distribution of the events in a seismogenetic context
      – Poisson distribution of occurrence periods
   A2. *Renewal process.* This method modifies the basic hypothesis of the preceding approach, abandoning the hypotheses of stationarity and the uniform spatial distribution to calculate the time window between subsequent events.
B. ***Deterministic approach:*** damage scenario. This considers a singular event and its propagation in surrounding areas (scenario), and permits the study of site effects (damage scenario).

Hazard Map at Regional/National Scale (>1:50,000)

In this scale of representation, it is possible to have a full picture of the national hazard, so it is useful to classify the national territory into seismic areas or not. Necessary data for assessment are seismic catalogues, compiled on the basis of geological, archeological, historical, and instrumental data. Finally, the resulting Seismic Hazard Map reports different horizontal acceleration values for different return periods.

Hazard Map at Local General Scale (1:5,000–1:50,000)

At the regional scale, it is possible to investigate the hazard for areas including a large number of municipalities and it is useful to classify the territory into seismic areas or not (as with the national scale too, but more in detail than in the national scale). This type of classification is of use to draft a preliminary hazardous areas list. Basic seismic hazard is analyzed using deterministic models to investigate the expected shock in a selected area. At this scale, several objectives can be pursued, such as identifying source areas and the events characterized by different recurrence periods. In this way, it is possible to calculate (using a specific attenuation model) the expected shaking at the site. Resulting hazard map reports Expected Maximum Acceleration values (Amax (g)) for different return periods.

Hazard Map at Local Detailed Scale (1:500–1:5,000)

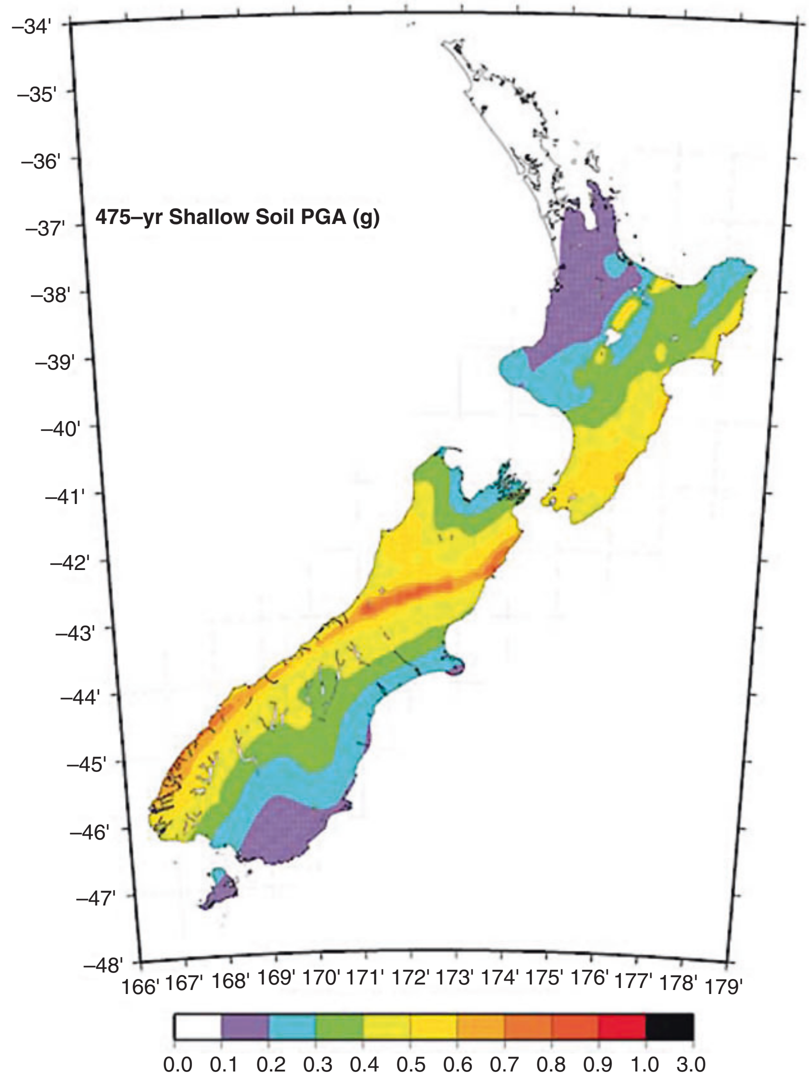
Local-scale studies allow one to find and analyze in depth the local hazard factors present in one or more municipalities. The integration of information regarding possible amplification areas or areas subject to secondary effects such as landslides or liquefaction is key as it provides differential soil characterization at a fine-grain level that is coherent for example with local urban plans.

Hazard Map at Site-Specific Scale (1:100–1:1,000)

Seismic microzonation involves subdividing a region into smaller areas having different potential for hazardous local earthquake effects. Local effects can be distinguished for instability effects and site effects, so the local hazard is investigated by using geological and morphological methods that reproduce conditions characterizing a specific area (i.e., topographical irregularity, geotechnical properties, geophysical properties, landslides). Hazard conditions can be analyzed adopting different methodologies, which provide different results: they include quantitative, semiquantitative, and qualitative approaches. An ideal standard outcome should consider in highly threatened areas: spectrum of elastic response and amplification coefficient according to geology, morphology, geotechnical, and geophysical data.

Landslide Hazard

Landslides play a prominent role in a wide range of engineering geology studies. Landslides are defined as the gravitational movement of a mass of rock, Earth, or debris down a



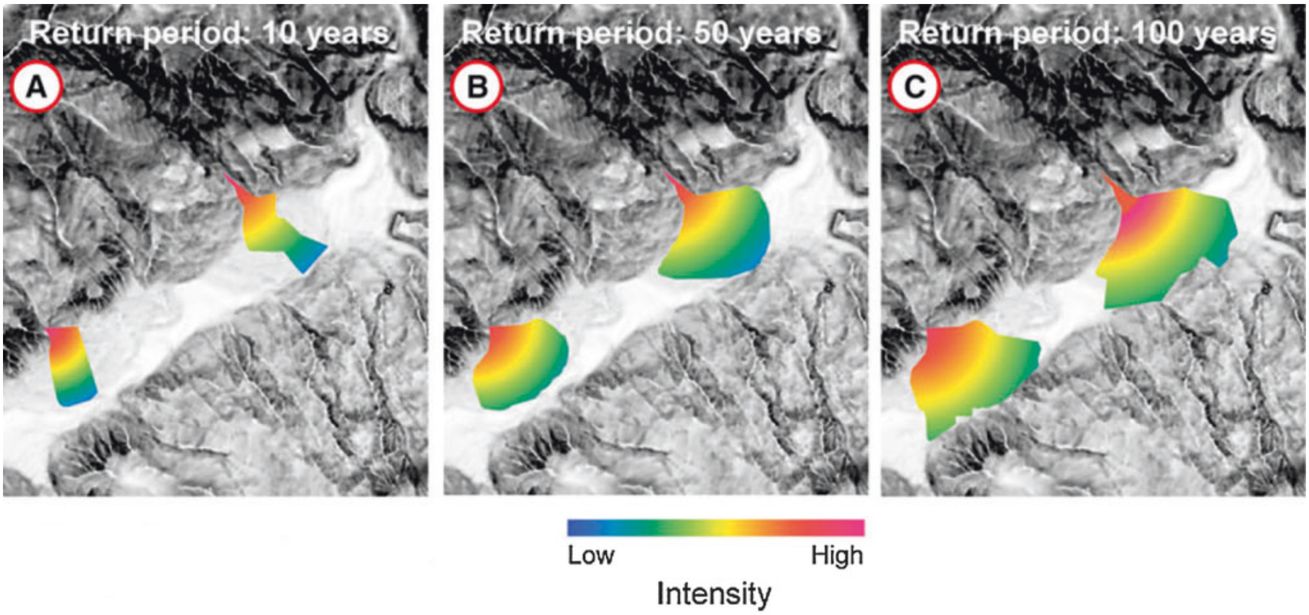
Hazard Assessment, Fig. 2 The 2010 National seismic hazard model for New Zealand showing expected peak ground accelerations (g) for a 475-year return period earthquake for shallow soils (Stirling et al. 2012)

slope (Cruden 1991), which are basically described by two characteristics: (1) the material involved (rock, debris, Earth) and (2) the type of movement (falls, topples, slides, spreads, flows) (Cruden and Varnes 1996). The above classification, that is, rockfall, debris-flow, Earth-slide, facilitates the understanding of the failure mechanism. Indicators to be used in hazard assessment generally refer to number of landslides in a given area and geometrical and mechanical severity of a landslide (depending on the scale). It may be expressed as a relative scale or as one or more landslide parameters (i.e., velocity, volume, energy) (Fig. 3).

A conceptual distinction between magnitude and intensity is reported by Hungr (2002) in the dynamic analysis of fast-moving landslides (flows and slides). The magnitude is a parameter that describes the scale of an event. As a general rule, the volume of the involved material can express the magnitude of a landslide. The intensity, as for earthquakes, is not a single parameter but a spatial distribution of different characteristics that describe, qualitatively or quantitatively, the impact of a landslide in different sites. Velocity, duration of movement, height of displaced mass, and depth of deposits are some of the quantitative parameters of intensity (Table 2). With respect to other fields such as seismicity and floods where the intensity may be clearly defined, the definition of the severity of a landslide (in terms of intensity or magnitude) is a very difficult task for experts due to the objective difficulty in the assessment of the various parameters. In addition, the expression of intensity related to potential damage or losses should be avoided since this depends on the vulnerability of potential exposed element.

Hazard Map at Regional/National Scale (>1:50,000)

Provide a general inventory of landslide areas or susceptibility maps with low level of detail. The maps are useful to national policy makers and the general public. Necessary data are a national summary of regional landslide inventories



Hazard Assessment, Fig. 3 Landslide (debris flow) hazard maps for various return periods. Each of the scenarios yields intensity maps in terms of impact pressure (Corominas et al. 2014, modified)

Hazard Assessment, Table 2 Velocity classes of landslides from Hungr et al. (2014)

Velocity class	Description	Velocity (mm/s)	Typical velocity	Response
7	Extremely rapid	5×10^{3}	5 m/s	Nil
6	Very rapid	5×10^{1}	3 m/min	Nil
5	Rapid	5×10^{-1}	1.8 m/h	Evacuation
4	Moderate	5×10^{-3}	13 m/month	Evacuation
3	Slow	5×10^{-5}	1.6 m/year	Maintenance
2	Very slow	5×10^{-7}	16 mm/year	Maintenance
1	Extremely slow			Nil

and map products. Susceptibility maps are generally derived from a geomorphological approach based on spatial distribution of landslides, landslide density, landslide activity; indexed maps; and descriptive statistical analysis. Methodologies refer to deterministic approaches (i.e., geotechnical modelling coupled with hydrological analysis), statistical modelling, geomorphological approach, and indexed maps. Rarely, when the study area is homogeneous in terms of geological, morphological, and landslide types, hazard description is possible and can be expressed as landslide probability (affected area, return time, intensity) or safety factor range. Generally, a relative hazard is provided in qualitative scales that depict spatial and/or temporal probability of occurrence (i.e., low, medium, high, very high) or simply with density of landslides per area (i.e., km^2).

Hazard Map at Local General Scale (1:5,000–1:50,000)

Identify the landslide relative hazard or susceptibility maps. The investigations may cover quite large areas and the required map detail is medium-low. The maps are generally addressed to large projects (feasibility studies) or developments. Detailed data collection for individual factors (i.e., landslide inventory, lithology, structural setting, land use), mostly derived by remote sensing techniques and bibliography in order to delineate homogeneous terrain units. Other data include statistical modelling, a geomorphological approach based on a detailed landslide inventory, and indexed maps. Hazard maps are possible only when the geomorphic and geologic conditions, as well as landslide types, are fairly homogeneous over the entire study area. A relative hazard is provided in qualitative scales (e.g., Delmonaco et al. 2003) that depict spatial probability of occurrence (i.e., low, medium, high, very high).

Hazard Map at Local Detailed Scale (1:500–1:5,000)

Provide an overview of potential unstable slopes for large engineering structures, roads, urban areas, and soil protection (detailed studies). Absolute hazard and/or relative hazard should be evaluated according to landslide types occurring in the study area. Data collection should support the production of detailed multitemporal landslide distribution maps and provide information about the various parameters required in the adopted methodology. Methodologies for hazard assessment include deterministic (i.e., geotechnical modelling coupled with hydrological analysis), statistical modelling, geomorphological approach, and indexed maps. Hazard maps are possible only when the geomorphic and geologic conditions, as well as landslide types, are fairly homogeneous over the entire study area. Rarely, when the study area is homogeneous in terms of geological, morphological, and landslide types, hazard determination is possible and can be expressed as landslide probability (affected area, return time, intensity) or safety factor range. Generally, a relative hazard is provided in qualitative scales (e.g., Delmonaco et al. 2003) that depict spatial and/or temporal probability of occurrence (i.e., low, medium, high, very high).

Hazard Map at Site-Specific Scale (1:100–1:1,000)

Provide absolute hazard classes and variable safety factor related to specific triggering factors. The maps are used for implementation and design of landslide hazard and risk mitigation projects. Data are related to slope stability modelling parameters (i.e., stratigraphy, geotechnical properties, hydrological data, seismic input). Hazard methodology is mainly deterministic, such as a geotechnical modelling–based geomophological survey and geotechnical laboratory data. Hazard classes expressed as failure probability (affected area, return time, intensity) or safety factor range. An ideal landslide hazard map shows not only the chances that a landslide may form at a particular place but also the chance that it may travel downslope a given distance.

Volcanic Hazard

Engineering geology and geotechnical engineering are always involved in the various stages of dealing with volcanic hazards. These include hazard identification, evaluation and zonation, risk assessment, monitoring, evacuation, exploration, redevelopment, and construction. Volcanic hazard assessment is the probability of a given area being affected by potentially destructive volcanic processes or products within a given period of time (Fournier d'Albe 1979). Technically, therefore, the actual destructive volcanic processes themselves should be referred to as "hazardous volcanic phenomena" rather than as "volcanic hazards." However, the popular understanding of the word "hazard" as a "source of danger" means that potentially dangerous eruptive and posteruptive phenomena such as pyroclastic flows, windborne ash, lava flows, volcanic gases, and lahars can also be referred to as "volcanic hazards" when not used in the context of probabilistic assessments.

Volcanoes can produce a variety of hazardous phenomena with variable frequency. Such phenomena may occur during an eruption (direct hazards) or before or after an eruption (indirect hazards). The latter include the ever-present hazardous phenomena related to the presence of a live volcano, such as volcanic earthquakes and volcanic gases. Crandell et al. (1984) have distinguished two broad categories of hazards: (1) short-term (or intermediate) hazards are those that occur at such high frequency (more than once per century) that inhabitants of the area will likely experience them; and (2) long-term (or potential) hazards are those that occur at such low frequency (less than once per century) that they will not likely be experienced by people alive today.

The intensity of an eruption is a measure of the rate at which magma is discharged during an eruption. It is defined as the mass eruption rate and is expressed in kg/s. An intensity scale, based on a logarithmic index of intensity is defined by:

Intensity $= \log_{10}$ (mass eruption rate, kg/s) + 3

On this scale, an extremely vigorous eruption will have an intensity of 10–12, whereas a very gentle eruption might have an intensity of 4 or 5 (Pyle 2000). In the case of a pyroclastic fall, the potential local intensity can be established on the basis of thickness of ash fall (Armonia 2005a) or the load (Fig. 4).

Newhall and Self (1982) developed the Volcanic Explosivity Index (VEI) that is still a relevant indicator for global volcanic eruption magnitude. It is a relative scale that enables explosive volcanic eruptions to be compared with one another (Table 3). Finally, Pyle (2000) defined the magnitude as the total mass of material ejected during an eruption, expressed in kg. A magnitude scale, based on a logarithmic index of magnitude, is defined as follows:

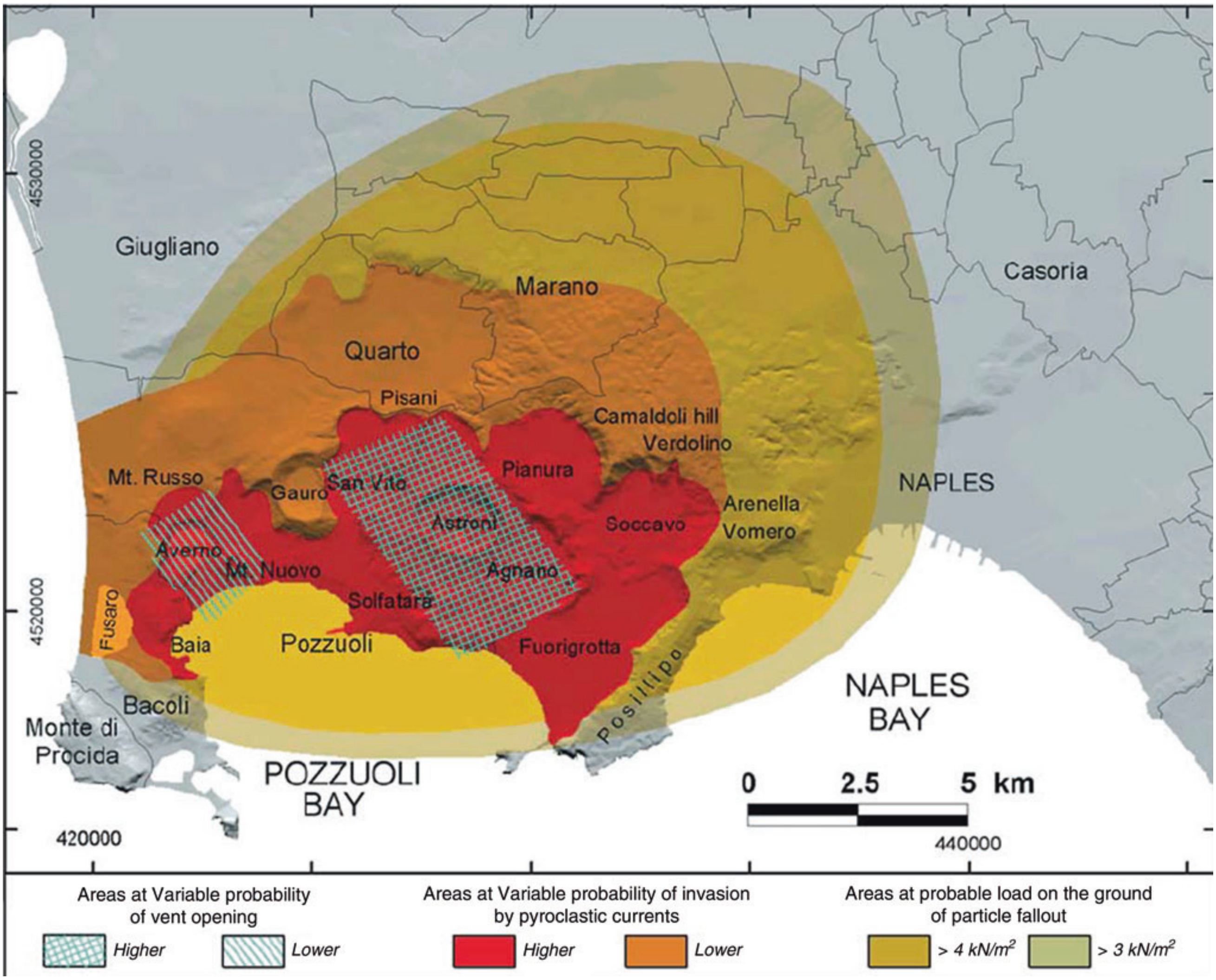
Magnitude $= \log_{10}$ (erupted mass, kg) $- 7$

According to this scale, a large eruption of a Plinian type is of magnitude 6 or more. Table 3 provides a compilation of the three above-mentioned categories of indicators.

Hazard Map at Regional/National Scale (>1:50,000)

For a given volcano, or more than one volcano, defines the areas which can be intersected by volcanic phenomena, characterized by a very extended impact on the territory (e.g., some kind of high mobility lava flows, ash and lapilli fallout, high mobility pyroclastic flows and surges, large debris avalanches, volcanic gases, tsunamis). The maps are used for national planning of volcanic emergencies.

Data acquisition and mapping include geological investigations, focused at defining the past behavior and the present state of a given volcano and the evaluation of paleomorphology and current topography; structural analysis aimed at the identification of the nature and mechanisms of past deformation events, such as caldera collapse and caldera resurgence; geomorphological studies aimed at the definition of the areas which could be affected by remobilization of tephra, also at great distance from the volcano; volcano monitoring, which provides an indication of when and where future activity may occur, and insights into the likely style of activity and possible areas affected. Comparisons with similar volcanoes provide an indication of possible activity that may be either unprecedented or not preserved in the geologic record for the volcano in question. The hazard methodology includes evaluation of possible phenomenologies;



Hazard Assessment, Fig. 4 Volcanic hazard map of the Campi Flegrei caldera, with joint reference to vent opening, probability of invasion by pyroclastic currents and tephra fallout (Orsi et al. 2004)

Hazard Assessment, Table 3 Indicators for volcanic hazard

Volcanic eruptions (Newhall and Self 1982)	0	1	2	3	4	5	6	7	8	Index	VEI index
	Nonexplosive	Small	Moderate	Moderate-large	Large	Very large				General description	
	Gentle	Effusive	Explosive		Cataclysmic, paroxismal					Qualitative description	
	10^4	10^6	10^7	10^8	10^9	10^{10}	10^{11}	10^{12}	10^{13}	Maximum erupted volume of tephra (m^3)	
	<0.1	0.1–1	1–5	3–15	10–25	>25				Eruption cloud column height (km)	
Volcanic eruptions (Pyle 2000)	<3 low		3–8 moderate	8–10 medium	10–11 high	>11 very high				Intensity = $\log_{10}$ (mass eruption rate, kg/s) + 3	
	<2 low		2–3 moderate	3–4 medium	4–5 high	>5 very high				Magnitude = $\log_{10}$ (erupted mass, kg) − 7	
Pyroclastic fall (Armonia 2005b)	0–1000/5000 (no upper limit)									Thickness of fall (cm)	

deterministic approach (e.g., load on the ground, dynamic pressure, temperature); statistical approach (probability); meteorological data on wind strength and direction; and numerical modelling. Hazard classes expressed as areas at different probability to be affected by the examined phenomenologies; isopachs and isopleths maps of pyroclastic fallout.

Hazard Map at Local General Scale (Approximately 1:5,000–1:50,000)

For a given volcano, a hazard map defines the areas which can be intersected by volcanic phenomena, characterized by an extended impact on the territory (e.g., some kinds of lava flows, ash and lapilli fallout, ballistic projectiles, some energetic lateral blasts, high mobility pyroclastic flows and surges, lahars and debris avalanches, debris flows and mud flows, volcanic gases, volcanic earthquakes, intermediate-scale tsunamis). Depending on the type of volcano, the maps are used for national planning of volcanic emergencies or regional management of volcanic crisis. In this case, these maps have to be inserted into the framework of national emergency plans.

Necessary data consider geological investigations focused at defining the past behavior and the present state of a given volcano and at the evaluation of paleomorphology and current topography; structural analysis aimed at the identification of the nature and mechanisms of past deformation events, such as caldera collapse and caldera resurgence; geomorphological studies aimed at the definition of the areas which could be affected by remobilization of tephra, also at large distances from the volcano; volcano monitoring, which provides an indication of when and where future activity may occur, and insights into the likely style of activity and possible areas affected; comparisons with similar volcanoes, which provide an indication of possible activity that may be either unprecedented or not preserved in the geologic record at the volcano in question.

Methodologies in use for hazard assessment include evaluation of possible phenomenologies; deterministic approach (e.g., load on the ground, dynamic pressure, temperature); statistical approach (probability); meteorological data on wind strength and direction; numerical modelling. Hazard classes are expressed by areas of different probability to be affected by the examined phenomenologies and isopachs and isopleths maps of pyroclastic fallout.

Hazard Map at Local Detailed Scale (Approximately 1:500–1:5,000)

For a given volcano, it defines the areas which can be intersected by volcanic phenomena, characterized by a limited extended impact on the territory (e.g., some kinds of lava flows, lava domes, some lateral blasts, low mobility and dilute and turbulent pyroclastic density currents, intermediate-scale lahars and debris avalanches, volcanic gases, volcanic earthquakes, lightning strikes, small-scale tsunamis). The maps are used for local management of volcanic crisis and have to be inserted in the framework of national emergency plans.

Necessary data consider geological investigations focused at defining the past behavior and the present state of a given volcano and at the evaluation of paleomorphology and current topography; structural analysis aimed at the identification of the nature and mechanisms of past deformation events, such as caldera collapse and caldera resurgence; volcano monitoring, which provides an indication of when and where future activity may occur, and insights into the likely style of activity and possible areas affected; comparisons with similar volcanoes, which provide an indication of possible activity that may be either unprecedented or not preserved in the geologic record for the volcano in question. Hazard methodology focusses on evaluation of possible phenomenologies, deterministic approach (e.g., dynamic pressure, temperature), statistic approach (probability), and numerical modelling. Hazard classes expressed as areas of different probability to be affected by the examined phenomenologies.

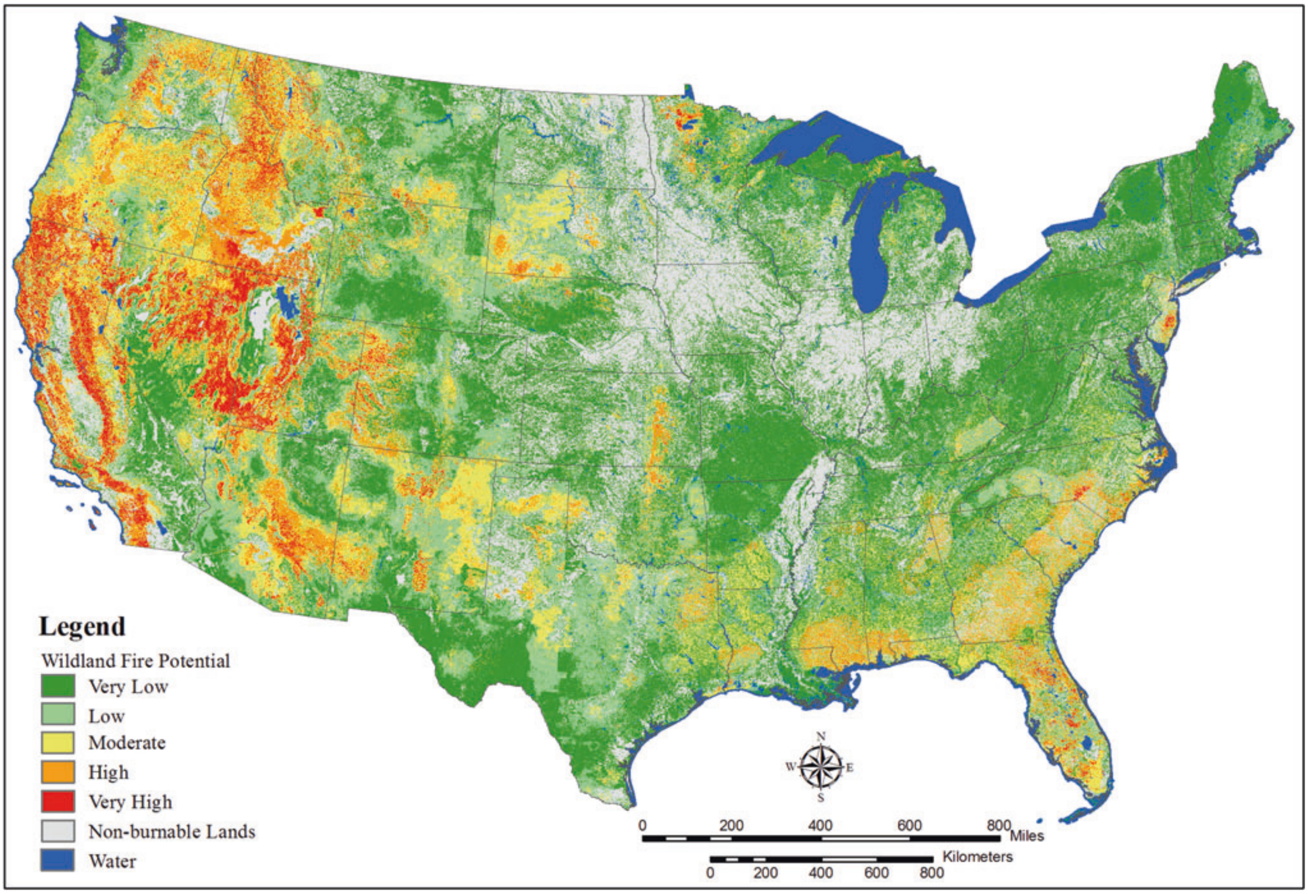
Hazard Map at Site-Specific Scale (Approximately 1:100–1:1,000)

For a given volcano, the map defines the areas which can be intersected by volcanic phenomena, characterized by a limited impact on the territory (e.g., some kinds of lava flows, lava domes, some lateral blasts, low mobility pyroclastic density currents, small lahars and debris avalanches, volcanic gases, volcanic earthquakes, lightning strikes). The maps are used for local management of volcanic crisis.

Necessary data consider geological investigations focused at defining the past behavior and the present state of a given volcano and at the evaluation of paleomorphology and current topography; structural analysis aimed at the identification of the nature and mechanisms of past deformation events, such as caldera collapse and caldera resurgence; volcano monitoring, which provides an indication of when and where future activity may occur, and insights into the likely style of activity and possible areas affected; comparisons with similar volcanoes, which provide an indication of possible activity that may be either unprecedented or not preserved in the geologic record at the volcano in question. Hazard methodology focusses on evaluation of possible phenomenologies; deterministic approach (e.g., dynamic pressure, temperature), statistic approach (probability), and numerical modelling. Hazard classes are expressed as areas of different probability to be affected by the examined phenomenologies. Maps are not available. These analyses are still experimental.

Forest Fire Hazard

Forest fires produce, very often, a severe acceleration for soil erosion and increased incidence of slope instability. These



Hazard Assessment, Fig. 5 Wildfire hazard potential for the conterminous United States (Dillon et al. 2015)

secondary effects of fire pose a considerable challenge to engineering geologists. Fires are a natural disturbance, which are essential for the regeneration of certain tree species and ecosystem dynamics. In addition, fire has been used in the environmental context for many purposes, including shrub removal in the forest and straw burning in agriculture.

Hazard Map at Regional/National Scale (>1:50,000)
National scale is used for regional fire management plans. It supports the spatializing of general fire protection priorities, the definition of protection strategies, the allocation of protection resources, and the establishment of general fire management guidelines.

The following basic information is normally required for this kind of map: DEM, land use, fuel types, fire data (last 10–15 years), administrative boundaries, climatic data, bioclimatic regions, and WUI (Wildland Urban Interface) areas (settlements, road network, socioeconomic variables). Fire occurrence is assessed using kernel density probability based estimates or fire frequency distribution analysis at the municipality level. Fire behavior potential is assessed based on fire simulation models or *ad hoc* empirical methods derived from statistical analysis of local environmental and anthropogenic factors. Quantitative legends show the average potential fire line intensities (in classes) under given meteorological scenarios. Qualitative classes are more conveniently applied for fire occurrence. The final overall legend would be qualitative with for example 5 classes of fire hazard (Fig. 5).

Hazard Map at Local General Scale (1:5,000–1:50,000)
This is the typical scale for local fire management plans. The map is aimed at spatializing protection priorities, the identification of prevention measure, and the establishment of management guidelines at a landscape level. The following basic information is normally required for this kind of map: DEM, fuel model map, settlements, road network, weather patterns, and administrative boundaries. If available: fire perimeters of the past 5–10 years or fire frequency in the municipalities of the past 10–15 years are needed. Fire occurrence is assessed considering buffers of given distances from roads and/or settlements. Fire behavior potential is assessed with fire simulation models. Quantitative legend that shows the average potential fire line intensities (in classes) under given meteorological scenarios, combined with 2–3 expected fire occurrence pattern (qualitative).

Hazard Map at Local Detailed Scale (1:500–1:5,000)
Detailed map produced for site-specific design and location of prevention measures (such as firebreaks, water reservoirs, look out points). Also used for creating management rules for individual settlements and/or specific ecosystems (fuel management, forest management, etc.). The following basic information is normally required for this kind of map: DEM, fuel model map, settlements, road network, weather patterns, and administrative boundaries. If available the fire perimeters of the past 5–10 years or fire frequency in the municipalities for the past 10–15 years might be used. Fire behavior potential is assessed with fire simulation models. Quantitative legend shows the average potential fire-line intensities under given meteorological scenarios.

Hazard Map at Site-Specific Scale (1:100–1:1,000)
Normally not used.

Extreme Rain Storm Hazard

Extreme rain storms are known for triggering devastating flash floods in various regions of the world. They are defined as rainfall greater than a given threshold in a defined time window. When this threshold is higher than the resilience of the affected territory the impact is often catastrophic. The rainfall intensity is a function of spatial density of monitored data and the consequent impact, and is a function of territorial vulnerability, season of occurrence, and other factors. The more local the intensity the more precise is the forecast. Early warning is one of the most relevant applications of rainfall intensity. Warning systems are generally based on rainfall measurements from rain gauges and weather radar and, in most advanced systems, on forecasts.

Studies on predicting and mapping rainfall hazard are characterized by the absence of accurate theoretical knowledge on start and development of large storm cells whose mechanism of localization and immobility (which will generate a concentration of rainfall on a restricted area rather than a wide dispersion) are partially unknown and not easily reproduced on the basis of numerical models. Modern radar surveys can deterministically overcome such limitations, especially for short term forecast.

Hazard assessment is mainly based on elaboration from single meteo data stations, by providing the return period for a given indicator (i.e., 6 h rainfall, 12 h rainfall, 24 h rainfall). Relevant issue is the definition of rainfall intensity as potential threshold values for an alarm. This is typically dependent on the element to investigate and, even if all thresholds are generally described with the same intensity indicator (e.g., mm/time), the adopted methodologies can be remarkably different (Table 4).

Snow Avalanche Hazard

Snow avalanche hazard assessment, monitoring, and mitigation are, similar to landslides, a prominent area of study within engineering geology. According to the multilanguage glossary developed by the Group of European Avalanche Warning Services (http://www.avalanches.org/), an avalanche is a "… rapidly moving snow mass in volumes exceeding 100 m^3 and minimum length of 50 meters…." Avalanches range from small slides barely harming individuals, up to catastrophic events endangering mountain settlements or traffic routes. Avalanche formation is the result of a complex interaction between terrain, snow pack, and meteorological conditions (EEA 2010).

The European Avalanche Danger Scale contains five ascending danger levels: low–moderate–considerable–-high–very high. These danger levels are described by reference to the snowpack stability and the avalanche-triggering probability, as well as the geographical extent of the avalanche-prone locations and the avalanche size and activity (Table 5). The snowpack stability forms the basis of all statements concerning the avalanche danger because it directly controls the probability of an avalanche being released (Ranke 2016).

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The influence of possible snow coverage as well as existing glaciers in the identification of avalanche susceptibility maps must be analyzed in a regional approach. The analysis is strictly connected with snow storm occurrences, and the frequency distribution is to be evaluated in order to be in agreement with these factors, especially in the lower altitude range.

At a local scale, all potential hazard areas will be zoned regardless of the frequency of avalanches. The hazard zones are divided into two areas:

- Starting zones
- Runout zones

The starting zones include all areas on the map which are steeper than 30° and are not covered by dense forest (Delmonaco et al. 1999). The identification of starting zones is done automatically by the computer using vector information. The runout zones are identified by using 3D terrain profile in each avalanche path. Depending on map content and methods used in data collection and data processing, three types of hazard maps can be distinguished (Delmonaco et al. 1999).

- Hazard registration maps; these maps contain historically known slides and avalanches, compiled from literature and documents, interviews, and field work
- Geomorphic hazard maps; maps containing information of hazard prone areas identified by geomorphological investigation in the field, and by the use of topographic maps and air photos
- Hazard zoning maps; maps which define risk areas compiled on the basis of known historic events, geomorphological investigations, and the use of frequency/runout calculation models

Hazard Assessment, Table 4 Rainfall and climate variables used in the literature for the definition of rainfall thresholds for the initiation of landslides. Table lists the variable, the units of measure most commonly used for the parameter, and the author(s) who first introduced the parameter. Nomenclature is not consistent in the literature, and different definitions have been used for the same or similar variables (Guzzetti et al. 2008)

Variable	Description	Units	First introduced
D	Rainfall duration; the duration of the rainfall event	h or days	Caine (1980)
DC	Duration of the critical rainfall event	h	Aleotti (2004)
E	Cumulative event rainfall; the total rainfall measured from the beginning of the rainfall event to the time of failure; also known as storm rainfall	mm	Innes (1983)
EMAP	Normalized event rainfall; cumulative event rainfall normalized to MAP (EMAP = E/MAP); also known as normalized storm rainfall	–	Guidicini and Iwasa (1977)
C	Critical rainfall; the total amount of rainfall from the time of a distinct increase in rainfall intensity (t0) to the time of the triggering of the first landslide (tf)	mm	Govi and Sorzana (1980)
CMAP	Normalized critical rainfall; critical rainfall divided by MAP (CMAP = C/MAP)	–	Govi and Sorzana (1980)
R	Daily rainfall; the total amount of rainfall for the day of the landslide event	mm	Crozier and Eyles (1980)
RMAP	Normalized daily rainfall; daily rainfall divided by MAP (RMAP = R/MAP)	mm	Terlien (1998)
I	Rainfall intensity; the average rainfall intensity for the rainfall event	mm h^{-1}	Caine (1980)
IMAP	Normalized rainfall intensity; rainfall intensity divided by MAP (IMAP = I/MAP)	h^{-1}	Cannon (1988)
IMAX	Maximum hourly rainfall intensity; the maximum hourly rainfall intensity	mm h^{-1}	Onodera et al. (1974)
IP	Peak rainfall intensity; the highest rainfall intensity (rainfall rate) during a rainfall event; available from detailed rainfall records	mm h^{-1}	Wilson et al. (1992)
Î(h)	Mean rainfall intensity for final storm period; "h" indicates the considered period, in hours, most commonly from 3 to 24 h	mm h^{-1}	Govi and Sorzana (1980)
IF	Rainfall intensity at the time of the slope failure; available from detailed rainfall records	mm h^{-1}	Aleotti (2004)
IC	Critical hourly rainfall intensity	mm h^{-1}	Heyerdahl et al. (2003)
IFMAP	Normalized rainfall intensity at the time of the slope failure; rainfall intensity at the time of the slope failure divided by MAP (IFMAP = IF/MAP)	h^{-1}	Aleotti (2004)
A(d)	Antecedent rainfall. The total (cumulative) precipitation measured before the landslide triggering rainfall event; "d" indicates the considered period in days	mm	Govi and Sorzana (1980)
AMAP	Normalized antecedent rainfall; antecedent rainfall divided by MAP (AMAP = A/MAP)	–	Aleotti (2004)
MAP	Mean annual precipitation; for a rain gauge, the long-term yearly average precipitation, obtained from historical rainfall records; a proxy for local climatic conditions	mm	Guidicini and Iwasa (1977)
RDs	Average number of rain days in a year (rainfall frequency); a rain day is a day with at least 0.1 mm of rain; for a rain gauge, the long-term yearly average of rain days obtained from historical rainfall records; a proxy for local climatic conditions	#	Wilson and Jayko (1997)
RDN	Rainy-day normal; for a rain gauge, the ratio between the MAP and the average number of rain days in a year (RDN = MAP/RDs)	mm/#	Wilson and Jayko (1997)
N	Ratio between MAPs in two different areas	–	Barbero et al. (2004)

Tsunami Hazard

A wave, or series of waves, generated when a large volume of water is vertically/horizontally displaced by an impulsive disturbance such as an earthquake, landslide, or volcanic eruption. Tsunami is distinguished from regular sea waves by their long wavelength and period. "Tsunami" and "tsunamis" are both used for the plural in English. There is no pluralizing suffix "s" used in the Japanese language (Power and Leonard 2013). The easiest tsunami indicator is the sea-surface elevation at various times. Some intensity scales have been proposed as evaluation of the impact on an urban and natural environment (among others Lario et al. 2016; Papadopulos and Imamura 2001; Tinti et al. 2011).

Tsunami hazard assessment can take two main forms. It can be based on a scenario approach, where models are made to represent one or more likely situations; or it can be based on a probabilistic approach, in which a spectrum of possible events are analyzed and weighted according to their likelihood. This latter approach is in its infancy for tsunami modelling, but allows for a more systematic comparison of hazards between different locations and across different types of phenomena (Power and Leonard 2013).

Hazard Assessment, Table 5 The European Avalanche Danger Scale (Ranke 2016)

Danger level	Snow pack stability	Avalanche trigger probability	Consequences for infrastructure	Consequences for persons outside secured zones
Low	The snowpack is generally well bonded and stable	Triggering is generally possible only with high additional loads on very few steep extreme slopes. Only sluffs and small natural avalanches are possible	No danger	Generally safe conditions
Moderate	The snowpack is only moderately well bonded on some steep slopes; otherwise it is generally well bonded	Triggering is particularly possible with high additional loads, mainly on the steep slopes indicated in the bulleting. Large-sized natural avalanches not expected	Low danger of natural avalanches	Mostly favorable conditions. Careful route selection, especially on steep slopes of indicated aspects and altitude zones
Considerable	The snowpack is moderately to weakly bonded on many steep slopes	Triggering is possible, sometimes even with low additional loads mainly on the steep slopes indicated in the bulleting. In certain conditions, a few medium and occasionally large-sized natural avalanches are possible	Isolated exposed sectors are endangered. Some safety measures recommended on those places	Partially unfavorable conditions. Experience in the assessment of avalanche danger is required. Steep slopes of indicated aspects and altitude zones should be avoided if possible
High	The snowpack is weakly bonded on most steep slopes	Triggering is probable even with low additional loads on many steep slopes. In certain conditions, frequent medium and also increasingly large-sized natural six avalanches are expected	Many exposed sectors are endangered. Safety measures recommended in those places	Unfavorable conditions. Extensive experience in the assessment of avalanche danger is required. Remain in moderately steep terrain/heed avalanche runout zones
Extreme	The snowpack is generally weakly bonded and largely	Numerous large natural avalanches are expected even on moderately steep terrain	Acute danger. Comprehensive safety measures required	Highly unfavorable conditions Avoid open terrain

According to Tinti et al. (2011), the following maps can characterize the hazard assessment.

- **Regional tsunami hazard scenarios.** They consist of a number of different type maps showing the large-scale tsunami propagation between the source zone and the target. They include tsunami sea-surface elevation fields taken at various times since the source initiation, as well as fields of tsunami travel times.
- **Local tsunami hazard scenarios.** Local maps focus on smaller scales in the target area and depict fields of various parameters including the maximum seawater elevation and speed, the line of maximum sea water ingression and regression. They are related to individual scenarios.
- **Aggregated scenarios (local maps).** Local maps for an aggregated scenario represent the synthesis of all the results calculated (or observed) for each potential tsunami scenario concerning the same target location, with extraction of extreme intensities of all scenarios for various parameters (principally sea water elevation, water particle speed, flow depth, receding extension).

Sea Level Rise Hazard

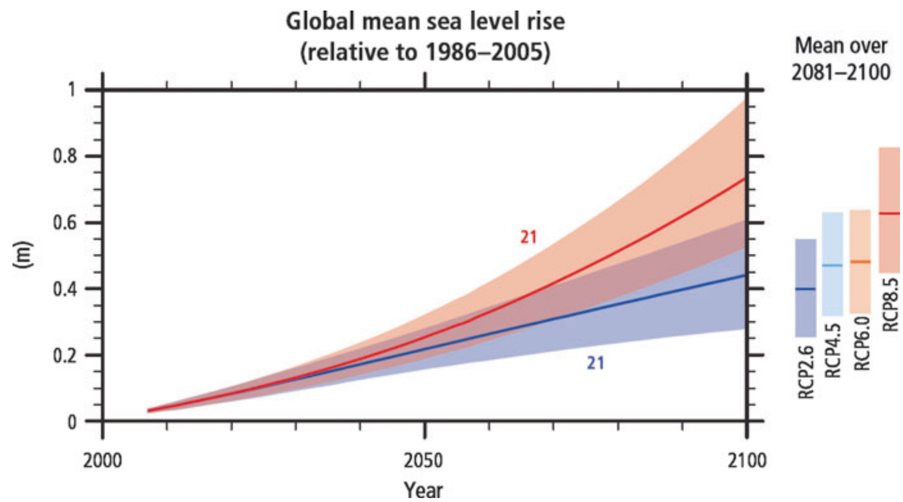
Sea level change is a process that has occurred naturally throughout the history of the Earth. Over the last century, the 15 cm increase in global mean sea level was small enough to have had little impact on humans, but steady enough to make it probably the most reliably documented of any climate-related trends (Hawkes 2013). Sea level rise is one of the main outcomes of global warming. The causes of global sea level rise can be placed into three categories:

1. Thermal expansion of sea water as it warms up
2. Melting of land ice
3. Changes in the amount of water stored on land

Other factors, from local sinking of land to changing regional ocean currents, also can play a role in relative sea level rise. These influences are contributing to “hot spots” that are facing higher-than-average local sea level rise, such as the Po river plain in Northern Italy (Venice).

Over the period 1901–2010, global mean sea level rose by 0.19 [0.17–0.21] m. The rate of sea level rise since the mid-nineteenth century has been larger than the mean rate during the previous two millennia (IPCC 2014). Sea level rise is described in h/t and measured from instrumental gauges, archeological evidence, and geological data such as submerged speleothems (Antonioli et al. 2004). Data must be obtained in tectonically stable areas, to avoid uncorrected information.

There has been significant improvement in the understanding and projection of sea level change. Global mean sea level



Hazard Assessment, Fig. 6 Projection of sea level change in XXI c. (IPCC 2014)

rise will continue during the twenty-first century, *very likely* at a faster rate than observed from 1971 to 2010. For the period 2081–2100 relative to 1986–2005, the rise will *likely* be in the range of 0.26–0.55 m or in the range 0.45–0.82 m according to the adopted scenario (*medium confidence*) (Fig. 6). Sea level rise will not be uniform across regions. By the end of the twenty-first century, it is *very likely* that sea level will rise in more than about 95% of the ocean area. About 70% of the coastlines worldwide are projected to experience a sea level change within 20% of the global mean (IPCC 2014).

Available hazard maps, at small scale, are compiled taking into account the rising of sea level in a defined time-window and the local topography. At a more local scale, also other geological factors, such as glacial isostasy adjustment or subsidence, must be considered in the hazard map. As an example, in Louisiana (US), a subsidence of about 0.6 m from today must be added to an average predicted sea level rise of about 0.5 m in the year 2100.

Subsidence Hazard

Subsidence is mainly the vertical downward displacement of the Earth's surface generally due to insufficient support from beneath, a superimposed load, or a combination of both. It can arise from natural causes, human activities, or, often, by human activities destabilizing natural systems (Galloway 2013). Subsidence results from a wide variety of circumstances and processes, is geographically widespread, and is associated with natural (geologic) or anthropogenic origin. Geologic causes are related to endogenous and exogenous processes, such as Earth crust modification, vertical displacement of the ground due to fault activation by earthquakes, isostasy and diagenesis of unconsolidated soil. Anthropogenic subsidence is connected to withdrawal of fluids or gas from the ground, compression of weak and/or water-logged soils under superimposed loads, because of vibrations, or withdrawal of support, collapse of underground cavities, both natural and manmade.

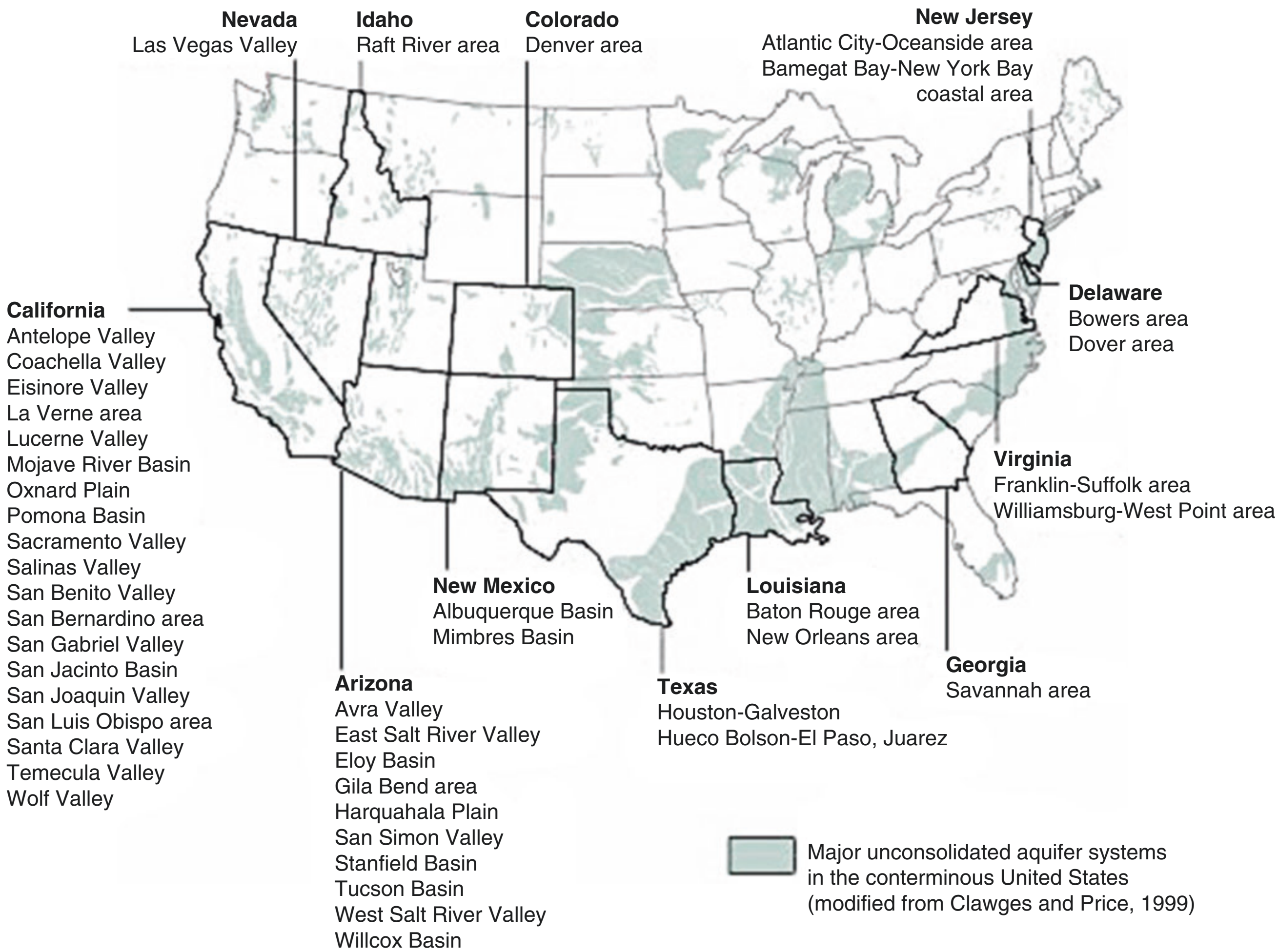
Noteworthy is hydrocompaction, or geotechnical subsidence, generally occurring at a local scale when a superimposed load is applied to an unconsolidated soil. A process of collapse and compaction occurs in silty to sandy sediment (soil) having a low bulk density, when the water is removed or overlaid, after being saturated for sustained periods. In extreme conditions with collapsible soils, subsidence can promote further collapse by sapping and piping in the subsurface. Indicators for the phenomenon can be referred to as potential susceptible areas (Fig. 7) or, mainly, to the velocity of lowering in terms of length/time (e.g., mm/year as in Fig. 8).

Sinkhole Hazard

The sudden and sometimes catastrophic subsidence associated with localized collapse of subsurface cavities is generally defined as a sinkhole. It is a closed depression generated by karstification that occurs naturally on the surface of the ground. Sinkholes are usually circular or subcircular, and range in size from one to several hundred meters in diameter and up to several tens of meters in depth (Soriano 2013). A classification of sinkholes was developed by Williams (2003) and Waltham et al. (2005). Available maps mainly refer to their distribution (Fig. 9) or potentially susceptible land.

Soil Erosion Hazard

Soil erosion is the detachment and movement of soil particles by the erosive forces of wind or water. The erosion of soil is a naturally occurring process on all land but, sometimes, it may occur at a very high rate. The latter may be reflected by the reduced crop production of farmlands, poorer quality of surface water, and disruption of drainage networks. Hazard can be expressed in predicted soil loss in ton/ha/year (Fig. 10) or vulnerability to a specific agent, such as wind.



Hazard Assessment, Fig. 7 Susceptible areas to land subsidence (USGS 2000)

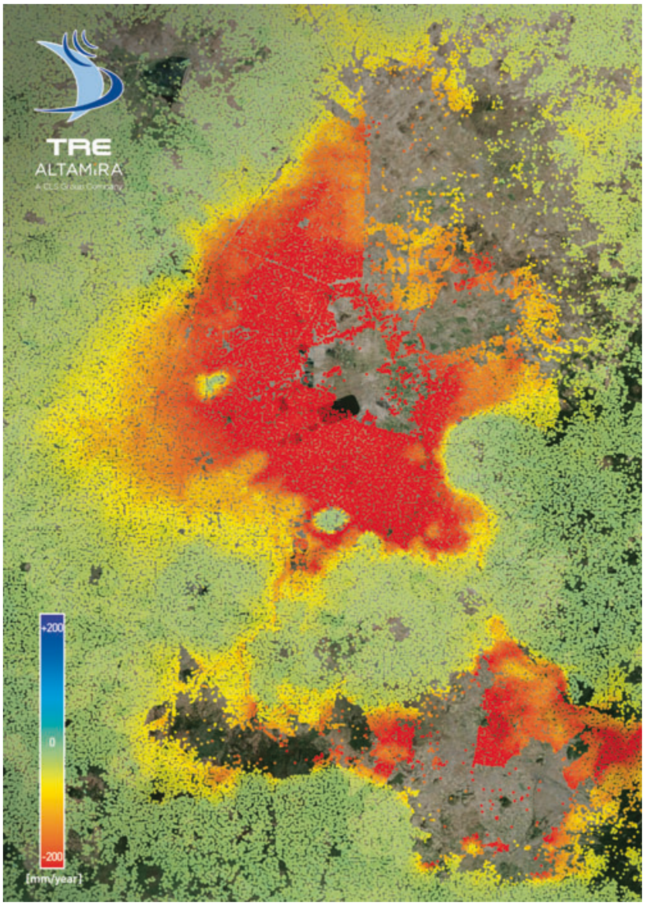
Heat Wave Hazard

Modern society is faced with the task to assess and evaluate the potential risks and probability of occurrence of all hazards including heat wave hazard. Impacts and mitigation often require the contribution of engineering geology, especially when there is the need for a tailored risk management strategy taking into account structural (e.g., heat-resistant construction) and/or nonstructural mitigation and prevention measures (e.g., risk-sensitive spatial planning) as a part of general land use planning and management.

Heat wave is a period of abnormally hot weather. Heat waves and warm spells have various and in some cases overlapping definitions (IPCC 2012). According to many investigations on climate change, there is high confidence that the future will exhibit much warmer temperature with respect to the present. Models project substantial warming in temperature extremes by the end of the twenty-first century. It is virtually certain that increases in the frequency and magnitude of warm daily temperature extremes and decreases in cold extremes will occur in the twenty-first century at the global scale. It is very likely that the length, frequency, and/or intensity of warm spells or heat waves will increase over most land areas (IPCC 2012). Hazard can be expressed in terms of projected annual changes in dryness, assessed in terms of change in annual maximum number of consecutive dry days (or days above a given temperature) and changes in soil moisture (IPCC 2012).

Drought Hazard

Drought may be described as a chronic, potential natural hazard characterized by prolonged and abnormal water shortage. According to the IPCC (2012), drought is a period of abnormally dry weather occurring long enough to cause a serious hydrological imbalance. Drought is a relative term; therefore, any discussion in terms of precipitation deficit must refer to the particular precipitation-related activity that is under discussion. For example, shortage of precipitation during the growing season impinges on crop production or ecosystem function in general (due to soil moisture drought, also termed agricultural drought), and during the runoff and percolation season primarily affects water supplies (hydrological drought).



Hazard Assessment, Fig. 8 Land subsidence intensity in the town of Mexico City in the period 07/03/2003–12/10/2007, from ENVISAT Satellite (Courtesy of http://tre-altamira.com/geo-hazards/)

Storage changes in soil moisture and groundwater are also affected by increases in actual evapotranspiration in addition to reduction in precipitation. A period with an abnormal precipitation deficit is defined as a meteorological drought. A megadrought is a very lengthy and pervasive drought, lasting much longer than normal, usually a decade or more (IPCC 2012).

There is medium confidence that droughts will intensify in the twenty-first century in some seasons and areas, due to reduced precipitation and/or increased evapotranspiration. This applies to regions including southern Europe and the Mediterranean region, central Europe, central North America, Central America and Mexico, northeast Brazil, and southern Africa (IPCC 2012).

During the last few decades, a large variety of drought quantification and monitoring models have been developed. Su et al. (2003) summarized these methods into meteorological based indices (e.g., the standardized precipitation index), process-based indices (e.g., evaporative fraction, EF), and satellite-based indices (e.g., vegetation indices). Some of them are derived from climate factors and less relative to surface water characteristics and crop conditions whereas some only consider single surface factors like soil moisture content neglecting plant water demand, completely different results may be achieved from the same input parameters.

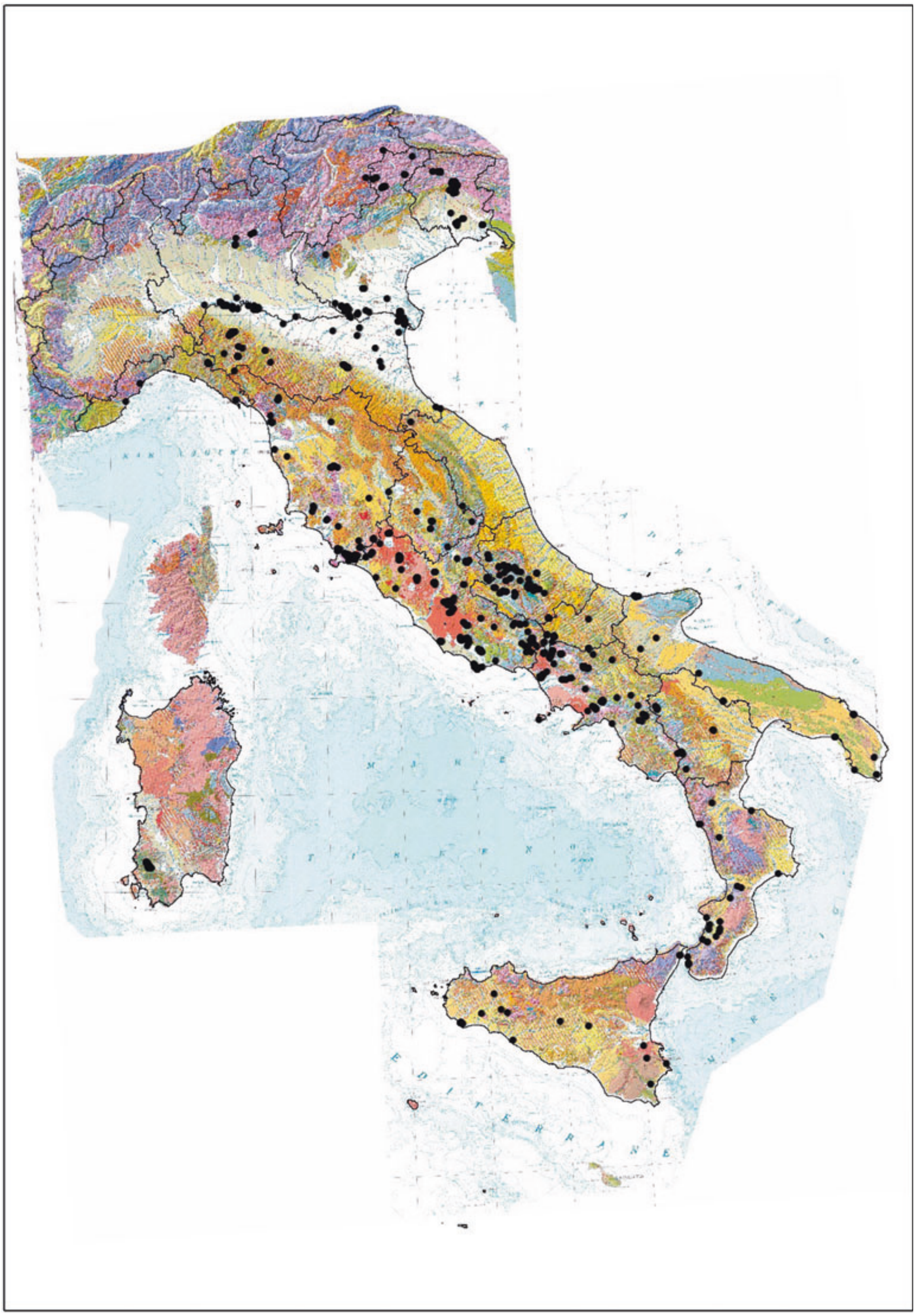
Multiple Hazard Assessment

According to the above state of art review, it is quite evident that every hazard has a proper specificity. The current state of art does not readily permit one to integrate such a variety of parameters and procedures into a single effort. When this is done the adopted approach is usually not sufficiently rigorous and results can be used only as an approximate reference for decisions. Rigorous multihazard maps are very difficult to design, unless they deal with very simple qualitative small scale or general maps (e.g., natural hazard in a given country or continent, mostly based on inventory of natural disasters). Other approaches can consider the development of multirisk scenarios combining hazard parameters with vulnerability indices specifically designed for all hazards at stake (Armonia 2005b) as well as considering cascading effects (Garcia-Aristizabal and Marzocchi 2013). Multihazard and multirisk assessments are certainly key for both spatial and emergency planning in areas that are affected by multiple threats that either coexist or may be triggered by one another in a cascading sequence.

At the national/regional scale, it is possible, considering limits and constraints of production of hazard maps, to adopt a simplified approach useful to produce a set of single hazard maps that can be examined together in a multilayered hazard map (not aggregating hazards) by simply overlapping the single hazard maps using a GIS environment. This approach can be considered as appropriate also when no other vulnerability functions (empirical or theoretical) or damage matrices are available for risk analysis at local scales. The table of intensity scales, expressed as parametric values grouped into three qualitative classes, is shown in Table 6. This approach can be used when detailed intensity parameters are not available in hazard maps, at any scale of analysis.

Cascade Effects

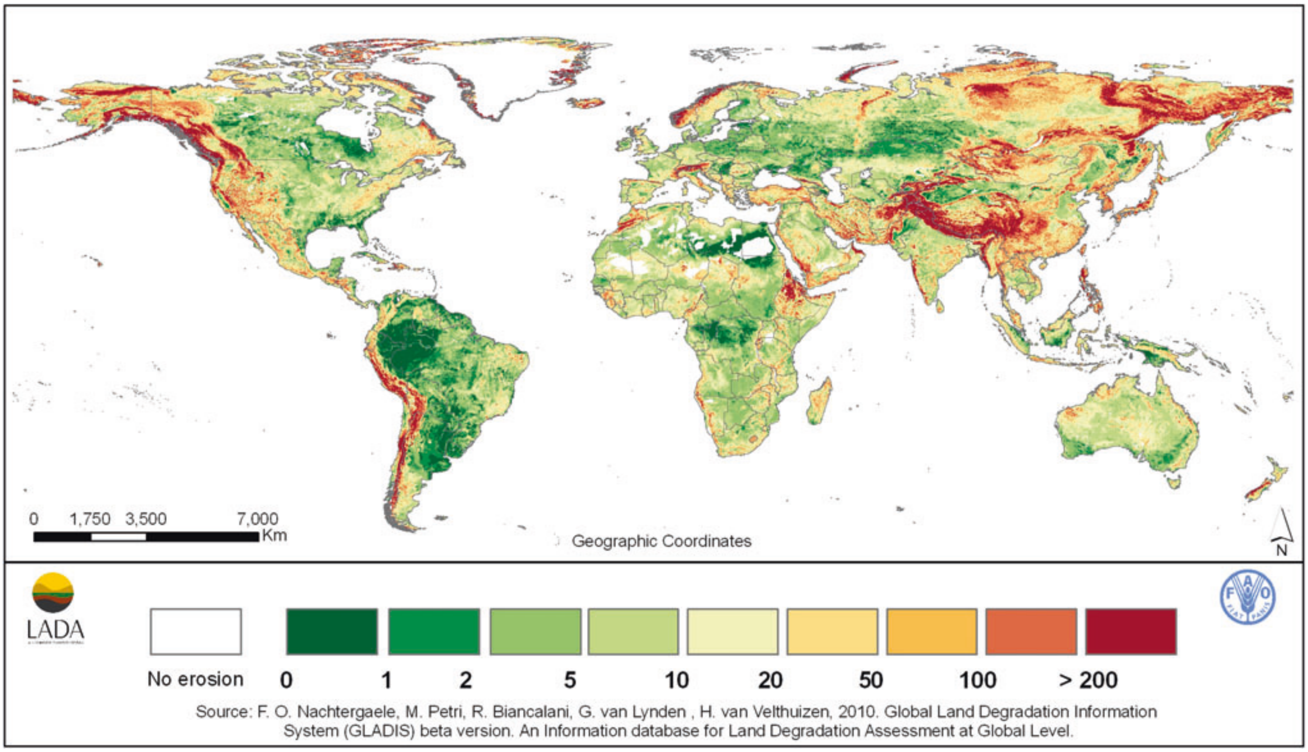
Different forms of "natural" hazard can interact through domino reactions and can be triggered by the same environmental event, for instance, landslides and floods as a result of heavy rainfall. For these reasons and also so that an integrated and holistic view of hazard potential can be achieved, a "multihazard perspective" that recognizes the range of hazards that can affect any one place needs to adopted. Figure 11 shows the interaction among different phenomena and the potential cascade effect (Garcia-Aristizabal and Marzocchi 2013).



Hazard Assessment, Fig. 9 Location of sinkhole reported in Italy until 2012 (Source: http://www.isprambiente.gov.it/it)

Climate Change and Changing Pattern of Hazard

A further driver has been the acceptance of the realities of climate change as a growing influence on patterns of current and future natural hazards. The Fourth Assessment Report published by the Intergovernmental Panel on Climate Change (IPCC 2007b) added empirical evidence of the observed impacts on the environment already caused by anthropogenic changes to the atmosphere, to the more theoretical projections in their three earlier reports. The warming trend of the climate system is now unequivocal and it is *very likely* (>90% confidence) that most of the *observed* increase

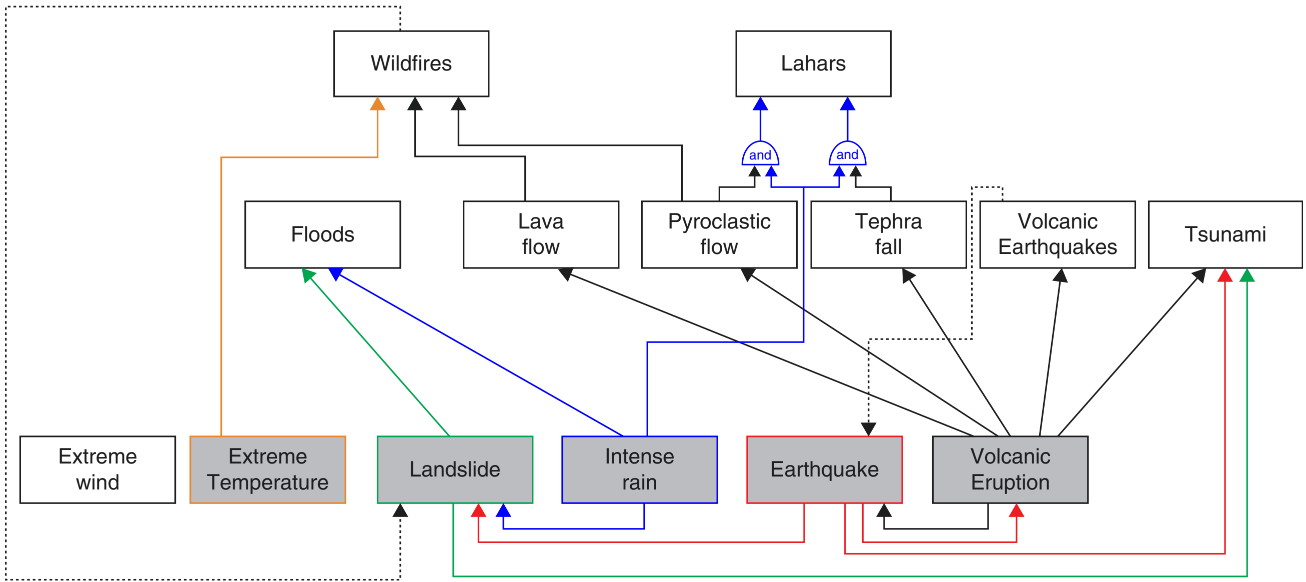


Hazard Assessment, Fig. 10 Predicted soil loss in ton/ha/year (USLE equation) (Source: http://www.fao.org/fileadmin/user_upload/soils/imgs/degradation_map/predic_soil_loss.jpg)

Hazard Assessment, Table 6 Simplified legend for national/regional scale in multihazard assessment (Armonia 2005b)

	Indicators at local scale (intensity)			
Natural hazard	Low	Medium	High	Parameters
Flood	<0.25	0.2–1.25	>1.25	Flood depth (m)
Forest fire	<350	350–1750	>1750–3500	Predicted fire-line intensity(*) (kW/m)
Forest fire	<1.2	1.2–2.5	>2.5–3.5	Approximate flame length (m)
Volcanoes	<5	5–10	>10	Intensity = Volcanic explosive index $\log_{10}$ (mass eruption rate, kg/s) + 3
Landslide (fast and slow movements)	<5 %	5–15 %	>15 %	Percentage of landslide surface (m^2, km^2, . . .) versus stable surface
Seismicity	<10 % g	10–30 % g	>30 % g	Peak ground horizontal acceleration (%g)

in temperature is due to the *observed* increase in anthropogenic greenhouse gas emissions (GHG). Even if anthropogenic GHG emissions were to cease tomorrow due to lags in the ocean/atmospheric system, the effects of the GHGs already emitted would continue to increase through the next century. This short-to-medium term inevitability of impacts makes explicit that there is a requirement for adaptation measures to be undertaken as part of any development strategy. However, it also reinforces the need for significant action to be taken to reduce emissions substantially if large *additional* positive feedback to the warming is to be avoided. According to some, these significant reductions need to be underway within the next 10 years (Hansen et al. 2007). Predicted climate change related hazards include more frequent and severe droughts, floods, and storms in addition to a large array of human health hazards and complex biological impacts on the productivity and stability of livelihoods that depend on natural resources (IPCC 2007a). Aside from the atmospheric effects of rising temperatures the increasing heat is also projected to contribute to a rise in global sea-level of between 0.18 m and 0.59 m this century (IPCC 2007b); however, extrapolation forward from the trend in rising global sea-level of the last decade has suggested that a rise of 1.4 m could be possible (Rahmstorf et al. 2007).



Hazard Assessment, Fig. 11 Interaction among different natural phenomena and the potential cascade effect. Climate change is further increasing the potential cascade effects, as in case of drought, coupled with extreme heat and low humidity, that can increase the risk of wildfire (Garcia-Aristizabal and Marzocchi 2013)

Integer Effect

A changing climate (IPCC 2012) leads to changes in the frequency, intensity, spatial extent, duration, and timing of extreme weather and climate events, and can result in unprecedented extreme weather and climate events and hazard. Changes in extremes can be linked to changes in the mean, variance, or shape of probability distributions, or all of these. Some climate extremes (e.g., droughts) may be the result of an accumulation of weather or climate events that are not extreme when considered independently. Many extreme weather and climate events continue to be the result of natural climate variability. Natural variability will be an important factor in assessing and shaping future extreme hazards, in addition to the effect of anthropogenic changes in climate.

Bridging Between Climate Change and Natural Hazard Studies

Investigating the relationship between climate change and natural disasters is a challenging issue since differences derive from intrinsic elements:

- Different time of occurrence; climate is an "average weather" where disasters are strictly derived from local extreme weather conditions (depending on disaster type), occurring suddenly in a very short time window.
- Different spatial domain; climate is global process whereas disasters, very often (depending on disaster type) involve quite local/district impact and external factors due to scientific gaps.
- Limitation of available data for clearly understanding the relationship between natural hazard and climate variability.
- Limitation of climate modelling in describing future occurrence and impact of natural hazards.
- Different schools of thought; a shared language and shared concepts are still missing, even inside a given scientific community. Osmosis through disaster science and climate science is still very week.

This dichotomy is also well debated in international literature which refers to two different approaches: disaster risk reduction (JRC/ISDR 2004) and adaptation to climate change. In practice (IPCC 2007b), there has been a disconnect between disaster risk reduction and adaptation to climate change, reflecting different institutional structures and lack of awareness of linkages. Disaster risk reduction, for example, is often the responsibility of civil defense agencies, whereas climate-change adaptation is often covered by environmental or energy departments. The first tends to focus on sudden and short-lived disasters, such as floods, storms, earthquakes, and volcanic eruptions, and has tended to place less emphasis on "creeping onset" disasters such as droughts. Furthermore, many natural hazards are not climate or weather related. Nevertheless, there is an increasing recognition of the linkages between natural risk mitigation and adaptation to climate change, since climate change alters not only the physical hazard but also the potential impact.

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Science of natural disaster risk has been endorsed since the 1950s, with unification of disaster-related definitions in the 1970s (UNESCO 1972; UNDRO 1980); scientific basis for adaptation plans, for instance, initiatives and measures to reduce the vulnerability of natural and human systems against actual or expected climate change effects, has been ratified by the IPCC (2001). The resulting situation is often perceived as the proverbial "Babylonian Confusion" also within the same community (Thywissen 2006). The two scientific communities started to discuss this issue together only recently.

For the first time, IPCC (2012) focused on interaction between extreme weather and climate events, with exposed and vulnerable human and natural systems. Thus, some limitations still exist: confidence in projecting changes in the direction and magnitude of climate extremes depends on many factors, including the type of extreme event, the region and season, the amount and quality of observational data, the level of understanding of the underlying processes, and the reliability of the simulation in models.

The main outcome of IPCC (2012), at a global scale, can be synthesized as follows:

- Models project substantial warming in temperature extremes by the end of the twenty-first century.
- It is likely that the frequency of heavy precipitation or the proportion of total rainfall from heavy falls will increase in the twenty-first century over many areas of the globe.
- Average tropical cyclone maximum wind speed is likely to increase, although increases may not occur in all ocean basins. It is likely that the global frequency of tropical cyclones will either decrease or remain essentially unchanged.
- There is medium confidence that there will be a reduction in the number of extratropical cyclones averaged over each hemisphere.
- There is medium confidence that droughts will intensify in the twenty-first century in some seasons and areas, due to reduced precipitation and/or increased evapotranspiration.
- Projected precipitation and temperature changes imply possible changes in floods, although overall there is low confidence in projections of changes in fluvial floods.
- It is very likely that mean sea level rise will contribute to upward trends in extreme coastal high water levels in the future.
- There is high confidence that changes in heat waves, glacial retreat, and/or permafrost degradation will affect high mountain phenomena such as slope instabilities, mass movements, and glacial lake outburst floods.
- There is low confidence in projections of changes in large-scale patterns of natural climate variability.

In practical terms, the above issues will affect the hazard assessment as it was implemented up to now, introducing a nonstationarity of results, function of forecasting time.

Conclusions

Hazard assessment is a fundamental step in the more comprehensive risk assessment chain. The latter can be defined as the combination of the probability (or frequency) of occurrence of a natural hazard and the extent of the consequences of the impacts. A risk is a function of the exposure and the perception of potential impacts as perceived by a community or system. Hazard assessment is the procedure to characterize and to map the location, magnitude, intensity, geometry, and frequency or probability of occurrence, and other characteristics of a given threat, event, phenomenon, process, situation, or activity that may potentially be harmful to the affected population and damaging the society and the environment. Hazard assessment is generally represented by an indicator, such as the intensity/magnitude of a natural phenomenon or human activities and the probability of occurrence.

There are a variety of definitions of "hazard," even if all of them deal with the attempt to characterize, in discrete categories, the different return period of energy (or magnitude) and impact's spatial distribution of natural phenomena and human activities. This definition, when applied in practice, encompasses a broad range of both natural phenomena and human activities which have the potential to cause damage and disruption, with territories being exposed to such hazards in an extremely heterogeneous fashion. Phenomenon can be either of slow (e.g., drought) or of rapid (e.g., earthquake) onset and have the potential to have effects which can impact across a range of scales. More in detail, any phenomenon has its own "size" in terms of involved area (see for instance landslide vs. earthquake), physical processes and availability of long-term data. The result is a variety of indicators and approaches to hazard assessment, each one depending on the input phenomenon. In conclusion, hazard assessment is phenomenon dependent.

Every parameter, and then its measurement, has inherent uncertainty also sometimes related to the ambiguity in the definition: this is known as definitional uncertainty. In the case of hazard assessment (Nadim 2013), one of the reasons for the ambiguity in the definition of hazard is that the term is used both to describe the temporal probability of occurrence of the event or the situation in question. There is another definitional issue that is dependent on the science school. In the American literature, often "hazard assessment or hazard management" actually refers to what in Europe and other regions of the world is referred to as "risk assessment or risk management."

Methods of hazard assessment include different approaches including deterministic, probabilistic, scenarios, indicators, process modeling, time-dependent, and time-independent. In all cases, they fall under two main categories: forecast (or projection) and prediction. Specific methodologies and spatial resolutions have been identified in hazard assessment, suggesting three different strategic approaches, corresponding to four different scales: the regional strategic approach (scale >50,000); the local general approach which exhibit two stages such as the general or preparatory land-use plan (scale 1:5,000–1:50,000) and the second detailed land-use plan (scale 1:500–1:5,000); and the site engineering approach (scale 1:100–1:1000). Different hazards are also interacting with each other, magnifying damage or generating secondary effects not expected in advance (domino effect).

Finally, climate change may generate an integer effect of damage, due to changes in the frequency, intensity, spatial extent, and duration, of extreme weather and climate events. The result is in unprecedented extreme weather and climate event and hazard. Climate change may also affect the temporal variability of hazard, making the temporal prediction not stable in time, but generating the clustering of damaging events in the near future.

In conclusion, hazard assessment is still a sectorial analysis, function of the different natural and human phenomenon, making the use from stakeholders rather complex. The consequence is that hazard analysis has difficulties entering in land use planning and management, sometimes approving solutions that are positive for the given hazards but negative for others. Similarly for the inclusion of climate change into ordinary practices, especially when implemented by local professionals who cannot have the same sensibility of scientist, can generate underestimation of hazards conditions in areas that will be developed and urbanized.

Cross-References

▶ Avalanche
▶ Catchment
▶ Climate Change
▶ Compaction
▶ Diagenesis
▶ Earthquake
▶ Earthquake Intensity
▶ Earthquake Magnitude
▶ Erosion
▶ Floods
▶ Hazard
▶ International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)
▶ Land Use
▶ Landslide
▶ Liquefaction
▶ Mass Movement
▶ Modelling
▶ Monitoring
▶ Probability
▶ Risk Assessment
▶ Risk Mapping
▶ Sea Level
▶ Sinkholes
▶ Subsidence
▶ Volcanic Environments

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Hazard Mapping

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Synonyms

Peril mapping; Risk mapping; Threat mapping

Definition

Hazard mapping is a process of preparing information for display using graphical representation of the distribution of attributes of features or conditions that have the potential to cause injury to people or animals or damage to property or the environment. Hazard features may be indications of past occurrences of a hazardous process, or future occurrences of a hazard may be interpreted from landscape features or subsurface data that indicate susceptibility to the process. Hazard maps can be made for land-use planning and development zoning or for actuarial or engineering purposes. Hazard maps for planners depict areas where studies must be performed by qualified professionals to support land development permit applications that may depend on mitigation of the hazard, including land use or design details. Hazard maps for actuaries and engineers depict hazardous-process intensity values that are associated with a specific probability of occurrence that support damage and loss estimates or design of new construction or rehabilitation of existing structures. Hazard mapping for actuaries and engineers is performed primarily for the insurance industry, disaster-response planning, and building code development, improvement, and compliance.

Some individuals use "hazard" to refer to an event that occurs where people can be injured or property can be damaged (White 1974); whereas others view the same event as an extreme natural event. Insurance actuaries in North America use "peril" to refer to the event that causes loss, such as a fire or a vehicle crash, and "hazard" to refer to features or conditions that would make a loss worse than if the feature or condition had not been present, such as flammable liquids stored improperly in a building or improperly inflated tires on a vehicle. Risk involves exposure of people to injury or elements of value to damage or loss caused by the occurrence of a hazardous process. Hazard is expressed in terms of both intensity or magnitude and frequency or probability of occurrence; risk is expressed in terms of fatalities, injuries, dollars, or disruption of services or activities. Generally, risk is the product of the hazard probability at or exceeding a specific intensity and the value of what is exposed to potential loss (injury or damage) if the hazard were to occur.

Introduction

Natural hazards, called perils by IRDR (2014), can be subdivided into six main types or families, which are further subdivided into subtypes or main events (Table 1). Many of

Hazard Mapping, Table 1 Classification of perils at the family, main event, and individual peril levels (Modified from IRDR (2014)). Note that the association of perils with main events is not necessarily unique. Some perils may be associated with multiple main events, such as landslide triggered by earthquake, coastal or river erosion, or rain storm

Family	Main event	Peril (selected)	
Geophysical	Earthquake (EQ) Tectonic deformation Volcanic activity	Ash fall/lahar Fault rupture Gas (SO_2) emission Ground vibration	Landslide after EQ Lava/pyroclastic flow Liquefaction Tsunami
Hydrological	Flood Mass movement Wave action	Avalanche (snow) Coastal/river erosion Debris/mud flow Expansion/shrinkage of soil	Flash flood Ice jam flood Landslide after erosion Riverine flood Sinkhole
Meteorological	Convective storm Extratropical storm Extreme temperature Fog Tropical cyclone	Cold/heat wave Derecho wind Frost/freeze/hail Lightning Landslide after storm	Sandstorm/dust storm Snow storm/ice storm Storm surge Temperature inversion Tornado
Climatological	Drought Glacial lake outburst Wildfire	Brush/forest fire Enhanced soil erosion Grassland fire	Subsidence induced by groundwater decline Sea level rise
Biological	Animal incident Disease Insect infestation	Bacterial disease Fungal disease Parasitic disease	Prion disease Viral disease Skin irritation
Extraterrestrial	Impact Space weather	Airburst/shockwave Geomagnetic storm	Telecommunication disruption

these potentially hazardous processes are of interest to engineering geologists who may be responsible for hazard mapping; however, some are outside the scope of engineering geology, such as those triggered by biological, climatological, extraterrestrial, and meteorological events. In general, engineering geologists use the term hazard to mean two types of mappable conditions in different contexts: Type 1) a process or event which could cause some level of injury or damage if it occurred, Type 2a) the probability that a process or event of damaging intensity might occur at a specific location within a specific time interval, or Type 2b) the intensity of a process or event at a specific location that corresponds to a designated exceedance probability within a specific time interval.

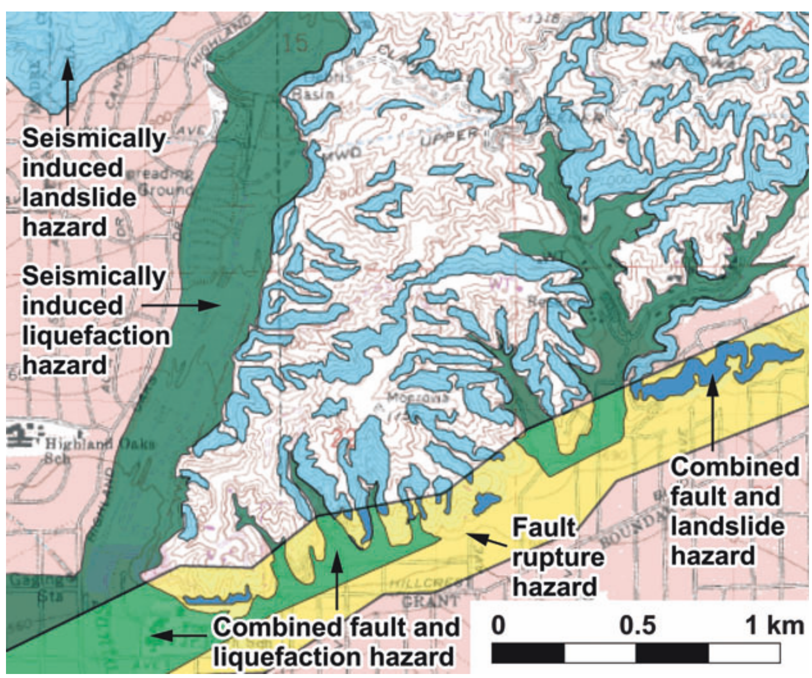
The term "mapping" refers to a process. Approaches to hazard mapping can be purely observational or purely analytical; however, hybrid approaches are common. For example, mapping the hazard associated with rock fall most likely would begin with a map of the distribution of rock blocks on a landscape. Depending on the age of the landscape, or confidence the geologist has in being able to estimate the age of the landscape, a physics-based rock-fall model might be used to forecast how far from a cliff rocks of different sizes and shapes might roll, or what kinetic energy they might have along their paths of travel.

Hazard Mapping for Planning and Zoning Purposes

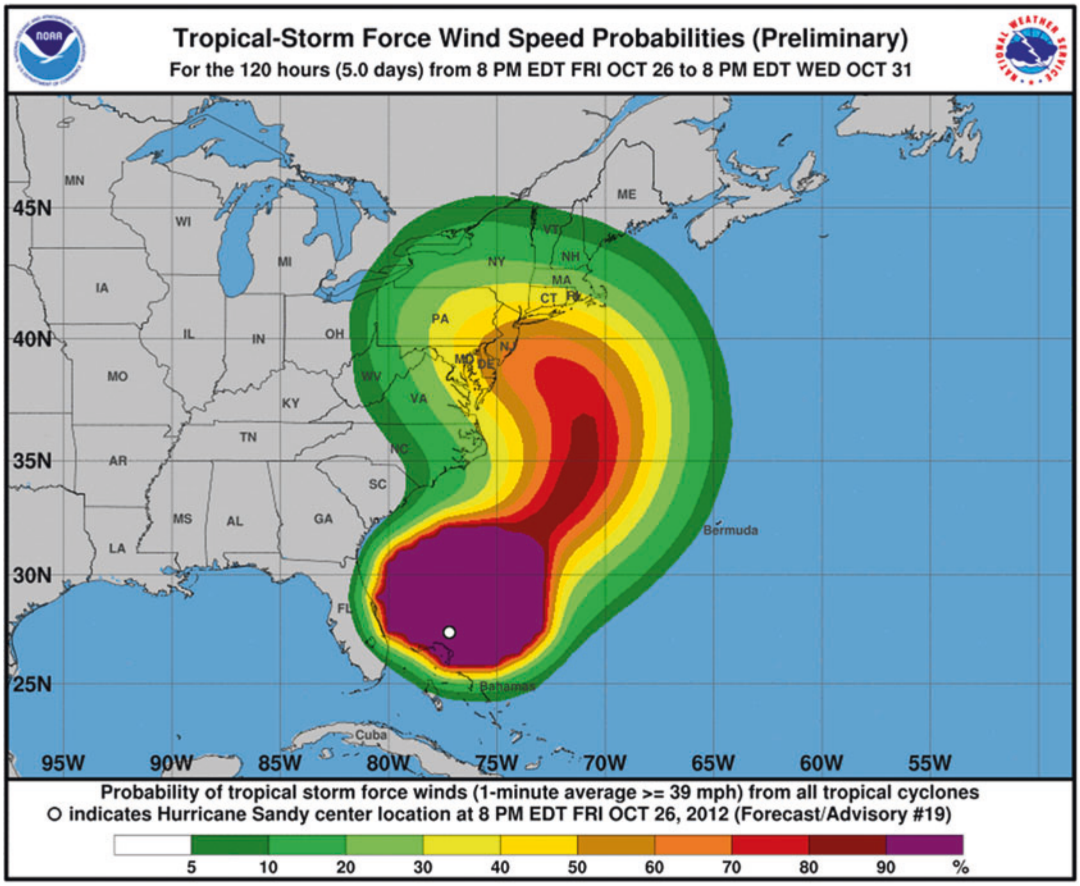
Hazard mapping of Type 1 would consist of locating and plotting the positions of one or more attributes of potentially dangerous features or the effects produced by potentially dangerous processes. Examples of Type 1 hazard maps include the locations of seismically active fault traces, the distribution of potentially liquefiable sediment deposits, the distribution of topographic features and deposits of past landslides, and areas considered to be susceptible to future slope movements (Fig. 1). Hazard maps of Type 1 can be made by direct observation of geologic features, such as is the case with sinkholes and soluble rock formations, or they may require some measurement and calculation, as would be the case with potentially liquefiable sediment deposits, which require knowledge of grain-size distribution, unit weight, and depth to groundwater, or inference of these qualities from geologic and topographic settings, such as younger sand-dominated alluvial deposits in stream channels with relatively low gradients or possibly near stable lakes or marine environments.

Hazard Mapping for Actuarial and Engineering Purposes

Hazard mapping of Type 2 requires calculations based on in-depth knowledge of a potentially hazardous process and of elements that might be subjected to the process. In-depth knowledge of a process typically is considered to be an analytical or numerical model that requires science-based inputs and which produces quantitative outputs. Examples of Type 2a hazard maps would be for specific buildings or facilities, or for types of buildings or facilities for which the damage thresholds or performance reliability information (known as fragility curves) has been developed, such as some types of power plants. An example of a Type 2a hazard



Hazard Mapping, Fig. 1 Earthquake hazard zones regulated by the State of California in a portion of the Mount Wilson 7.5-min quadrangle. Earthquake fault zones require investigation by a licensed professional geologist to ensure that buildings for public occupancy avoid being located on active fault traces. Liquefaction and landslide hazard zones require investigation by a licensed professional to comply with requirements for mitigation of permanent ground displacements (Modified from CGS (2017))



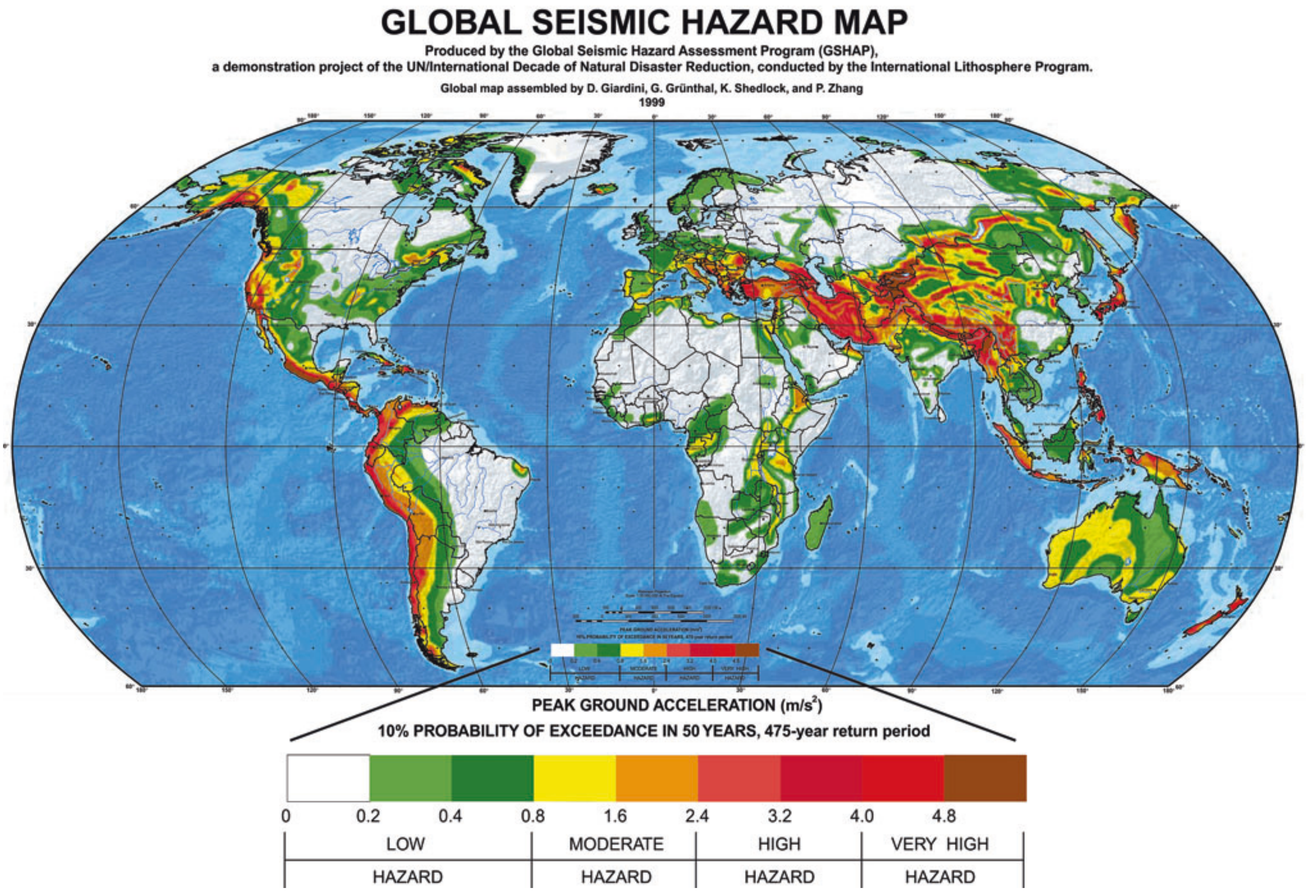
Hazard Mapping, Fig. 2 Exceedance probabilities of tropical-storm-force winds related to Hurricane Sandy for a 120-h (5.0-day) period beginning at 8 PM Eastern Daylight Time on Friday, 26 October 2012. Tropical storm force winds are defined as having 1-min average speeds equal to or greater than 34 knots (63 km/h) (From NOAA (2012), http://www.nhc.noaa.gov/aboutnhcgraphics.shtml accessed 23 November 2017)

map would be a map of exceedance probabilities during a specific time interval for a specific hazard intensity that commonly causes damage to buildings or disruption to communities, such as the probability of wind speed equaling or exceeding a specified value during a tropical cyclone condition (Fig. 2). An exceedance probability is the probability that an event condition of a specified intensity or higher will be experienced during a specified exposure time.

Hazard maps of Type 2b are based on in-depth knowledge of a potentially hazardous process (an analytical or numerical model) to produce values of process intensity for a specified probability of exceedance and time interval; however, the damage thresholds of buildings or facilities exposed to the hazardous process are not included in the map. An example of a Type 2b hazard map would be the Global Seismic Hazard Assessment Program (GSHAP) map (Fig. 3). The GSHAP map was a collaborative international effort to produce a uniform hazard map of earthquake ground motion for the world north of 60° South Latitude and above sea level (i.e., land). The uniform hazard selected for the GSHAP project was the ground motion expressed as the peak ground acceleration in m/s^2 associated with an annual frequency, AF, of 0.00210721 events per year, which is calculated with Poisson statistics (Eq. 1) with a time interval, t, of 50 years and an exceedance probability, p, of 0.10. The seemingly excessive number of decimal places is needed to produce more familiar numbers for a related result.

$$t = 50 \text{ year}; \quad p = 0.10; \quad AF = \frac{-\ln(1-p)}{t};$$
$$AF = 0.00210721t = 1 \text{ year}; \quad p = 0.00210721;$$
$$AF = \frac{-\ln(1-p)}{t}; \quad AF = 0.00210943 \tag{1}$$

Poisson statistics requires each event to be independent of the previous events. A 1-year exceedance probability set equal to the annual frequency of the 50-year, 10% exceedance probability (Eq. 1) returns a 1-year annual frequency nearly equal to the annual exceedance probability, supporting the concept that the annual frequency and the annual exceedance probability are equivalent. Although it is desirable to avoid



Hazard Mapping, Fig. 3 The Global Seismic Hazard Map (Modified from Giardini et al. (1999))

implying periodicity of earthquakes, it is relatively common and convenient to refer to the inverse of the annual frequency (Eq. 2) as the average return period (RP):

$$\begin{aligned} RP &= \frac{1}{AF} = \frac{1}{0.00210721 \ \frac{\text{events}}{\text{year}}} \\ &= 474.56 \ \frac{\text{years}}{\text{event}} \ RP \cong \frac{1}{0.00211 \ \frac{\text{events}}{\text{year}}} \\ &= 473.93 \ \frac{\text{years}}{\text{event}} \cong 474 \ \frac{\text{years}}{\text{event}} \end{aligned} \tag{2}$$

The 50-year, 10% exceedance probability typically is reported as a return period of 475 years. In some cases, it is rounded to the nearest 100 years (i.e., 500 years per event). The details of the ground motion for the entire world are not readable in Fig. 3, but it is clear that the seismically more hazardous areas of the Earth are the Pacific Ring of Fire and a complicated zone extending from southeast Asia to southern Europe (the north-central Mediterranean Sea countries). An important aspect of this depiction of a geophysical hazard is that a value of earthquake ground motion hazard intensity associated with a constant annual frequency (i.e., a 50-year exceedance probability of 0.10) has been calculated for a regularly spaced grid of points in every country of the world.

Flooding is another recurring hazardous process that has been mapped regionally or nationally. Sampson et al. (2015) have developed a flood hazard model for all land surfaces between 56° South Latitude and 60° North Latitude. The model inputs consist of six datasets: rainfall, hydrography, satellite imagery, urbanization, elevation, and vegetation. The model output is at a resolution of 90 m because the elevation dataset is from the satellite radar topography mission (SRTM; https://www2.jpl.nasa.gov/srtm/). The flood hazard is shown as the maximum one-in-100 year water depths of 0.5, 1.5, 2.5, 3.5, and >5 m. In the United States, the Federal Emergency Management Agency (FEMA) is charged with administering the National Flood Insurance Policy which requires mapping of land that will be inundated by the flood that has a "one-in-100 chance of occurring, or an average return period of 100 years" (FEMA 2017). Therefore, floodplain mapping in the United States defines the elevation below which inundation is expected to occur with an annual frequency of 0.01. In the context of conventional 30-year-duration home mortgages, this annual frequency corresponds to a 30-year exceedance probability of about 27%. The definition of a one-in-100 chance flood is challenging with the realization that the past measurements of stream flow and precipitation at a point are affected by both upstream development that can transmit greater runoff and climate change that can produce storms of greater intensity that produce higher peak flows.

Additional Hazard Mapping Examples

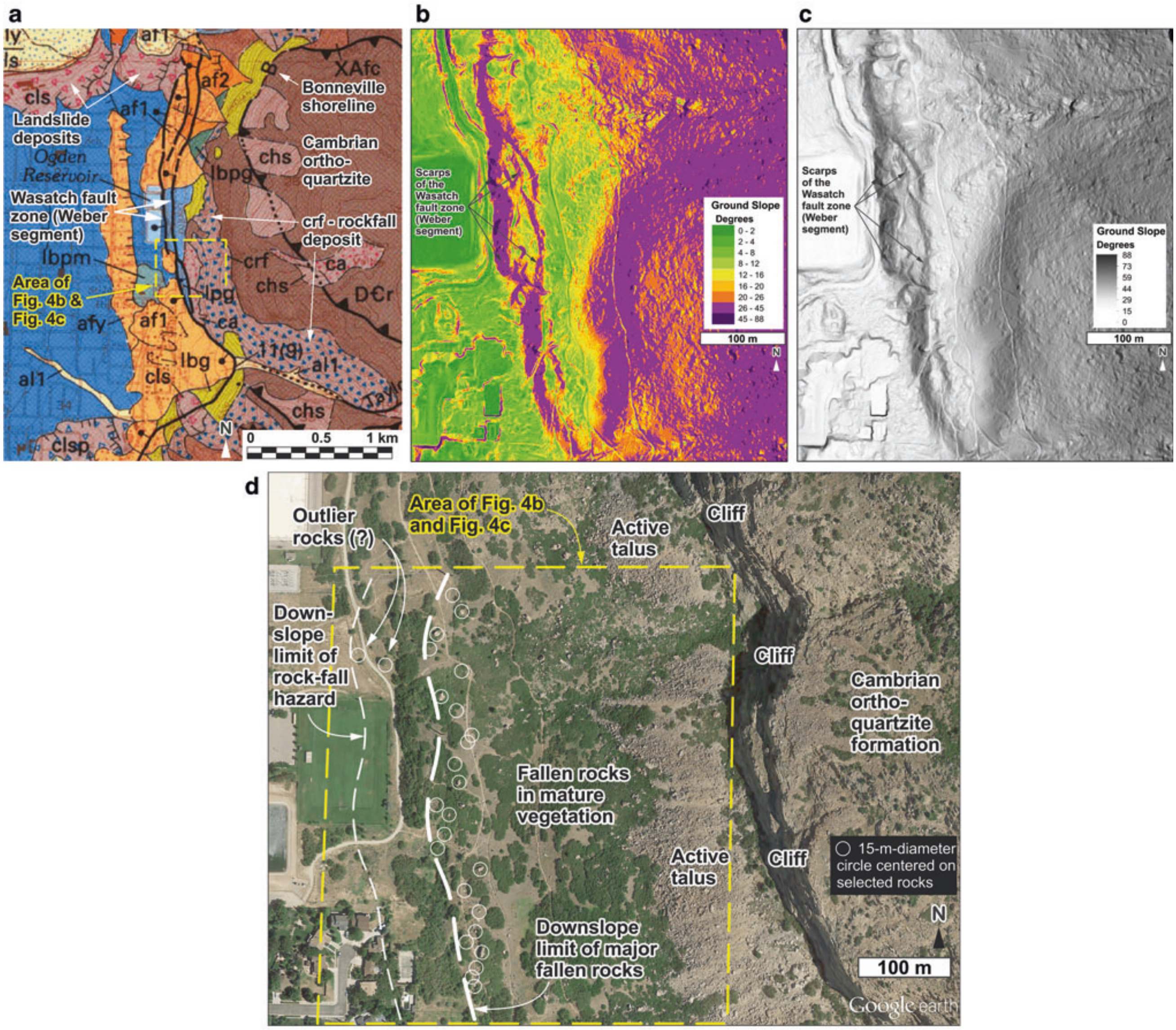
As an example, the steps involved in mapping rock fall hazard are described using information in Fig. 3a through d for a location on the west flank of the Wasatch Range in northern Utah, USA. A number of locations in Utah have experienced damaging rock falls (Castleton 2009), including some in the area covered by Fig. 3a. The primary focus of the geologic map in Fig. 3a was the Weber segment of the Wasatch fault zone (Nelson and Personius 1993), a major, seismically active normal fault that extends north approximately 390 km from central Utah into southern Idaho; the Weber segment is 61 km long. Traces of the Weber segment displace deposits of Lake Bonneville age and younger. The 15 ka Bonneville shoreline is mappable in places within Fig. 3a, and projects along its elevation contour across the rock-fall study area, which is designated by a rectangle outlined with dashed yellow lines. The rock formation exposed in the cliffs of the Wasatch Range on the east side of Fig. 3a is Cambrian ortho-quartzite. Rock-fall dominated colluvial deposits (crf in Fig. 3a) have accumulated on and below the Bonneville shoreline, indicating that they are younger than 15 ka. Analysis of scarp heights in geomorphic surfaces of different ages have been interpreted by Nelson and Personius (1993) to indicate at least 10 and perhaps 15 surface faulting earthquakes were generated on the Weber segment during the past 15 ky. Consequently, the Wasatch Range cliffs have been shaken violently many times since Lake Bonneville retreated from its highest shoreline, which undoubtedly triggered widespread rock falls from the local cliffs. Other rock falls must have been triggered by freeze-thaw cycles, strong storms, and other slope processes over the past 15 ky.

A 0.5-m resolution, lidar-based digital terrain model (DTM) of this area provided the basis for GIS calculations of ground slope (Fig. 4b and c). The steep scarp of the Weber segment of the Wasatch fault zone is clearly evident. Irregularities in the slope angle depiction in Fig. 4b east of the Weber segment fault scarps appear as orange or red pixels within the area of green and yellow pixels, and could represent rock blocks. However, the appearance of the same area in Fig. 4c suggests that the irregularities may be mature vegetation, which happens to be scrub oak and gamble oak that have dense networks of irregular branches. Even though the DTM was based on lidar data and is supposed to be a bare-earth dataset, the laser strikes may not have penetrated the oak brush to reach the ground surface. The ground slope data in both Fig. 4b and c are identical; the only difference is the selection of how slope in degrees is displayed. The inverse grayscale in Fig. 4c gives the appearance of a hillshade map, but without any harsh shadows produced by the GIS hillshade utility in Spatial Analyst.

A natural color aerial photograph taken on 8 July 2016 provides useful detail. Annotations in Fig. 4d show the cliff in Cambrian ortho-quartzite, which is the source of the rocks, and an apron of talus that is essentially free of visible vegetation, which is interpreted to be an active geomorphic feature that is accumulating most of the rocks that fall from the cliff. Large blocks of rock are visible through the vegetation between the toe of the active talus slope segment and the top of the steep fault scarp. The western-most rock blocks are marked by circles that are 15 m in diameter. Six of the rock blocks appear to have a long axis that is on the order of half of the circle diameter, indicating that some large blocks have traveled a substantial distance from the cliff. The heavy dashed white line marks the approximate downslope limit of the major rock blocks.

Two circled rocks are located west of the base of the Weber segment fault scarp in Fig. 4d; these are labeled “outlier rocks (?)” because they are isolated and not part of a trend of rocks, such as the trend marked by the heavy dashed line. The source of the two “outlier” rocks may have been the nearby fault scarp, and not the quartzite cliff. The “outlier” rocks may still represent a rock-fall hazard, but the energy that the rocks would have along their paths could be much lower if their source were the fault scarp, rather than the quartzite cliff.

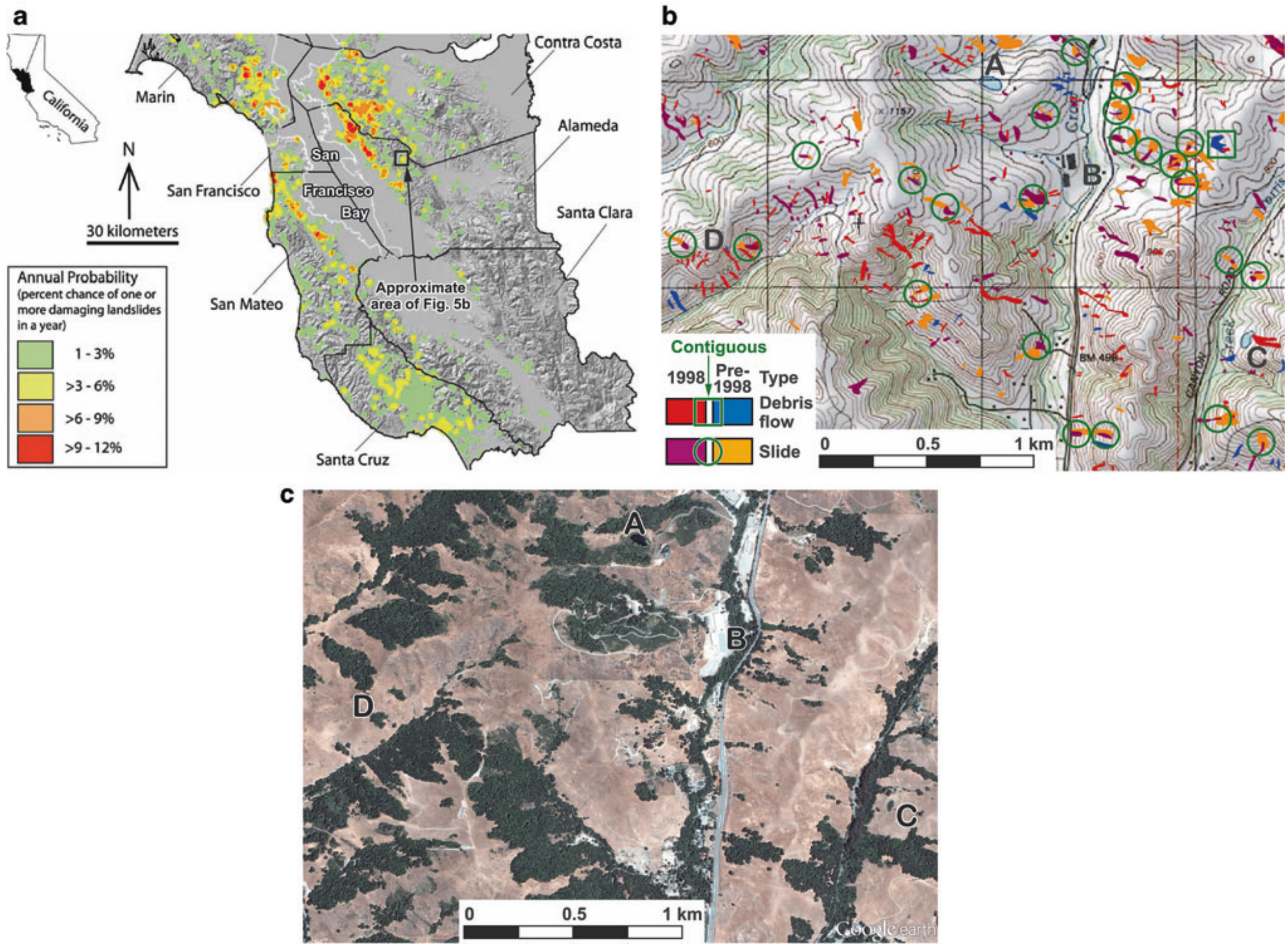
The rock-fall hazard in the area of Fig. 4b and c as depicted in Fig. 4d could be used by planners. A logical land-use planning approach might restrict development of most types east of the heavy dashed line. West of the heavy dashed line are the scarps of the Weber segment of the Wasatch fault zone, which would impose limitations on development that are independent of rock fall; the scarps are not mapped as specific hazards in this rock-fall hazard mapping example. The area between the fault scarps and the light dashed line might have a land-use restriction that requires rock-fall hazards to be evaluated by a qualified professional so that structures for human occupancy would be located or designed according to site-specific recommendations. Further analysis could be performed using runout distances from slope geometry similar to the study area. Alternatively, analytical or numerical models could be used to generate thousands of cases to develop statistical distributions of parameters such as maximum and average values of bounce height, velocity, and kinetic energy. Even though statistical methods may be used for some attributes of falling rocks, the results would not be probabilistic because they would be based on at least some deterministic aspects and not be expressed with a



Hazard Mapping, Fig. 4 (**a**) A small part of a published surficial geologic map along a portion of the Weber segment of the Wasatch fault zone by Nelson and Personius (1993) where rock-fall dominated colluvial deposits have been mapped. The location of (**b**) and (**c**) is indicated by the dashed yellow rectangle. (**b**) Ground slope in degrees colorized into nine intervals, calculated from a 0.5-m digital terrain model (DTM) using ESRI ArcGIS Spatial Analyst. DTM obtained from State of Utah Automated Geographic Reference Center (lidar tile 12TVL2200062000; https://gis.utah.gov/data/elevation-and-terrain/2013-2014-lidar/ accessed 22 November 2017). (**c**) Ground slope in degrees depicted with inverse continuous grayscale stretch, which gives a hillshade appearance without strong shadows and is known as "slope shade." Identical data described in title of (**b**). (**d**) Natural color aerial photograph dated 8 July 2016 obtained from Google Earth Pro annotated to show the position of (**b**) and (**c**), the cliff, areas of active talus accumulation, fallen rocks within mature vegetation, and selected rock blocks marked by circles that are 15 m in diameter. The heavy dashed line denotes the apparent downslope limit of major blocks of fallen rock; the light dashed line denotes the apparent downslope limit of rock-fall hazard

combination of annual frequency and some intensity parameter, such as kinetic energy or velocity.

The rock blocks that have come to rest near the top of the fault scarp are on a geomorphic surface that is younger than 15 ka. If the year in which each of the large rock blocks came to rest could be determined or estimated, for example, by cosmogenic isotope dating, then the age distribution might serve as a basis for estimating a frequency of occurrence of rock falls large enough to result in the blocks coming to rest where they are encircled in Fig. 4d. The close proximity of this example area to the Weber segment of the Wasatch fault zone, and the conclusion that at least 10 surface-faulting earthquakes occurred in the past 15 ky, suggests that seismic activity is a likely trigger for rock fall from the nearby cliffs. A characteristic to be evaluated in mapping the rock-fall hazard would be whether the large rock blocks came to rest individually over the course of the past



Hazard Mapping, Fig. 5 (**a**) A small part of a published of a landslide map that distinguishes those that occurred during a specific storm season from those that existed prior to the storm season. Two types of earth movements (debris flow and slide) are mapped (Modified from Crovelli and Coe (2009)). (**b**) A small part of a published of a landslide map that distinguishes those that occurred during the 1997–98 storm season from those that existed prior to that storm season. Two types of earth movements (debris flow and slide) are mapped (Modified from Coe et al. (2004)). Letter designations A, B, C, and D on the map are in the same positions as the letters on (**c**). (**c**) Natural color aerial photograph of the area of the map in (**b**). Letter designations A, B, C, and D on the map are in the same positions as the letters on the aerial photo. Google Earth Pro image dated 24 June 2007

15 ky, or if they came to rest at about the same time or otherwise clustered in time, which would support earthquake as the primary trigger for rock fall.

Landslides happen nearly every year in the San Francisco Bay region, California, USA, mostly during seasonal wet periods in fall through spring months (Crovelli and Coe 2009). Landslides were particularly widespread during four wet seasons: 1968–69, 1972–73, 1981–82, and 1997–98, with the damage in the 1981–82 season being caused by a single major storm on January 3–5, 1982. The US Geological Survey compiled cost information on landslide damage in a 10-county region for the four wet seasons that produced widespread damage from landslides; damage from landslides triggered by earthquakes was excluded from the compilation. Crovelli and Coe (2009) produced a map of annual probability of one or more damaging landslides (Fig. 5a) by creating clusters of landslides by year in which damage was reported for each year from 1968 to 2008. A 1-km-radius circle was used to count landslide clusters in the study area by moving the count-circle from point to point across a 200-m by 200-m grid of points. If a count-circle at one point encompassed two landslides clustered in 1968–69 and four landslides clustered in 1997–98, that point would be attributed with two clusters, one for each of the 2 years. This level of detail is a major step toward a landslide hazard map that actuaries could use, although the annual probability is based on damage-cost data attributed to the landslide, rather than on an intensity parameter of the landslide and fragility information of the feature that was damaged. "Damage" in this context might range from a fence being knocked over to a building being demolished. The map in Fig. 5a would be what was labeled Type 2a earlier in this definition of hazard mapping.

A landslide inventory map is shown in Fig. 5b for a small area identified in Fig. 5a. The landslide inventory (Coe et al. 2004) identified two ages of landslides, 1998 and pre-1998, and two types of landslides for each age, debris flow and slide. Coe et al. (2004) use “debris flow” to refer to fast-moving flows of mud (approximately equal amounts of sand, silt, and clay-size particles), gravel, and organic material. They use “slide” to refer to slow-moving rotational and translational slides, earth flows, and complex slope movements. A large number of landslides were mapped within the area of Fig. 5b, yet the annual probability of one or more damaging landslides appears to be in the >3 to 6% range designated by yellow in Fig. 5a. An aerial photograph taken in 2007 (Fig. 5c) shows that little development exists in the area of the numerous landslides mapped by Coe et al. (2004) (Fig. 5b); therefore, damage in rural areas as an indicator of potentially damaging landslides may miss important occurrences.

The global seismic hazard map (Fig. 3) is an example of Type 2b hazard mapping. It is a uniform hazard depiction of a key seismic hazard parameter, peak ground acceleration, for a specific annual frequency associated with an exposure time and exceedance probability. Landslides are more challenging hazards than earthquake ground motion because they are secondary features that are triggered by a primary hazard, typically earthquake ground motion, heavy precipitation, or erosion at the toe of a slope. However, given two essentially identical events of seismic shaking or precipitation, landslides may or may not occur. This is demonstrated in Fig. 5b with landslides mapped following the 1998 storms occurring in largely different places than landslides mapped following earlier storms. A dark green square outline is used to identify a single debris flow that appears to have occurred partially in the same place in 1998 as in an earlier storm year. A dark green circle outline is used to identify 23 slides that appear to have occurred partially in the same place in 1998 as in an earlier storm year. Additional knowledge would be needed for hazard mapping to identify key indicators of the damage potential of landslides associated with an annual frequency and locations where they are likely to occur only once or not at all.

Conclusions

Hazard mapping is performed for land-use planning and development zoning and for actuarial and engineering purposes. The first type of hazard mapping typically depicts areas within a political jurisdiction that limits development or requires sites to be investigated by a qualified professional so that appropriate consideration of the particular hazard is used in selecting the locations for facilities and designing them for compliance with safety ordinances and regulations. Hazard mapping of the second type can produce two results. One result depicts probabilities that a potentially damaging intensity will occur at specific locations over a specific time interval. A common example of this type of result is a weather forecast map of a tropical cyclone path showing the probability distribution of wind speed that meets the definition of hurricane or tropical storm. The second result depicts values over a map area of the intensity of a potentially hazardous process that are associated with a specific annual frequency; the annual frequency could be expressed as a specific probability that the hazard intensity would be equaled or exceeded (i.e., an exceedance probability) within a specific exposure time occurrence over the mapped area. Two potentially damaging natural processes that can be mapped to uniform hazard level for actuarial or engineering purposes are flooding and earthquake ground motion. Other natural hazards have not yet been mapped in terms of intensity associated with an annual frequency or a probability of occurrence (i.e., a 50-year exceedance probability of 0.02).

An important aspect of geophysical hazard mapping for earthquake ground motion, and soon also for flooding, of the type useable by insurance actuaries and engineers is that a value of hazard intensity associated with a constant annual frequency has been calculated for a regularly spaced grid of points across the entire world, not just for the high-hazard areas. Science-based process models are available for forecasting hazard intensities for wind speeds of tropical cyclones, water depths for riverine flooding, and ground acceleration for earthquake shaking. Damage caused by these three natural hazards is insurable because losses can be estimated. Models of other natural hazards, including landslides, do not exist in a form that allows losses to be estimated; therefore, damage caused by these is uninsurable until hazard mapping advances.

Cross-References

▶ Aerial Photography
▶ Clay
▶ Climate Change
▶ Collapsible Soils
▶ Earthquake
▶ Engineering Geological Maps
▶ Engineering Geomorphological Mapping
▶ Erosion
▶ Expansive Soils
▶ Faults
▶ Floods
▶ Geohazards
▶ GIS
▶ Ground Shaking
▶ Hazard
▶ Hazard Assessment

▶ Land Use
▶ Landforms
▶ Landslide
▶ Lidar
▶ Liquefaction
▶ Loess
▶ Mass Movement
▶ Organic Soils and Peats
▶ Probability
▶ Risk Mapping
▶ Site Investigation
▶ Subsidence
▶ Surface Rupture

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Hoek-Brown Criterion

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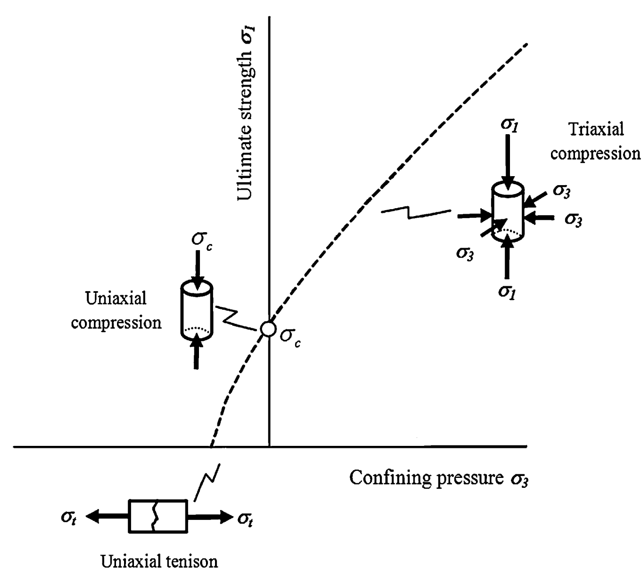
Definition

The **Hoek-Brown criterion** is an empirical, rock mass failure prediction criterion based on the relationship between principal stresses. The Hoek-Brown criterion was developed in the late 1970s and first published in 1980 (Hoek and Brown 1980a, b) to provide input for the design of underground excavations (Eq. 1 and Fig. 1). A fundamental assumption of the original Hoek-Brown criterion is that the rock mass to which it is being applied is homogeneous and isotropic. The criterion has been updated with time to accommodate more applications. The major updates include (1) the 1988 extension for applicability to slope stability and surface excavation problems (Hoek and Brown 1988), (2) the modified 1992 Hoek-Brown criterion for jointed rock masses (Hoek et al. 1992), and (3) the 2002 update to include improvements in the correlation between the model parameters and the Geological Strength Index (GSI). Subsequently this index was extended for weak rock masses (Hoek et al. 2002).

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m\frac{\sigma_3}{\sigma_c} + s} \tag{1}$$



Hoek-Brown Criterion, Fig. 1 The normalized Hoek-Brown envelope (Modified from Girgin 2009) where σ_1 is the major principal stress at failure, σ_3 is the minor principal stress, σ_c is the uniaxial compressive strength of the intact rock material, and m and s are constants that depend on the properties of the rock and on the extent to which it had been broken before being subjected to the failure stresses σ_1 and σ_3.

For intact rock material, $s = 1$, $m >> 1$ and can be approximated as $\sigma_c/|\sigma_t|$. For previously broken rock, $s < 1$; for a completely granulated rock mass specimen or a rock aggregate, $s = 0$. However, because of the difficulty involved in adopting the uniaxial tensile strength (σ_t) as a fundamental rock property, it is more practical to treat m simply as an empirical curve-fitting parameter. The value of m decreases with an increase in the degree of prior fracturing of a rock mass specimen (Hoek and Brown 1980a). Tables 1 and 2 in Hoek and Brown (1980a) are available to determine the value of m.

Since no suitable methods for estimating rock mass strength appeared to be available at the time when the Hoek-Brown criterion was developed, efforts focused on developing a dimensionless equation that could be scaled in relation to geological information. The original Hoek-Brown equation was a dimensionless equation, neither new nor unique – an identical equation had been used for describing the failure of concrete as early as 1936. The significant contribution that Hoek and Brown made was to link the equation to geological observations in the field. The Hoek-Brown criterion has continued to evolve to meet new applications and to deal with unusual conditions encountered by users (Hoek and Marinos 2007).

Cross-References

- ▶ Engineering Properties
- ▶ Excavation
- ▶ Failure Criteria
- ▶ Ground Pressure
- ▶ Mechanical Properties
- ▶ Pressure
- ▶ Rock Mass Classification
- ▶ Strain
- ▶ Strength
- ▶ Stress

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Hooke's Law

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Definition

Robert Hooke, a British physicist in the mid-1600s (UCMP, 2006), recognized a linear relationship between the weight of an object suspended from a conventional coil spring and the distance the spring deflected, provided that the object did not stretch the spring beyond its elastic range. Objects of different weights deflected the spring different distances that were proportional to the objects' weight, and the spring's deflection was uniformly distributed along its deformed length. The constant of proportionality for a spring is known as the spring constant, which is the stiffness of the spring over its elastic range. Hooke realized that his discovery was widely applicable to objects made from many materials that have become known generally as "deformable bodies" that have elastic ranges of response to loads or stresses. Thus, Hooke's law is the basis for the theory of elasticity.

The load-deflection concept from a spring experiment can be applied to a solid, uniform, right circular bar or a rock core sample; in its simplest form, a load applied axially to a bar or core sample results in a change in its length that is proportional to the magnitude of the load. The change in length (Δl) divided by the initial length (l_o) is the definition of strain (ε_x). Since the bar or core sample has a cross-sectional area, the load can be converted to an axial stress (σ_x). Stress applied to the bar or core sample divided by the strain it produces is the modulus of elasticity (E) for the bar or rock material. The load applied axially to the bar or core sample also results in a change in its diameter. The ratio of the change in diameter to the change in length is a material property known as Poisson's ratio (ν). The relationship between stress and strain in the elastic range is the basis for the elastic properties of most common materials and is important in engineering geology.

Cross-References

- ▶ Modulus of Elasticity
- ▶ Poisson's Ratio
- ▶ Strain
- ▶ Stress

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Hydraulic Action

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Definition

Hydraulic action refers to the physical weathering and mechanical response of Earth materials to flowing water in rivers and streams or breaking waves and storm surge along shorelines. Physical weathering by flowing water is a rock-water interaction phenomenon (Keaton 2013). Hydraulic action implies that the water exhibits Newtonian behavior (velocity-dependent hydraulic shear strength) and has sediment concentration less than about 33% by mass corresponding to a fluid unit weight no greater than about 13 kN/m^3. Such clear water could erode degradable rock by gradual and progressive abrasion and grain-scale wearing away and might erode jointed fragments of durable rock by quarrying and plucking depending on the size and shape of the rock fragments and turbulence intensity and velocity of the flowing water. Gradual and progressive wear of degradable rock persists in response to the stream power of the flowing water; degradable rock wears away faster in response to flow that has higher unit stream power. Quarrying and plucking of durable rock fragments is a threshold phenomenon that is indexed to the flow velocity; dislodgment of rock fragments tends to happen at flows that reach and exceed certain velocities.

Breaking waves and storm surge along shorelines have substantial energy and repeated application over a relatively small range of Earth material extent. Degradable rock material tends to wear rapidly, leading to creation of ragged and rough coastal bluffs that continually slough Earth material to the coastline below. The ability of the breaking waves and storm surge creates a slope profile that is limited by the rate of weathering and sloughing of material on the slope above sea level, because the shoreline processes have capacity to transport as much eroded rock and soil as the slope can produce. Durable rock material tends to erode slowly after the smaller fragments defined by closely spaced rock defects have been removed.

Erosion protection of river banks and beds, bridge piers and abutments, and shorelines from the hydraulic action of flowing water and waves and storm surge typically is provided by placement of armour stone or riprap.

Hydraulic action also can refer to the effect of hydrostatic pressure acting in all directions and its destabilizing action on soil and rock slopes.

Cross-References

- ▶ Abrasion
- ▶ Armor Stone
- ▶ Classification of Rocks
- ▶ Coast Defenses
- ▶ Coastal Environments
- ▶ Current Action
- ▶ Durability
- ▶ Engineering Properties
- ▶ Erosion
- ▶ Fluvial Environments
- ▶ Levees
- ▶ Mechanical Properties
- ▶ Nearshore Structures
- ▶ Physical Weathering
- ▶ Rock Coasts
- ▶ Rock Mechanics
- ▶ Rock Properties
- ▶ Sea Level
- ▶ Soil Mechanics
- ▶ Soil Properties
- ▶ Stabilization
- ▶ Tsunamis

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Hydraulic Fracturing

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Synonyms

Fracking; Hydrofracking

H

Definition

Hydraulic fracturing is the process by which fluids are injected under pressure down a borehole and into a targeted rock formation to generate fractures.

Hydraulic fracturing has several applications. Because the injection pressures must overcome the near and far-field stresses to generate and propagate a hydraulic fracture, hydraulic fracturing is commonly used as an *in situ* stress measurement technique. Because the fractures generated produce higher permeability flow paths, hydraulic fracturing is the primary means of enhancing well productivity in the development of unconventional (i.e., low permeability) oil and gas reservoirs, as well as enhanced geothermal projects. And because the fractures also weaken the rock mass, hydraulic fracturing has been used in the mining industry to precondition an orebody to ensure suitable fragmentation for caving and mining. Nevertheless, hydraulic fracturing is not without its public concerns, especially with respect to water use and contamination, induced seismicity, and greenhouse gas emissions through fugitive gas.

History and Applications

Hydraulic fracturing was first introduced in the United States in the late 1940s by Stanolind Oil and later commercialized in the early 1950s by Halliburton to increase production from oil and gas wells. The early technique involved injecting a blend of crude oil and gasoline at treatment pressures that would fracture the reservoir rock, together with sand to prop the fractures open (Montgomery and Smith 2010). The first wells treated saw an average increase in production of 75%. This led to a rapid growth in the application of hydraulic fracturing, and by the mid-1950s, more than 100,000 individual treatments had been performed (Hubbert and Willis 1957).

Hydraulic fracturing saw its next step change in the 1990s when it was combined with advancements in horizontal drilling. This greatly expanded the viability of low permeability geological plays, particularly shale gas. This has led to increasingly longer horizontal wells reaching out several kilometers with numerous hydraulic fracturing stages in a single well. Today, Montgomery and Smith (2010) estimate that more than 2.5 million hydraulic fracture treatments have been carried out worldwide and that hydraulic fracturing has increased recoverable reserves of oil and gas in the United States by at least 30% and 90%, respectively.

In parallel to these developments, hydraulic fracturing was developed as a technique to measure the *in situ* stress state in rock at depth (Haimson and Fairhurst 1969). The *in situ* stresses are a key boundary condition in the engineering analysis and design of underground excavations. Hubbert and Willis (1957) observed that hydraulic fractures form relative to the orientation of the *in situ* stresses, opening in the direction of the minimum principal stress and propagating in the direction of the maximum principal stress. It was realized that these observations could be used to determine the minimum principal stress by measuring the pressure required to keep the hydraulic fracture open after pumping has stopped. This is also referred to as the shut in pressure. The maximum principal stress can then be calculated based on the breakdown pressure required to initiate the hydraulic fracture and rupture the rock. By the 2000s, hydraulic fracturing established itself as one of the suggested methods for rock stress determination by the International Society of Rock Mechanics (Haimson and Cornet 2003).

Familiarity with hydraulic fracturing in the mining industry also led to further investigations on its use in the 1990s and 2000s for other mining applications. Of interest was the use of hydraulic fracturing to "precondition" the rock to alter its characteristics in advance of or during mining. This led to experiments at the Northparkes mines in Australia to use hydraulic fracturing to induce caving of the orebody, as required for the block caving mining method being employed (van As and Jeffrey 2000). Hydraulic fracturing has since been employed to improve fragmentation and mitigate risk of poor caveability associated with stronger rock masses being encountered at a number of block caving operations. More recently, hydraulic fracturing has also been suggested as a means to mitigate rockburst hazard by reducing the stiffness of the rock mass in critically stressed areas and redistribute stress away from active mining advances (Kaiser et al. 2013).

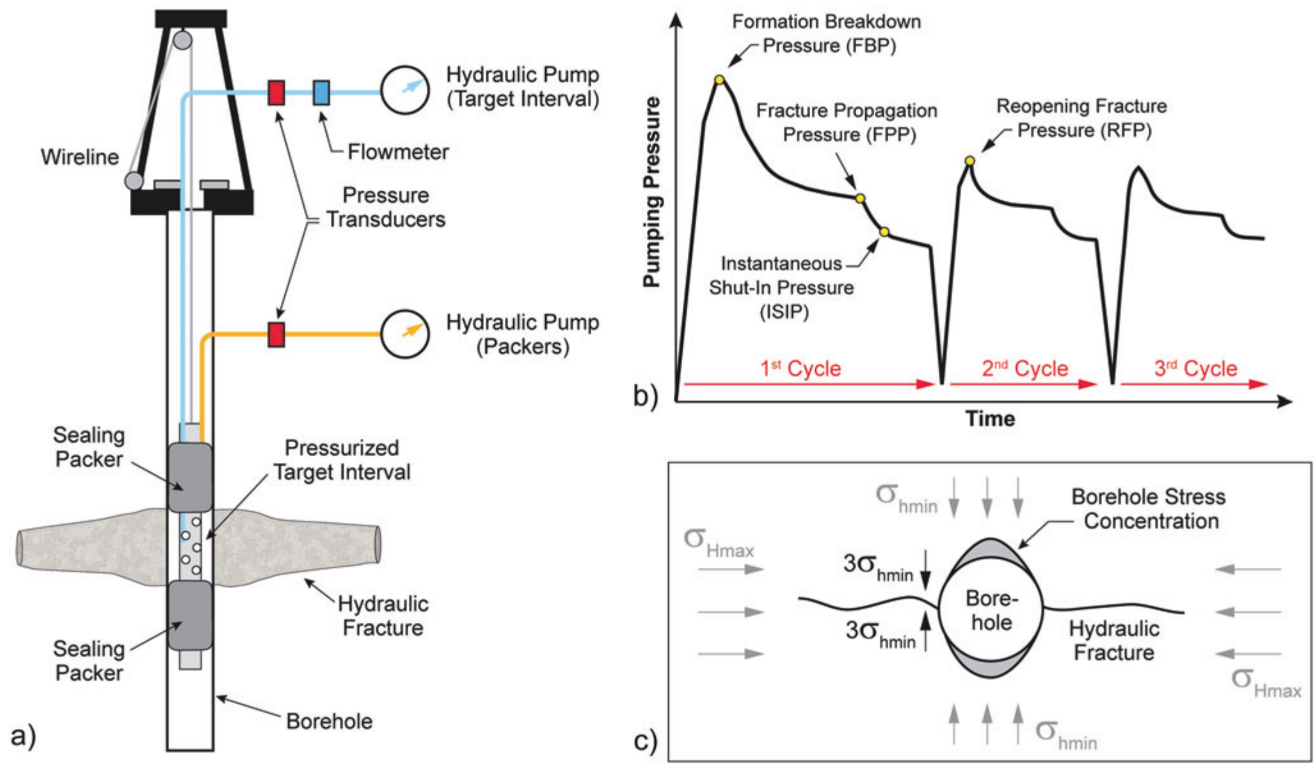
Fundamentals

To generate a hydraulic fracture, a sealed-off borehole interval is pressurized by pumping water-based fluids into the borehole faster than the fluid can escape into the rock (Fig. 1a). As the resulting pressure increases, it will eventually exceed the critical pressure required to initiate a hydraulic fracture at the borehole wall (Fig. 1b). This is referred to as the Formation Breakdown Pressure (FBP), which is a function of the stress concentration generated around the borehole wall and the tensile strength of the rock. If assuming the targeted rock interval is elastic and impermeable, the FBP can be calculated for a vertical borehole as:

$$FBP = 3\sigma_{hmin} - \sigma_{Hmax} + T_0 \qquad (1)$$

where $\sigma_{h\mathrm{min}}$ and $\sigma_{h\mathrm{max}}$ are the minimum and maximum horizontal stresses, respectively (Fig. 1c), and T_0 is the tensile strength of the rock.

If the pumping rate and pressure are maintained, then the initiated hydraulic fracture will continue to propagate and grow (FPP in Fig. 1b). After pumping has been stopped, the



Hydraulic Fracturing, Fig. 1 (**a**) Schematic illustration of wireline hydraulic fracturing setup (Modified after Haimson and Cornet 2003); (**b**) typical hydraulic fracture treatment record of pumping pressure versus time; and (**c**) geometry of a hydraulic fracture relative to the maximum and minimum horizontal in situ stresses, for a horizontal plane through a vertical borehole

Instantaneous Shut-In Pressure can be determined (ISIP in Fig. 1b). The ISIP is the minimum pressure needed to keep the hydraulic fracture open and is equated to the minimum horizontal stress:

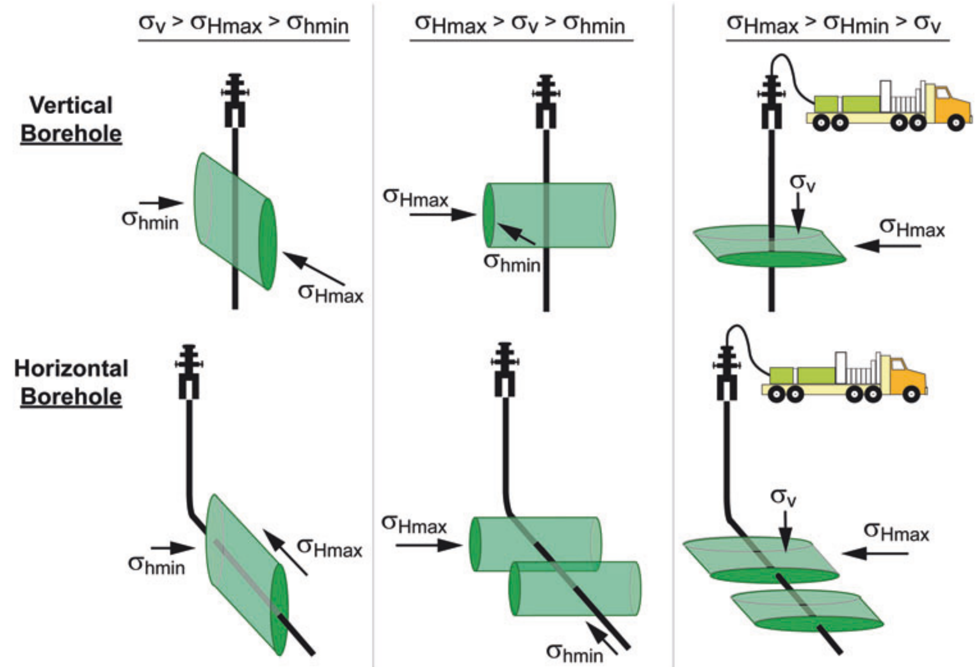
$$ISIP = \sigma_{hmin} \tag{2}$$

Thus, with measurement of the breakdown and shut-in pressures, Eqs. 1 and 2 can be used to calculate the *in situ* stresses, assuming that they are aligned with vertical and horizontal. Hubbert and Willis (1957) observed that hydraulic fractures form relative to the orientation of the in situ stresses: in extensional regimes, where the maximum principal stress is vertical, hydraulic fractures propagate vertically; in compressional regimes, where the maximum principal stress is horizontal, hydraulic fractures propagate horizontally (Fig. 2). In effect, hydraulic fractures open in the direction of the minimum principal stress and propagate in the direction of the maximum principal stress.

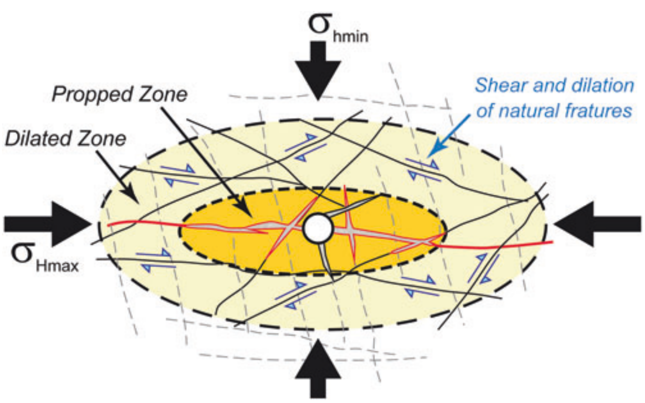
It should also be recognized that the elastic continuum assumptions on which hydraulic fracturing theory is based represent a significant simplification of the actual geological conditions present. This includes the presence of natural fractures, such as bedding, joints, and faults, which the hydraulic fracture will interact with, which in turn can influence the size, orientation, and path of the hydraulic fracture (Zangeneh et al. 2015). Correspondingly, Kaiser et al. (2013) describe a hydraulic fracture not as a single feature but as a zone of branching and dilating fractures adjacent to the propagating fracture (Fig. 3). From this, a distinction can be made between hydraulic fracturing where new fractures initiate and propagate in response to fluid injection, and hydraulic shearing where fluid pressure leaks off into adjacent natural fractures inducing shear slip and dilation (Preisig et al. 2015). Note that hydraulic fracturing and hydraulic shearing are conceptual end-members which act to varying degrees in combination. Preisig et al. (2015) further demonstrated that tensile opening of a hydraulic fracture will generate an increase in stress, termed a stress shadow, which may limit the response of adjacent hydraulic fractures in terms of both tensile opening and hydraulic shearing.

Design Variables

The design of a hydraulic fracture treatment for oil and gas or geothermal reservoir enhancement depends on several key parameters. These include both factors related to the reservoir geology and operational factors related to the hydraulic fracturing treatment.



Hydraulic Fracturing, Fig. 2 Relationship between hydraulic fracture opening and propagation directions relative to the in situ principal stresses, assuming these are aligned with vertical and horizontal



Hydraulic Fracturing, Fig. 3 Stimulated volume in a naturally fractured rock mass, including influence of natural fractures on hydraulic fracture propagation path, and shear and dilation along critically oriented adjacent natural fractures (Modified after Kaiser et al. 2013)

Examples of key geological factors include the thickness of the targeted formation and the rock's elastic modulus and permeability (both matrix and that from natural fractures). Evidence from production logs and other data indicate that hydraulic fractures often terminate shortly after penetrating into over-/underlying formations with contrasting rock properties. Formation thickness therefore represents a key design input in the form of fracture height, which in turn governs the propagation of the hydraulic fracture. For a thick formation, the net pressure (i.e., fluid pressure inside the fracture minus the fracture closure stress) will be much lower than for a thin formation, making it easier to confine a fracture to a thicker target zone. The net pressure will also influence the maximum opening width of the hydraulic fracture, but this also depends on the elastic stiffness of the formation rock. For a given net pressure, higher rock stiffness values will result in reduced fracture opening widths. Maintaining fluid pressure, for both breakdown and initiation of the hydraulic fracture as well as controlling net pressure to open and propagate the hydraulic fracture, is dependent on the formation permeability. This is often inputted as a fluid loss coefficient. Fluid loss controls how much fluid escapes into formation and therefore affects the net pressure.

Key operational factors include the use of proppants, fluid viscosity, and pump rates. During pumping, the hydraulic fracture is held open by the fluid pressure. However, once pumping stops and the injection pressure dissipates, the minimum principal stress will act to close the hydraulic fracture created. For applications where hydraulic fracturing is used to increase the permeability of the reservoir rock, fracture closure will significantly reduce the fracture permeability created. To prevent this, a propping agent, typically sand, is added to the hydraulic fracturing fluid to maintain an open, conductive fracture. Montgomery and Smith (2010) note that today's oil and gas reservoir treatments average approximately 45 metric tons of propping agent and 200 m^3 of fluid, with the largest treatments exceeding 2000 metric tons of propping agent and 4000 m^3 of fluid.

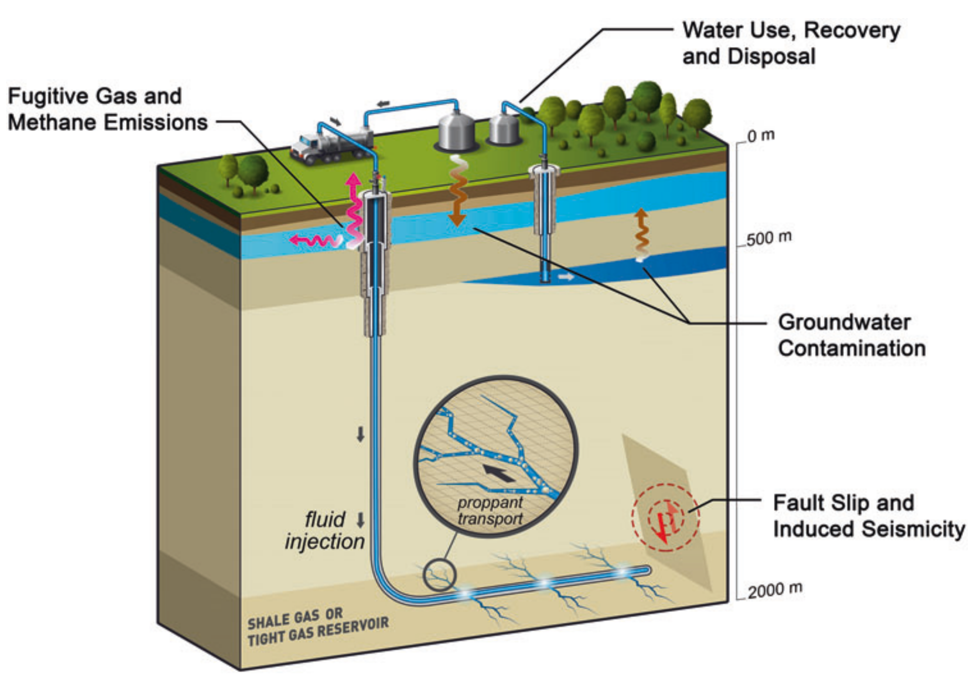
Fluid viscosity and pump rate work in unison to control the net pressure to attain the desired hydraulic fracture height, as well as to ensure sufficient opening to allow proppant to enter the fracture and carrying velocity to transport the proppant deep into the hydraulic fracture. Fluid viscosity also plays an important role in minimizing friction pressure losses during injection, which can limit fracture propagation. Gelling agents are added to water-based fracturing fluids to obtain the desired fluid viscosity, with gel stabilizers to contend with high-temperature boreholes. Hydraulic fracturing operations targeting shale gas formations include a combination of additives in what is referred to as a “slickwater” treatment, including friction reducers, biocides, scale inhibitors, and surfactants. In this case, an ultra-low viscosity is preferred resulting in minimal use of gels to enable a greater breakdown of fissures, microcracks, and bedding in shales to open up more fracture contact area and therefore permeability (King 2010). Friction reducers are used to allow pumping of the fluid at higher rates to transport proppant, in place of the use of a higher viscosity fluid. Biocides are added to reduce equipment corrosion from acid producing bacteria, as well as bio-clogging of fractures that can inhibit gas extraction. The use of biocides also allows the use of recycled water by preventing souring using sulfate reducing bacteria, helping to minimize water use and wastewater volumes.

Issues and Hazard Mitigation

Although hydraulic fracturing has application in many industries, its use in the development of shale gas and enhanced geothermal projects have attracted public concern over its pace and environmental footprint. These environmental impacts include water use, potential contamination of groundwater resources, induced seismicity, and in the case of shale gas, methane emissions during and after hydraulic fracturing operations (Fig. 4).

Water use requirements can be substantial, especially in the case of multistage fracturing used to maximize horizontal well performance for shale gas extraction. Gallegos et al. (2015) report average hydraulic fracturing water usage of 10,000–36,000 m^3 per well for shale gas areas across the United States where multistage fracturing is utilized. Given these volumes, wastewater associated with flowback from shale formations after a hydraulic fracturing treatment raises concerns regarding potential contamination of groundwater resources. Wastewater may contain salt, elements such as selenium, arsenic, and iron, and small amounts naturally occurring radioactive materials, all of which come from the gas-producing shale formations (Zoback and Arent 2014). To reduce the volumes of wastewater requiring treatment and disposal, flowback waters are often reused for subsequent treatments. Zoback and Arent (2014) note that this reduces both the need for new water sources and concerns associated with wastewater disposal. The use of fresh water has been further mitigated by using brackish or saline water for drilling and hydraulic fracturing. Note that shale gas formations in North America are typically 2000–3000 m deep and well separated from the much shallower aquifers, which are typically less than a few hundred meters deep (Gallegos et al. 2015). Thus, the likelihood of groundwater contamination directly related to hydraulic fracturing and the migration of fracturing fluids is remote (King 2010). Instead, more likely sources would require spills, leaks, or improper disposal of inadequately treated wastewater.

Hydraulic Fracturing, Fig. 4 Environmental concerns associated with multistage hydraulic fracturing operations for shale gas development



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It has also been well established that injecting large volumes of fluid into deep formations during hydraulic fracturing treatments or wastewater disposal can trigger small earthquakes, referred to as induced seismicity. Fluid injection increases the pore pressures in the formation, which in the presence of a critically stressed fault, will reduce the effective stresses and shear resistance along the fault, causing it to slip and release the elastic strain energy stored in the surrounding rocks. Induced seismicity events of up to magnitude 4.6 have been recorded at The Geysers enhanced geothermal project in Northern California, and induced seismicity has contributed to the cancellation of the Basel Deep Heat Mining enhanced geothermal project in Switzerland. McGarr (2014) reports several induced seismicity events larger than magnitude 4.0 associated with oil and gas production activities, including a magnitude 5.7 event associated with wastewater injection in 2011 in Prague, Oklahoma (although a natural origin cannot be ruled out). Notable events associated with hydraulic fracturing operations targeting unconventional oil and gas reservoirs include a magnitude 4.6 event in 2015 in the Montney play in northeastern British Columbia. To mitigate induced seismicity hazards, injection rates are managed to minimize pore pressure increases (i.e., injecting at lower rates), and pore pressures and microseismicity are monitored to establish protocols in advance that define how operations should be modified in the event of seismicity (Zoback and Arent 2014).

Another key concern is the accidental release of greenhouse gases during hydraulic fracturing of shale gas reservoirs and subsequent leaking of wells during production. Natural gas is mostly composed of methane, which is a powerful greenhouse gas, meaning even small releases to the atmosphere can greatly influence the greenhouse gas footprint of shale gas. Howarth (2015) cites satellite data that suggest methane emissions from shale gas operations may be as high as 12% of the total gas produced, when considering the full life cycle including storage and delivery to consumers. Higher emissions can be attributed to venting of gas during the flow back period following high-volume hydraulic fracturing. This has led to efforts to recover gas produced by separating the gases and solids after completing the well to allow the gas to be sent into production instead of being released into the atmosphere. Other efforts to mitigate methane emissions include identifying sources of leaks and devising methods to stop them.

Summary/Conclusions

Hydraulic fracturing has been extensively used for more than 60 years as a primary means of increasing the productivity of oil and gas wells in low permeability reservoir rocks. It has evolved and been adapted for similar permeability enhancement purposes for geothermal projects, as well as to increase rock fragmentation for mining projects, and notably as one of the key recommended methods for measuring *in situ* stresses. Its integration with advancements in horizontal drilling has led to the viability and expanded development of low permeability unconventional oil and gas plays, particularly shale oil and shale gas, fueling more than 50% and 70%, respectively, of current US oil and gas outputs. The process involves injecting fluids under pressure down a borehole and into a targeted rock formation to generate fractures. Proppants such as sand are added to maintain the openness and conductivity of the fractures, as are other chemicals to optimize the effectiveness of treatments. Water use requirements and the handling and disposal of hydraulic fracturing fluids have contributed to environmental concerns, as have other related issues like induced seismicity and fugitive gas. These are the subjects of ongoing research and industry solutions to further mitigate the impacts and minimize the environmental footprint of hydraulic fracturing operations.

Cross-References

▶ Aquifer
▶ Bearing Capacity
▶ Bedrock
▶ Compaction
▶ Compression
▶ Consolidation
▶ Contamination
▶ Deformation
▶ Dispersivity
▶ Effective Stress
▶ Elasticity
▶ Engineering Properties
▶ Faults
▶ Fluid Withdrawal
▶ Geothermal Energy
▶ Ground Pressure
▶ Hazard
▶ Hydrocompaction
▶ Induced Seismicity
▶ Instrumentation
▶ Lateral Pressure
▶ Mechanical Properties
▶ Monitoring
▶ Normal Stress
▶ Pore Pressure
▶ Rock Mechanics
▶ Rock Properties
▶ Strain
▶ Subsidence
▶ Water
▶ Water Testing

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Hydrocompaction

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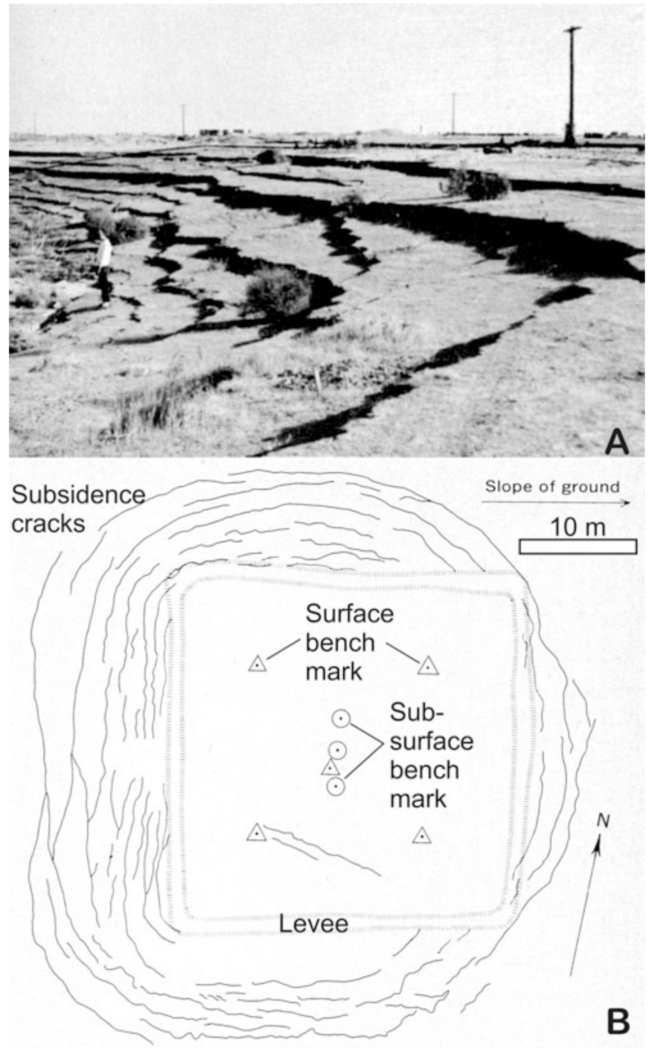
Definition

A reduction in porosity of earth materials, accompanied by an increase in unit weight, as a result of water soaking. Compacting soil solely by adding water, sometimes called “jetting” if the application is done with a hose and nozzle system, has been used to increase the unit weight of loosely placed sandy soil backfill in shallow trenches around utility pipelines. Natural



Hydrocompaction, Fig. 1 Surface cracks adjacent to test plots in the arid San Joaquin Valley, California, USA, where water was ponded during characterization of the alignment for the California Aqueduct in the 1960s (Bull 1964). (**a**) Subsidence cracks after about 14 months of ponding; ground surface subsidence exceeded 3 m and the depth of documented soaking-induced hydrocompaction exceeded 40 m (Bull 1964, Fig. 21B). (**b**) Concentric subsidence cracks mapped 42 days after initial filling of a test pond (Bull 1964, Fig. 23)

deposits susceptible to hydrocompaction under self-weight loading are called collapsible soils. Collapsible soils are a type of moisture-sensitive soils, a term which also applies to soils that swell upon application of water and shrink as they dry (i.e., expansive soils). “Collapse” implies that the process begins suddenly and advances rapidly upon soaking.

Natural sediments that may be susceptible to hydrocompaction were deposited in a moisture-deficient condition, usually in arid and semiarid climate conditions, and have a depositional fabric or structure that allows the landscape to be apparently stable under ambient conditions, meaning that the landscape is stable under the self-weight of the deposits while remaining dry. Three general types of surficial deposits can be susceptible to hydrocompaction: a) wind deposited silts (loess), b) some primarily fine-grained colluvial soils, and c) some debris flow or mudflow deposits forming alluvial fans. These deposits in humid-subtropical climate conditions can become hydrocompacted to the depth of natural wetting, and retain their collapse potential below that depth to the groundwater Table. A change to a tropical climate can result in deeper wetting and additional collapse in the soils that become wetted for the first time since they were deposited. An increase in the amount of compaction with depth at a test plot with a constructed pond was documented by Bull (1964) and attributed to the overburden load of soaked soil and the thickness of hydrocompactible deposits, which exceeded 40 m (Fig. 1). Human activities can trigger collapse of susceptible soils: for example, (a) landscape irrigation, (b) redirection of storm runoff, (c) leaking buried pipelines, and d) ponding. Zones of soils that may have moderate susceptibility to hydrocompaction can attain higher susceptibility by action of burrowing animals and insects and by growth of plant roots that subsequently decay and disintegrate.

Construction of a building, such as a house, may impose a load small enough to be supported by the metastable soil structure without inducing deformation or collapse of the soil formation (Houston et al. 2001). However, it is common for storm drainage from building rooftops to be discharged adjacent to buildings, as well as for landscape irrigation to take place, which can lead to excessive water infiltration into the ground. Dramatic damage to buildings and infrastructure has occurred as a consequence of urban development in areas of thick hydrocompactible soils that have not been detected prior to construction.

Cross-References

- ▶ Characterization of Soils
- ▶ Collapsible Soils
- ▶ Compaction
- ▶ Compression
- ▶ Desert Environments
- ▶ Fluvial Environments
- ▶ Infiltration
- ▶ Loess
- ▶ Subsidence
- ▶ Subsurface Exploration

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Hydrogeology

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Synonyms

Geohydrology

Definition

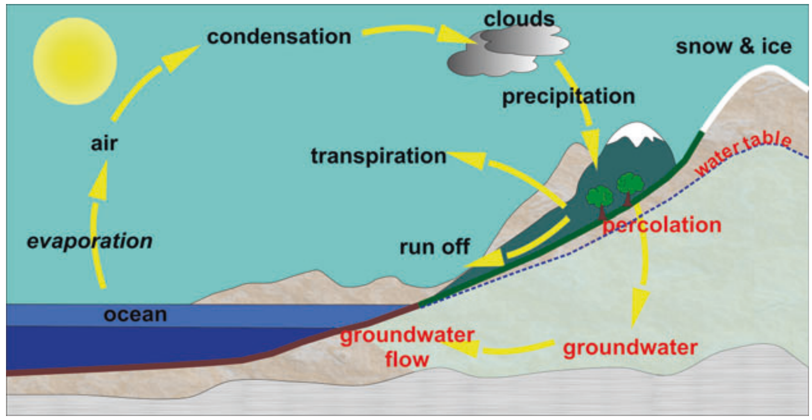
The study of the part of the global water cycle (hydrologic cycle) that takes place underground (Fig. 1).

Hydrodynamics (i.e., water flow-paths, water residence time) and hydrochemistry (i.e., water chemistry) of groundwater are both intimately related to the geologic material properties (porosity, permeability, mineralogy) and to the geological scenarios (e.g., stratigraphic and structural pattern, volcanism, gas-water interaction). This interrelationship is the main concern of hydrogeology. Applied hydrogeology is mainly focused on the assessment of the actual availability of water resources for different purposes and their protection against both overexploitation and pollution. As well, in hydrogeological studies the interaction between groundwater resources and surface water should be carefully considered to provide a comprehensive view of the river-aquifer system (Winter et. al. 1998) and to gain a sustainable management of renewable, but not endless, water resources· Rivers and springs represent the surface manifestation of groundwater and represent the *base level* of hydrogeological system (Castany 1982).

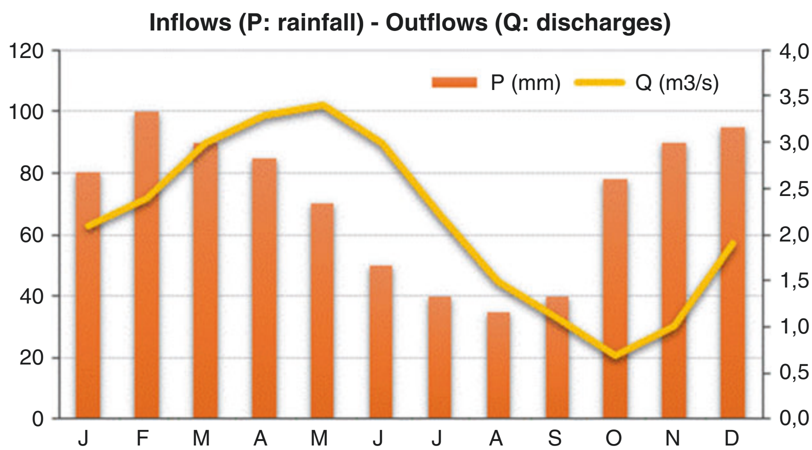
Hydrogeological Methods

Hydrogeology is a multidisciplinary science that requires descriptive and analytic disciplines. Field *geological survey* supports the identification of *aquifers* (see "Definition") and its boundaries. Other disciplines such as *geophysics*, *hydrology*, *geochemistry*, *meteorology*, *mathematical modeling*, *hydraulics*, *biology*, and *remote sensing* are necessary in the modern hydrogeology. Therefore composite expert teams should be encouraged when facing complex hydrogeological studies.

Hydrogeology, Fig. 1 The global water cycle. The section occurring underground (marked in red) defines the subjects of hydrogeology



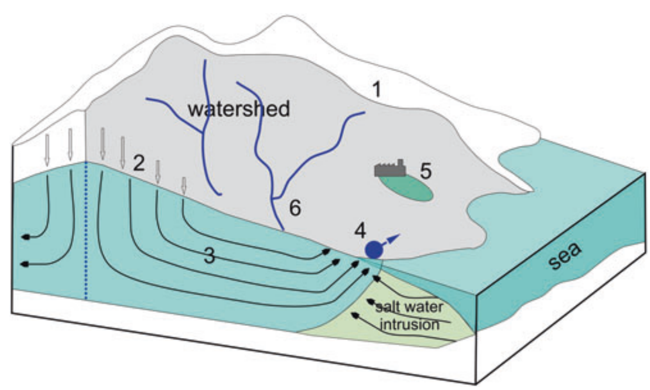
Hydrogeology, Fig. 2 Monthly hydrograph (discharge, Q (m3/s)) and rainfall (input of the hydrogeological system, P(mm)) of a hypothetical spring: represent the basic data for water balance analysis



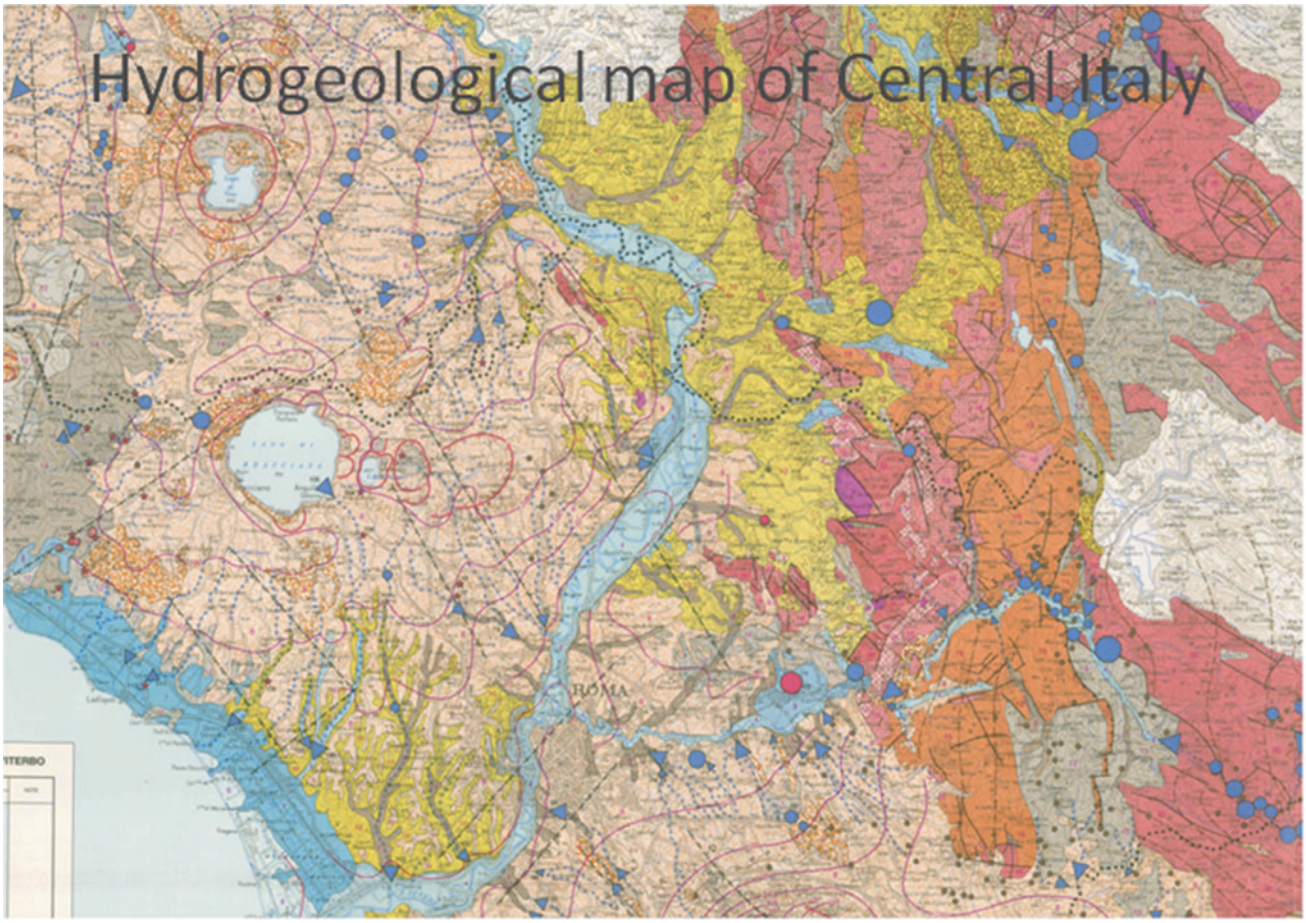
Hydrogeological investigation methods depend on targets (e.g., evaluation of available resources, water use planning, prevention, protection, and restoration of polluted groundwater).

Quantitative hydrogeology uses direct measurement methods in order to acquire the basic parameters as: flows measurements at discharge points (springs and rivers coming out of the hydrogeological systems); groundwater level monitoring; direct measurement of effective infiltration; inflow and precipitation; air temperature; air humidity; chemical-physical parameters of groundwater and surface water such as pH, temperature, electrical conductivity, content of dissolved salts, among others (see Rosenberry and LaBaugh 2008).

Furthermore, indirect methods are used in order to assess – at the scale of the whole hydrogeological system – the availability of water resources as well as the response of the aquifer to the hydrodynamic impulses (Fig. 2). Among the "indirect methods" of hydrogeological investigation, water balance analysis allows to identify the amount of effective infiltration and aquifer recharge, which represent the basic knowledge for water resources planning and management.



Hydrogeology, Fig. 3 Hydrogeological conceptual model (Modified after G. Castany 1982)



Hydrogeology, Fig. 4 Hydrogeological map of Central Italy (Boni et al. 1986)

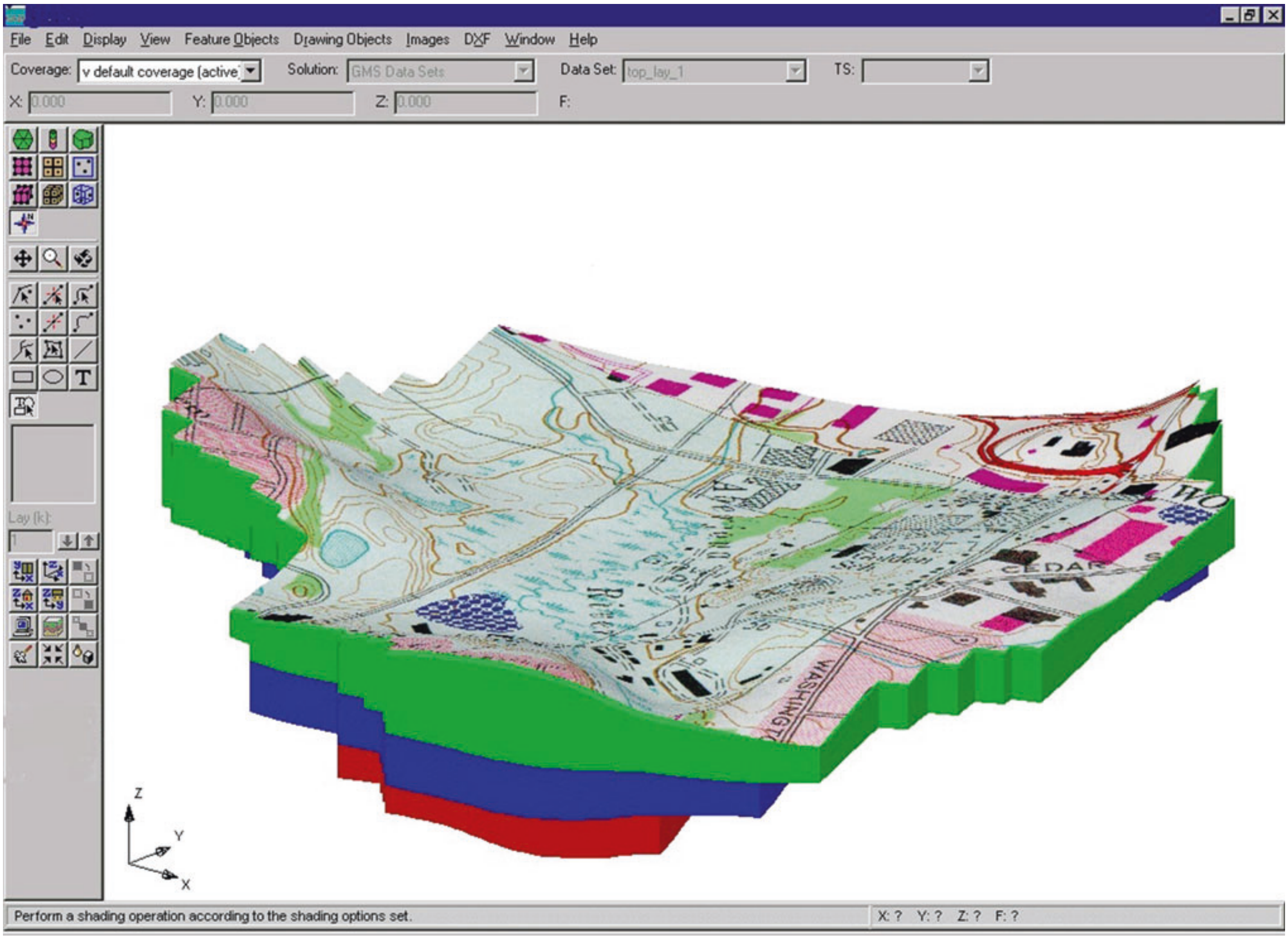
Hydrogeological Conceptual Model and Output of the Hydrogeological Assessment

The most important tool for any hydrogeological investigation is the *hydrogeological conceptual model*, a simplified three-dimensional scheme (Fig. 3) that reports the essential parameters of the hydrogeological characterization and explains how the hydrogeological system works: (1) geometry of reservoir and its boundaries; (2) recharge areas; (3) groundwater circulation paths; (4) discharge zones, represented by the localized springs and rivers coming out from the system; (5) the pressures threatening groundwater; and (6) interactions with surface waters and connected ecosystems.

The construction of the hydrogeological model requires the availability of basic hydrological parameters monitored over time (flow rates, precipitation, water table levels, water temperature, as well as other chemical-physical parameters). This information availability is achieved through a proper management of *data monitoring networks*, both quantitative (volume of water) and qualitative (water chemistry), covering the whole hydrogeological system.

Field data, elaborations, and conceptual model are summarized in the *hydrogeological maps* (Fig. 4); these are drawn on the basis of geology and contain two kinds of information: hydrogeological characteristics of the aquifer (geometry, boundaries, permeability, storage capacity, transmissivity) and water body characteristics (water table geometry, groundwater paths, discharge zones such as springs and rivers).

Whereas hydrogeological maps offer a static representation of the hydrogeological systems, mathematical modeling allows a dynamic representation of water body and hydraulic functioning (Fig. 5). It can show several scenarios by setting different boundary conditions, depending on the purposes of the study (e.g., water abstraction, remediation of contaminated sites, research projects, etc.).



Hydrogeology, Fig. 5 Mathematical modeling of a three-layer aquifer. It is a powerful tool for prevision in water abstraction and remediation of contaminated sites issues (From web: http://igwmc.mines.edu/software, Integrated Groundwater Modeling Center, Colorado, modified)

Cross-References

- ▶ Aquifer
- ▶ Aquitard
- ▶ Artesian
- ▶ Catchment
- ▶ Desert Environments
- ▶ Desiccation
- ▶ Environments
- ▶ Floods
- ▶ Fluid Withdrawal
- ▶ Fluvial Environments
- ▶ Groundwater
- ▶ Hydrology
- ▶ Mountain Environments
- ▶ Percolation
- ▶ Piezometer
- ▶ Run-off
- ▶ Saturation
- ▶ Tropical Environments
- ▶ Water

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Hydrology

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Synonyms

Engineering hydrology; Geohydrology; Hydraulics; Hydrometeorology

Definition

A sub-discipline of Geology concerning the study of complex water systems including occurrence, movement, distribution, quality, and sustainability of water and relationships with the environment (Anderson and MacDonnell 2005).

Water is one of our most important natural resources. It covers 70% of the Earth's surface and is an important groundwater resource. Table 1 provides an estimate for the amount of water present on the Earth at a single time. At present only about 0.8% of the world's fresh water remains, and it is continuously being depleted in a number of locations worldwide.

Hydrology, Table 1 Estimated volumes of water held at the Earth's surface (After Shiklomanov 1993)

Water source	Water volume, in cubic miles	Water volume, in cubic kilometers	Percent of total water
Oceans, seas, and bays	321,000,000	1,338,000,000	96.5
Ice caps, glaciers, and permanent snow	5,773,000	24,064,000	1.74
Ground water	5,614,000	23,400,000	1.69
Fresh	2,526,000	10,530,000	0.76
Saline	3,088,000	12,870,000	0.93
Soil Moisture	3,959	16,500	0.001
Ground Ice & Permafrost	71,970	300,000	0.022
Lakes	42,320	176,400	0.013
Fresh	21,830	91,000	0.007
Saline	20,490	85,400	0.006
Atmosphere	3,095	12,900	0.001
Swamp Water	2,752	11,470	0.0008
Rivers	509	2,120	0.0002
Biological Water	269	1,120	0.0001

The hydrologic cycle, also known as the water cycle, is the fundamental concept in hydrology. That is the process by which water, following rainfall or snow melt, moves downhill to the streams, rivers, and finally, to the oceans. Surface water can be absorbed into the soil, recharge the ground water reservoirs (aquifers) and remain stored for years, or may discharge in wells, springs, or streams. Water from rivers and oceans returns to the atmosphere by evaporation and transpiration – evaporation through plants, to continue the cycle (Viessman and Lewis 2002). Humans use water for domestic, agricultural, industrial, and electric power supply purposes. After use, water is generally returned back to the hydrologic cycle. But the recycled water is normally lower in quality and often poses environmental problems if it is not properly treated.

The balance in hydrologic cycle can be represented by a water balance equation:

$$\mathrm{S} = P - Q - E - G \tag{1}$$

where S is the change of water storage in the area over a given time period, P is the precipitation input during that time period, Q is the stream discharge from the area, E is the total of evaporation and transpiration to the atmosphere from the area, and G is the subsurface outflow. This equation assumes the conservation of mass in a closed system and is the conceptual basis for any hydrological model (Jayawardena 2014).

The water balance equation can help predict water supply and its shortages, and can be used for designing irrigation systems, runoff assessment, flood control, and contamination studies. The equation is mostly applied at the drainage basin scale, where a drainage basin is defined by an area of land where precipitation collects and discharges off into a common outlet, such as into a stream, river, or other body of water.

Drainage Patterns

There are four basic types of drainage patterns: dendritic, trellis, rectangular, and radial. A dendritic drainage pattern is a branching stream of streams and is developed on horizontally bedded sedimentary rocks or homogeneous igneous and metamorphic rocks or thick soil sequences. Trellis drainage consists of elongated, parallel channels, developed in weaker rocks, with short, nearly perpendicular tributaries joining at right angles from the ridges made of harder rock units. Rectangular drainage, controlled by rock structure, consists of perpendicular segments of streams without the dominant elongation of one orientation as seen in trellis drainage. Radial drainage is caused by streams radiating from a high central point, such as a volcanic peak or conical dome.

Hydrograph

Drainage discharge is expressed in volume per unit time (e.g., cubic meters per second) and is represented in the form of a hydrograph which shows the variation of discharge with respect to time. The peak discharge on a hydrograph represents a flood stage. Based on hydrograph analysis, a hydrologist usually estimates the flood with a recurrence interval of 50 or 100 years, or longer, for design of hydraulic structures (Maidment 1993).

Hydrology has evolved as an important discipline of earth sciences and there are several branches including engineering hydrology, chemical hydrology, hydrogeology, hydrometeorology, etc.

Cross-References

▶ Catchment
▶ Desert Environments
▶ Desiccation
▶ Environments
▶ Floods
▶ Fluvial Environments
▶ Groundwater
▶ Hydrogeology
▶ Mountain Environments
▶ Percolation
▶ Piezometer
▶ Run-off
▶ Saturation
▶ Tropical Environments
▶ Water

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Hydrothermal Alteration

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Synonyms

Alteration

Definition

Changes in the mineralogical, chemical, and textural properties of rocks due to the progressive and complex chemical and isotropic reaction between hydrothermal fluids and rocks.

Characteristics

Many rock forming minerals, such as plagioclase, orthoclase, quartz, biotite, muscovite, amphibole, pyroxene, olivine, etc., and volcanic glass are generally unstable during flow of hot geothermal fluids, characterized by a different chemical composition and a temperature ranging between 150 °C and 400 °C (Shanks 2012), through rock masses. Hydrothermal alteration represents the dissolution and then replacement of such primary rock minerals with new mineral assemblages called alteration minerals. In addition to replacement, hydrothermal alteration minerals can also be found directly as infilling materials in vesicles, vugs, veins, and fractures of rock masses. Quartz, chalcedony, opal, amorphous silica, clay minerals (illite, smectite, chlorite, kaolinite), zeolites, sericite, serpentine, albite, epidote, pyrite, calcite, talc, pyrophyllite, anhydrite, barite, alunite, jarosite, magnetite, hematite, and goethite are the most common alteration minerals found within matrix-intact rock and fractures of rock masses. Alteration minerals can be useful in many engineering studies for different purposes such as geothermometry, indicators to predict the permeability of original rock mass, and to understand the characteristics of geothermal reservoirs. The chemical reaction involving the replacement of olivine with serpentine is given below as a typical example of hydrothermal alteration.

$$\begin{aligned}&2Mg_2SiO_4\ (\text{olivine}) + H_2O + 2H^+\\&\quad = Mg_3Si_2O_5(OH)_4\ (\text{serpentine}) + Mg^{2+}\end{aligned}$$

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The factors affecting hydrothermal alteration are temperature, initial rock composition, fluid composition, activity and chemical potential of the fluid components (Pirajno 1992) as well as discontinuities and density and so permeability of rock masses, and pressure. As stated by Browne and Ellis (1970), the duration of hydrothermal process also controls the distribution and magnitude of hydrothermal alteration. Based on laws of thermodynamics, the types of hydrothermal mineral depend on the temperature, pressure, and chemical composition of geothermal system (D'Amore and Arnórsson 2000).

Hydrothermal alteration should not be confused with weathering. Both change physical, chemical, mineralogical, and eventually strength-deformation properties of geomaterials. In some rocks, it is rather challenging to distinguish hydrothermal alteration from weathering in field conditions. In the case of such a difficulty, it should be considered that the effects of weathering decreases with increasing depth and then completely disappears after certain depth, whereas the distribution and magnitude of hydrothermal alteration increase with increasing depth until reaching the bottom boundary of geothermal reservoir. In field investigation, alteration intensity is qualitatively described as "weak" (incipient), "moderate" (patchy), and "strong" (pervasive) (Shanks 2012). Whereas the term incipient alteration is used to describe clearly observation of original textures and partially altered phenocrysts, pervasive alteration indicates the replacement of significant proportion of initial minerals with alteration minerals. In addition to qualitative classification to determine styles of alteration, it is also possible to measure alteration intensity by quantitative methodologies (Large et al. 2001) based on elemental gains and losses (Shanks 2012).

Cross-References

► Alteration
► Rock Properties

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International Association of Engineering Geology and the Environment (IAEG)

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Definition

IAEG is an international organization of engineering geologists formed in 1964 to unite national groups of scientists and professionals in the developing field of engineering geology through collaboration, communication, and exchange of knowledge and research in the discipline. The intent was to encourage research, training, and dissemination of knowledge in applied geology. Engineering geology was a rapidly emerging field in geology in the 1960s as geologists became more involved with engineers in construction, mining, infrastructure development, land use planning, and natural hazards. At the IGC (International Geological Congress) meeting in New Delhi, India, in 1964, there were no sessions on applied geology. A group of engineering geologists at the meeting noted the lack of topical sessions and approached IGC with the idea of forming a commission in engineering geology. IGC dragged their feet on the formation of this commission, so these geologists then decided to form IAEG. The initial scope was the application of geology to engineering practice, but this expanded to embrace environmental concerns with a change of the name to International Association of Engineering Geologists and the Environment in 1997. First president was an Israeli engineering geologist, Asher Shadmon, and first executive secretary was Marcel Arnould from France. The structure of IAEG followed that of the previously established ISSM (International Society of Soil Mechanics) with membership through national groups. The organization of IAEG also followed the previously established engineering geology organizations in the United States (the first division of the Geologic Society of America, Engineering Geology, in 1947 and the Association of Engineering Geology in 1958).

By 2018, the organization comprised more than 4000 members in over 45 active national groups. The group was motivated initially to support the profession worldwide through congresses, regional meetings, publications, and awards. IAEG has had 12 congresses every 4 years since 1970 with the 13th in 2018 in San Francisco, USA. It has 37 subject commissions (about 17 are active at the time of writing) on topics such as landslides, geologic mapping, karst problems, soft soils, building stones, aggregates, marine engineering geology, rock slope stability, and collapsible soils.

An important part of the organization is the quarterly Bulletin of the International Association of Engineering Geology and the Environment published continuously since 1970. Originally published by IAEG, it is now published by Springer and edited by the IAEG.

A very active website (www.iaeg.info) has all of the workings of the organization (news, commission reports, videotaped lectures, statutes, and by-laws) and for members only, a section on all of the members and their specialties. There are also functioning committees to help the organization "work" such as the finance, enterprise, fees, awards, and outreach committees. Newsletters are sent to all members at least four times a year electronically, but starting in January 2018, there will be a bi-weekly electronic newsletter sent to all members around the world called the "IAEG Connector." It will contain IAEG news, national group news, news items in geology and engineering geology, and communications from the commissions and IAEG to its members.

Awards are an important part of the organization's mission. Four major awards are given: Hans Cloos Medal to an eminent engineering geologist internationally (every 2 years since 1977), Richard Wolters Award to an outstanding young (<35 years old) member (every 2 years since 1988), Marcel Arnould Award to an outstanding engineering

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geologist who has also made major contributions to the IAEG (every 2 years since 2014), and Honorary Member who has had an outstanding career in engineering geology (only 6 since 1992).

Cross-References

- ▶ International Association of Hydrogeologists (IAH)
- ▶ International Society for Rock Mechanics (ISRM)
- ▶ International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

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Igneous Rocks

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Synonyms

Magmatic rocks

Definition

Rocks resulting from the solidification of molten or partially molten material, called magma, which is generated inside the Earth's crust.

Introduction

Igneous rocks are classified into two types according to the settings in which they were formed:

– Plutonic or intrusive: formed deep inside the Earth's crust by the slow cooling and solidification of magma, which results in crystalline materials that are usually coarse grained, such as granite, gabbro, syenite, and diorite. As they rise to the upper crust, they can fragment and incorporate blocks of the host rocks, called xenoliths.
– Volcanic or extrusive: formed at the Earth's surface, around volcanic vents, by the ejection of lava, which may be explosive or not. The cooling is usually too rapid for the formation of coarse-grained mineral crystals, and glassy or fine-grained crystalline materials result; examples include rhyolites and basalts.

Another type of volcanic rocks are pyroclastic rocks, which originate from the accumulation and the subsequent compaction and cementation of fragments of crystal, glass, or rocks ejected from a volcano. Despite their igneous origin, pyroclastic rocks are predominantly classified in a similar way to sedimentary rocks, based mainly on the size of the constituting fragments.

In general, unweathered igneous rocks exhibit high mechanical strength due to the relative structural homogeneity and the strong cohesion of the mineral constituents.

For engineering geology purposes, the smaller grain size usually corresponds to the greater mechanical strength of volcanic rocks relative to plutonic rocks, mainly due to their better mineral imbrication and cohesion.

Although compact volcanic rocks tend to greater mechanical resistance than plutonic rocks, the presence of vesicles, amygdales, and columnar jointing can reduce their strength.

Larger proportions of quartz in some types of igneous rocks generally confer greater mechanical strength. On the other hand, this also generally contributes to increased abrasiveness, which leads to increased wear on equipment (drills, crushers, diamond saws, etc.).

Strong igneous rocks have the best technological characteristics for use in construction, and some are also important industrial raw materials.

Composition

The magma from which igneous rocks are formed consists mainly of silicon and oxygen, and its viscosity is directly proportional to the content of silica (SiO_2). Thus, the constituent minerals of igneous rocks are essentially silicates that are forming as the temperature of the magma reaches their crystallization conditions.

In general, the first minerals to crystallize are iron and magnesium silicates, called mafic or ferromagnesian minerals (generally dark in color), whereas as temperature falls, the last are potassium aluminosilicates, muscovite, and quartz, which are called felsic minerals (generally light in color). Accessory minerals, such as zircon, apatite, and titanite, are the first to crystallize.

The crystallization sequence is represented by two series, according to N.L. Bowen (cited in Klein and Dutrow 2008), which converge on the crystallization of potassium feldspars, mica (muscovite), and quartz:

- Discontinuous series: olivine, pyroxenes (augite), amphiboles (hornblende), and micas (biotite)
- Continuous series: calcic plagioclases followed by sodic plagioclases

Due to higher temperature and pressure crystallization conditions, the ferromagnesian minerals tend to be less stable under shallow crustal and Earth surface conditions and may be altered, in terms of chemical composition and crystal structure, by an interaction with late-stage magmatic liquid (richer in volatiles and/or siliceous materials) or by an exposure to the atmospheric elements (weathering). In the latter case, there is a formation of secondary minerals, such as iron oxides and hydroxides, and clay minerals.

Main Forms of Occurrence

The main forms of occurrence of igneous rocks in the Earth's crust are listed below.

- Batholith: large-volume igneous mass with irregular contours and a domical top.
- Stock: plutonic igneous mass of smaller volume, generally vertical, almost cylindrical bodies.
- Dike: result of rising magma-filled fractures in crustal rocks. The thickness of a dike can range from centimeters to hundreds of meters.
- Sill: an igneous body of tabular format that is concordant in relation to bedded host rocks. A sill is a layer of notable uniformity and thickness due to the intrusion of magma into the bedding planes of sedimentary deposits.

With regard to lava flows, volcanic activity can occur in two ways:

- Central eruptions: these generally form a cone on the surface, connected with the volcanic conduit through which lava, gases, and pyroclastic materials are ejected.
- Fissure eruptions: in these, lava escapes through a network of fractures in the Earth's surface, generally extending through large areas.

Structures and Textures of Igneous Rocks

The structural and textural aspects of igneous rocks frequently overlap, so for clarity in the present chapter, structure refers to the meso and macroscopic features of rock that are more easily observed in the field, and texture refers to microscopic aspects, such as the size (granularity) and shape (euhedral, subhedral and anhedral) of mineral crystals or grains and the interrelations between them and with any glass or other materials present.

Structures

Igneous rocks are usually massive in structure, but some have fluidal, vesicular, or columnar structure.

- Massive: minerals exhibit no preferential orientation along specific directions. Both in hand samples and outcrops, they have the appearance of a compact rocky mass. In the case of plutonic rocks, they may have vertical and subhorizontal fracturing systems, which arise after magma solidification and favor the breaking of the rock into blocks.
- Fluidal: minerals exhibit iso-orientation as an expression of the directional movement of the magma during its emplacement and prior to its complete cooling. They are commonly observed in the margins of intrusions or dikes, near the walls of the host rocks.
- Vesicular: volcanic rocks may contain a circular, elliptical, or irregularly shaped cavities resulting from the expansion of gases in the lava while it cools, giving the rock a vesicular structure. Vesicles tend to be concentrated in the upper portion of the flow due to the tendency of the volatiles to rise. In a subsequent stage, these cavities may be filled with secondary minerals or with deuteric minerals arising from the interaction of preexisting minerals with late-stage magmatic solutions, such as quartz (which can form geodes), calcite, zeolites, chalcedony, and chlorite, in which case they are described as amygdaloidal structure.

The term columnar refers to the structure provided by the disposition of the volcanic rock in five or six-sided columnar prisms as a result of the lava contracting during its cooling (Fig. 1).

Textures

Plutonic rocks exhibit variable grain size, usually distinguishable to the naked eye, generating a phaneritic texture (Fig. 2).

Volcanic rocks are so very fine-grained that grains are not distinguishable to the naked eye, which is called an aphanitic texture. If the lava cools very rapidly, crystalline minerals do not form, and the result is volcanic glass and a vitreous texture.

When one mineral is conspicuously larger and stands out in the matrix, this is called a porphyritic texture.

Igneous Rocks, Fig. 1 Columnar jointing in basalt rocks of Staffa Island, Scotland

Igneous Rocks, Fig. 2 Granitic rock (biotite syenogranite) showing massive structure and phaneritic texture (*bottom left*)

Classification

Igneous rock classification is based in two main features: the modal mineralogy and grain size, which is also a criterion to distinguish volcanic from plutonic rocks even though there is no specific grain size set for this.

Exceptions are made for glassy or very fine-grained rocks (Shelley 1992) that may be classified on their chemical composition by using Total Alkali Silica (called TAS) diagrams.

The most widely adopted classification of igneous rocks is based on the recommendation of International Union of Geological Sciences (IUGS) in which relative proportions of the essential mineral are plotted in triangular diagrams for each different group of rock – e.g., plutonic, volcanic, or ultramafic. These give the root names such as granite, syenite, basalt, rhyolite, etc. (Le Maitre 2003).

For the classification of acidic to basic igneous rocks, there are considered the following groups of minerals: QAP (Q (quartz), A (alkali feldspar, including albite up to 5%), and P (plagioclase)) and PAF, where F is feldspathoids or "foids" (including nepheline, leucite, sodalite, and cancrinite).

Ultrabasic and ultramafic rocks are classified in the content of orthopyroxene, clinopyroxene, hornblende, plagioclase, and olivine (Le Maitre 2003). Other igneous rocks, subjected to specific classifications, are carbonatites, melilitic rocks, lamprophyres, etc.

Charnockitic rocks constitute a special group of plutonic rocks that resembles granitic rocks but are characterized by the presence of the orthopyroxene (En_{50-70}) and perthitic feldspar. They may be named by adding the qualifier orthopyroxene to the QAP general classification or by adopting some special names as charnockite (orthopyroxene granite) or enderbite (orthopyroxene tonalite).

Pyroclastic rocks are usually named according to the size of the fragments (or clasts) ejected from the volcano (Table 1).

Some Common Igneous Rocks

There is a wide variety of igneous rocks, but for engineering geology, the most common are included in Table 2. Their main characteristics and formation processes may be found in Hall (1996), Best and Chistiansen (2001), Philpotts and Ague (2009), Gill (2010) and Klein and Philpotts (2017) among other.

Granites are acidic plutonic rocks composed of feldspar (K-feldspar, generally microcline and plagioclase, generally oligoclase, making up 50–70%), quartz (20–30%), and ferromagnesian minerals, mainly biotite and hornblende (5–25%). The accessory minerals are magnetite, titanite, zircon, apatite, and sometimes garnet. The textural arrangement is granular or, less commonly, porphyritic.

Depending on the relative contents of quartz and feldspars, rocks can be classified as granodiorites, which have a predominance of plagioclase (65–90%) over the alkali feldspars and higher content of mafic minerals, or tonalites, wherein the plagioclase amounts to 90% to 100% of the feldspars.

Syenites are intermediate plutonic rocks, also called alkaline rocks due to the high content of alkali elements (K and Na) in the composition of the essential minerals. K-feldspar is the main component, and the most common mafic minerals are alkali silicates (pyroxenes and amphiboles), with associated biotite and opaque minerals, such as magnetite.

In the absence of quartz, feldspathoids (nepheline, sodalite, and others) may occur, constituting the nepheline/sodalite syenites.

Weathering may alter these rocks into a clayey material (mainly kaolinite), which through the action of leaching can result in bauxite deposits.

Rhyolites are the volcanic equivalents of granites. Varieties of rhyolites are felsite, granophyre, vitrophyre, and pumice (used as an abrasive and polishing agent).

Trachyte and phonolite are the volcanic equivalents of syenite and feldspathoid syenite, respectively.

Dikes or veins may be found in the margins and interiors of granitic plutons, as a result of the filling of fractures in the newly consolidated rock by other igneous rocks crystallized from the residual magma. These have the following names:

– Pegmatite: when showing very coarse granularity. It is composed of quartz, alkali feldspar, and muscovite, usually accompanied by rare minerals that are rich in lithium, beryllium, niobium, and rare earths. It may contain mineral species of economic interest, especially for jewelry.
– Aplite: when fine grained and containing mainly quartz and alkali feldspar.

Diorite is an intermediate plutonic rock that consists predominantly of plagioclase and mafic minerals such as biotite, hornblende and/or pyroxenes, and opaque minerals (magnetite). Its black color makes it widely used as an ornamental rock, especially in funerary art. Weathering results in a clayey material rich in iron oxides and hydroxides, which give it a reddish or yellow-orange coloration.

Igneous Rocks, Table 1 Pyroclastic rocks classification

Fragment size (mm)	Fragment designation	Rock designation
>64	Bomb (partial to totally molten)	Agglomerate
	Block (if not molten)	Volcanic breccia
64–2	Lapilli	Lapilli tuff
<2	Ash	Tuff

Igneous Rocks, Table 2 Main mineralogy and colors of some common igneous rocks

Essential minerals Rock classification	Qtz, Pl, Kfs (Bt/Hbl)	Kfs (Bt/Hbl) (Aeg) (*Ne/Sdl*)	Pl, Bt, Hbl (Qtz ± Kfs)	Pl, Aug, Op	Ol ± Px (Mag)
Plutonic	Granite	Syenite *Nepheline syenite*	Diorite	Gabbro	Dunite/Peridotite/Pyroxenite
Volcanic	Rhyolite	Trachyte/ *Phonolite*	Andesite	Basalt	–
Colors	Grey to reddish pink	Pink to reddish brown/grey to dark green	Dark grey/greenish brown	Dark grey to black	Black to greenish black
Chemical classification (SiO_2 content)	>63% (acidic)	52–63% (intermediate)		45–52% (basic)	<45% (ultrabasic)

Abbreviations: *Qtz* quartz, *Pl* plagioclase, *Kfs* K-feldspar, *Bt* biotite, *Hbl* hornblende, *Aeg* aegirine, *Ne* nepheline, *Sdl* sodalite, *Aug* augite, *Op* opaque minerals, *Ol* olivine, *Px* pyroxene, *Mag* magnetite

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Igneous Rocks, Fig. 3 Dark-colored gabbroic rock extracted as building stone

Andesite is the equivalent volcanic rock to diorite, usually consisting mainly of plagioclase (andesine) and amphibole.

Gabbro (Fig. 3) is a basic plutonic rock with a granular texture that also consists of calcic plagioclase (labradorite), augite, and opaque minerals (magnetite and/or ilmenite). Olivine or orthopyroxenes may occur in small amounts (up to 10%).

Diabase has a similar composition to gabbro, but with a finer texture. It occurs in dikes and, less commonly, sills.

Basalt is the equivalent volcanic rock to gabbro, and the mineralogy also consists mainly of calcic plagioclase (labradorite, up to 50%), clinopyroxene (augite, up to 40%), magnetite or ilmenite, and very variable amounts of glass. Its color is dark grey to black, with reddish or brownish tones conferred by iron oxides/hydroxides generated by weathering. It is widely used as crushed stone, in aggregates for asphalt and concrete, as railroad track ballast, and as rock fill.

In the some part of basalt rocks, the glassy material has been devitrified, that is, transformed into clay minerals (especially of the montmorillonite group, minerals that are expansive in water). Their presence promotes the rapid disintegration of the rock when exposed to moisture (rain) and drying (drought).

Peridotite, pyroxenite, and dunite are ultramafic igneous rocks composed of different proportions of olivine, pyroxene, and amphibole (see Le Maitre 2003). In peridotites and dunites, olivine – the essential constituent – is frequently altered, along fractures, into serpentine and, less commonly, talc.

Acidic rocks are highly resistant to alteration under normal conditions of use, even in an aqueous environment. On the other hand, basic and ultrabasic rocks tend to be altered when exposed to atmospheric conditions, which in moisture-rich environments or tropical climates may lead to mechanical disintegration or decomposition into clay minerals that are often expansive.

So, in spite of being more easily excavated and having physical and mechanical properties similar to those of granites, the relatively low resistance to alteration of the constituent ferromagnesian minerals is a matter for special attention in the major engineering projects.

Igneous Rocks in Engineering Geology

The abundance and good physical and mechanical properties (isotropy, mineral cohesion, low porosity, etc.) of igneous rocks, when not fractured or extensively weathered, favor their use in civil works as foundations, crushed rock and as building stone. Their appearance also makes them highly valued for use as slabs for covering floors, walls, and facades and as finished or semifinished pieces like countertops, wash basins, etc.

However, before actually using igneous rocks, care must be taken to perform geological and geotechnical fieldworks on the rock mass in order to determine and quantify discontinuities such as fracture, fault, and other features that could constitute areas of weakness or percolation/loss of water (see the IAEG recommendations in Matula (1981)).

It is also necessary to perform laboratory physical and mechanical determinations and petrographic analyses in order to check the kind and degree of mineral alteration as well as the presence of microdiscontinuities and types of filling materials.

When using igneous rocks as ornamental stones or as aggregates, the petrographic features are particularly important to check for the presence of unstable, altered, or potentially deleterious minerals that could interfere with their esthetic appearance and durability, unless preventive measures are adopted when they are used.

Summary

Igneous rocks result from solidification of molten or partially molten material (magma) generated inside the Earth's crust. According to their formation conditions, they are distinguished in two types, plutonic or intrusive, when formed deep inside the crust by the slow cooling and solidification of magma, resulting in crystalline materials, and volcanic or extrusive, when formed at the Earth's surface either by flow of lava, resulting in glassy or fine-grained materials due to the rapid cooling, or by fragments of crystal, glass, or rocks explosively ejected from a volcano – the pyroclastic rocks. Plutonic and volcanic rocks are classified in terms of their grain size and the predominant mineral components. Pyroclastic rocks are predominantly classified in the size of the constituting fragments.

Unweathered igneous rocks usually exhibit high mechanical strength. For engineering geology purposes, smaller grain sizes usually correspond to greater mechanical strength, although the presence of discontinuities as cavities, joints, faults, as well as mineral alterations can reduce strength. Higher proportions of quartz that generally confer greater mechanical strength may also contribute to increased abrasiveness and wear on equipment.

Good physical and mechanical properties (isotropy, mineral cohesion, low porosity, etc.) of igneous rocks, when not fractured or extensively weathered, favor their use in civil works as foundations, as crushed rock, and as building stones. They are also highly valued facing, flooring, and other decorative uses.

For proper use, it is recommended that careful fieldwork should be undertaken to determine and quantify discontinuities such as fractures, faults, and other features that could constitute areas of weakness, as well as laboratory physical and mechanical determinations and petrographic analyses to check, for example, for deleterious minerals, types and degree of mineral alteration, microdiscontinuities, and filling materials.

Cross-References

▶ Aggregate
▶ Building Stone
▶ Crushed Rock
▶ Volcanic Environments

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Inclinometer

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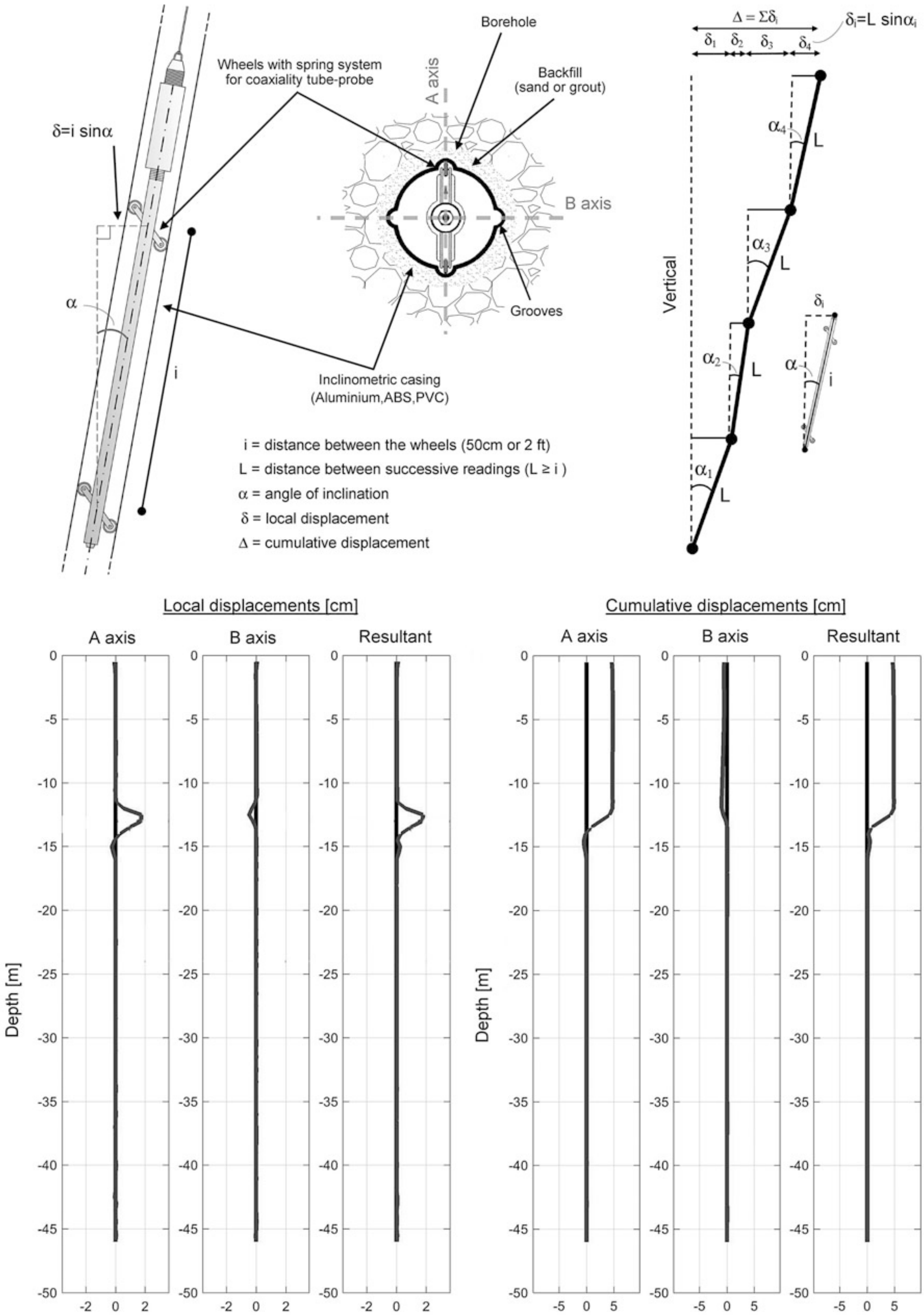
Definition

Inclinometer measurements are widely used for monitoring of landslides, retaining walls, piles, and in all the contexts where is necessary to measure deep-seated ground deformations. This value is calculated indirectly by using the difference of the inclination of an inclinometer casing installed on a borehole.

Characteristics

The horizontal deformation is calculated analyzing the difference of inclination over the time, obtained with the measurements at predefined depths. The measurement is done using a dedicated probe, lowered to the bottom of the inclinometer tube. The probe holds two sensors for measuring the inclination of the tube in two orthogonal planes. This allows the calculation of the displacement vector of each analyzed point. The probe is kept coaxial to the tube by



Inclinometer, Fig. 1 Principle of inclinometer measurement and typical results

Inclinometer, Table 1 Comparison between the main types of inclinometers systems

Type	Measurements steps	Reading interval	Double redundant reading (opposite measurements 0–180°)	Probe positioning in the borehole	Connection between probe and readout unit	Global accuracy	Complete measure of all the borehole	Usable after large tube deformations
Probe inclinometers for manual operation (with technician)	Typical (50 cm–2 ft) or (100 cm–4 ft)	Periodic depending on the goals of the monitoring	Yes. It is also possible fourfold reading (0–180°) and (90–270°)	Probe positioned at the measurement depth using a graduated electrical cable	Calibrated electrical cable for reading and probe support	High, but related to the expertise of the operators	Yes	Yes
Automated inclinometer system	Custom (usually 50 cm)	Custom. (usually 1÷4 meas/day)	Yes	Probe automatically positioned at the measurement depth using electric motor and high precision encoder	Kevlar® cable only for probe support. Electronic device into the probe for data reading and transmission	High	Yes	Yes. Electronic control system identifies tube locking problems to preserve the probe integrity
In-place inclinometers	Custom (usually 1 fixed probe/ 100 cm) but not always for all the length of the borehole	Custom. (usually 1÷24 meas/day)	No	Probe permanently positioned at the measurement depth	Electrical cable for data reading and steel cable for probe support	High	Yes, but in most cases the installation of the sensors in some part of the borehole was chosen	Not always due to tube locking

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means of wheels running inside the grooves that characterize the inclinometer tube (Fig. 1). In order to minimize the influence of the bias-shift effect and to identify coarse measurement errors, a redundant reading technique is usually used: during the same measurement session, the probe measures the inclination of the same points twice, with opposite orientation (Dunnicliff 1988). The inclinometer measurements can be done manually by a technician or fully automatically by different in-place complex instruments. The possible solutions for in-place automatic measurement are the Automated Inclinometer System (Lollino et al. 1992, 2006) or in-place inclinometer columns (e.g., DMS©, ShapeAccelArray©). The AIS and the in-place system are equipped with automatic management and remote control, which allow a complete and frequent monitoring of the deep-seated ground deformation. Table 1 presents the main characteristics of these systems. The errors in the inclinometer measurements are not simple to calculate, but recent studies estimate a value of about 2 mm for every 10 m of tube (Mikkelsen 2003). The automatic measurements with AIS or the in-place inclinometer generally provide better results due to the automation of the systems and to the permanence of the systems on site.

Cross-References

- ▶ Borehole Investigations
- ▶ Boreholes
- ▶ Casing
- ▶ Deformation
- ▶ Drilling
- ▶ Extensometer
- ▶ Instrumentation
- ▶ Landslide
- ▶ Mass Movement
- ▶ Monitoring
- ▶ Tiltmeter

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Induced Seismicity

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Synonyms

Anthropogenic seismicity; Stimulated seismicity; Triggered seismicity

Definition

Induced seismicity refers to earthquakes and seismic events resulting from human activity (Lamontagne 2013). This often involves critically stressed faults which subsequently experience an increase in stress, for example, due to stress redistributions during mining or loading due to the added weight of a dam reservoir. Alternatively, induced seismicity may be triggered if a critically stressed fault experiences a decrease in effective stress due to fluid injection, for example, during waste water injection or hydraulic fracturing for geothermal or hydrocarbon energy extraction.

Characteristics and Experiences

It is well established that earthquakes can be induced by stress changes or fluid injection. Underground mining alters the *in situ* stress state present at depth. Where faults intersect or are adjacent to an excavation, this can cause both an increase in shear driving stress and a reduction in the normal stress clamping the structure (Blake and Hedley 2003). The resulting energy release if fault slip occurs can be sudden and violent, presenting a significant safety risk to mine personnel. Blake and Hedley (2003) report North American experiences with mining-induced seismicity involving local magnitudes of up to M_L 5.2. Grobbelaar et al. (2017) report a recent M_L 5.5 event as the largest mining-induced earthquake in South Africa.

Gupta (2002) reports more than fourteen cases where impounding of a dam reservoir has triggered seismic events greater than M_L 5.0, with four of these exceeding M_L 6.0. The Koyna reservoir-triggered earthquake in India represents the largest of these (M_L 6.3) and resulted in more than 200 fatalities. Bell and Nur (1978) summarize the main effects of reservoir impoundment with respect to triggering induced seismicity as including the stress increase due to the weight of water that follows the filling of the reservoir (up to 1 MPa for some of the deepest reservoirs), and the increase in pore fluid pressure due to fluid migration and compaction of the rock pore space in response to the stress increase.

Induced seismicity due to fluid injection has been recognized since the 1960s when Healy et al. (1968) presented statistical evidence correlating fluid injection in waste water wells to induced seismicity. Since then, numerous studies have studied induced seismicity related to waste water injection and fluid injection for hydraulic fracturing to increase rock mass permeability for Enhanced Geothermal Systems (EGS) and unconventional oil and gas reservoirs. Fluid injection can trigger earthquakes through increases in pore pressure (and a consequential decrease in effective normal stress) in the vicinity of critically stressed faults. The increased pore pressure reduces the effective stresses and shear resistance to fault slip, allowing elastic energy already stored in the surrounding rocks to be released. Majer et al. (2007) report that most induced seismicity events related to EGS projects have been less than M_L 3.0, but that events up to M_L 4.6 have been recorded, with the largest occurring at The Geysers field in Northern California in the 1980s when fluid production was at its peak. Induced seismicity has contributed to the cancelation of the Basel Deep Heat Mining EGS project in Switzerland. McGarr (2014) reports several induced seismicity events larger than M_L 4.0 associated with oil and gas production activities. This includes a maximum M_L 5.7 event associated with wastewater injection in 2011 in Prague, Oklahoma, although a natural origin cannot be ruled out. Notable events associated with hydraulic fracturing operations targeting unconventional oil and gas reservoirs include a M_L 4.6 event in 2015 in the Montney play in northeastern British Columbia (Babaie Mahani et al. 2017).

Cross-References

- ▶ Dams
- ▶ Dewatering
- ▶ Earthquake
- ▶ Effective Stress
- ▶ Faults
- ▶ Fluid Withdrawal
- ▶ Ground Shaking
- ▶ Hazard
- ▶ Hydraulic Fracturing
- ▶ Mining
- ▶ Mining Hazards
- ▶ Normal Stress
- ▶ Pore Pressure
- ▶ Reservoirs
- ▶ Stress

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Infiltration

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Synonyms

Penetrate; Percolation; Permeate

Definition

Infiltration is defined as the entry of water from the surface into the subsurface.

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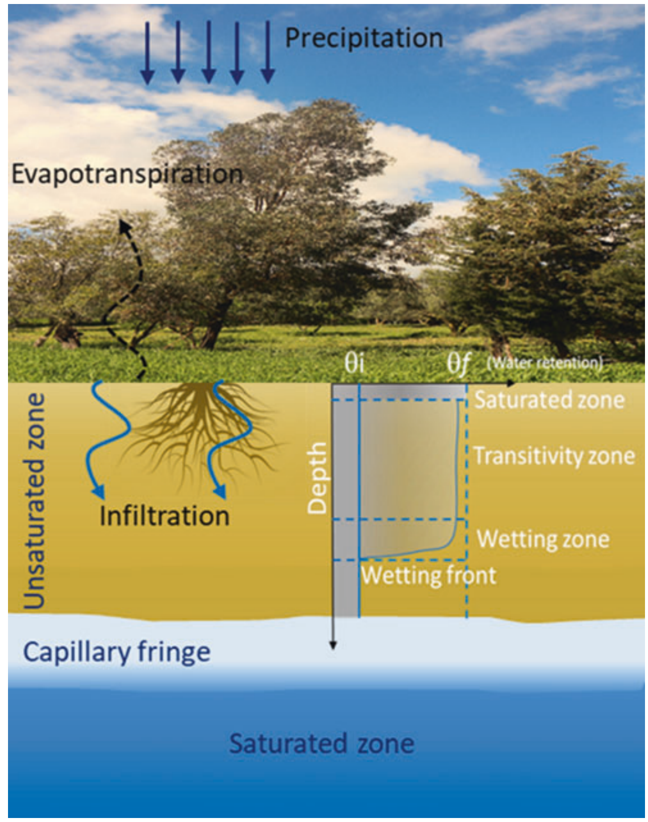
Introduction

Infiltrated water may originate from rainfall; irrigation; water bodies such as ponds, rivers, and lakes; or other anthropogenic activities. The terms infiltration and percolation are frequently interchangeably used even though they represent two different processes.

Infiltration describes the entry from the surface to the subsurface, whereas percolation focuses on the flow of water through soil and porous media. The amount of water percolation that reaches the groundwater represents the groundwater recharge. When the soil surface is exposed to rainfall or submersion, infiltrated water fills the interstices between soil grains of the upper layers of the soil. The upper part soil profile may contain a saturated horizon that extends a few millimeters in depth. Water continues to penetrate into the subsurface forming a transmission zone. The water content in this zone varies with depth, and the water flow is essentially vertical and driven by gravitational forces. For the wetting zone, located below the transmission zone, the water content decreases sharply to reach the initial water content of the soil. The limit between dry and wet compartments of the soil profile is called the wetting front (Fig. 1). It is characterized by a steep hydraulic gradient and presents a sharp limit between dry and wet horizons of soil. Over the infiltration process, the wetting front progresses vertically, and the transmission zone expands (Hendriks 2010). Infiltration rate is variable with time. It demonstrates a steep decline from the beginning of the infiltration episode and reaches a steady state when the soil becomes saturated. At this stage, the infiltration rate is approximately equal to percolation rate. Infiltration rate depends also on infiltration capacity of each soil type expressed in mm/h and the presence of macropores (Wilson 1990).

Soil water infiltration is controlled by soil physical properties, slope, vegetation, surface roughness, and the rate and duration of water application. Infiltration capacity is commonly determined by hydrograph analysis and infiltrometer experiments. Infiltrometers are classified into two types: rainfall simulators and flooding devices. Rainfall simulators produce drops of water falling onto the soil surface at a measured rate providing a method of investigating the infiltration process. They are used to give assessment of the time needed to ponding when a uniform rainfall rate occurs on the soil surface. The rainfall can be simulated by sprinkling water from an array of capillaries or by using a rotating swirl plate device. Flooding devices are typically rings or tubes inserted in the soil. The rings infiltrate water at suction; then the water volume is converted to depth of water infiltrated by subtracting the starting volume.



Infiltration, Fig. 1 Schematic figure showing the process of infiltration and relevant soil zones

Infiltration rate calculation is an important issue for rainfall/runoff models, estimation of groundwater natural recharge, managed aquifer recharge, and irrigation and drainage project design. Infiltration models are divided into three categories (Mishra et al. 2003): (1) physically based models, derived from the combination of Darcy's law (Darcy 1856) and the mass conservation equation (e.g., Green and Ampt 1911; Richards 1931; Philip 1957), (2) semiempirical models based on continuity equation and simple hypotheses on the relation between the infiltration rate and cumulative infiltration (e.g., Horton 1938; Holtan 1961; Overton 1964; Singh and Yu 1990), and (3) empirical models based on field and laboratory experiment data (e.g., SCS-CN 1972; Kostiakov 1932; Huggins and Monke 1968; Gargouri-Ellouze and Eslamian 2014).

Cross-References

▶ Clay
▶ Conductivity
▶ Desiccation

▶ Dewatering
▶ Dilatancy
▶ Dispersivity
▶ Equipotential Lines
▶ Fluid Withdrawal
▶ Fluidization
▶ Groundwater
▶ Hydraulic Action
▶ Hydraulic Fracturing
▶ Hydrocompaction
▶ Hydrogeology
▶ Hydrology
▶ Percolation
▶ Piezometer
▶ Pore Pressure
▶ Run-Off
▶ Saturation
▶ Voids
▶ Water

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Infrastructure

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Definition

The structures and facilities that, together, constitute the essential basis for running a society and economy.

Context

Efficient and well-maintained infrastructure is the essential basis for a modern society and economic growth (Hamada et al. 2015; Hayes, 2006; Smets and Shannon, 2009). It consists of facilities relating to:

Energy – Power stations; renewable energy facilities (wind, solar, tidal, and hydroelectric); oil, gas, and coal abstraction/extraction facilities; and distribution supply systems (power lines and grids)

Water – Abstraction and treatment facilities, storage facilities (dams, reservoirs), pipelines, aqueducts, and recharge schemes

Waste – Solid and liquid waste collection, treatment and management facilities and sewage collection, treatment and disposal facilities

Transport – Roads, railways, tramways, airports, ports and harbors, bridges, tunnels, and pipelines

Communications – Television, radio, telephone, and internet facilities

Protection from hazards – River and coast defenses against flooding and protection against other types of natural hazards (e.g., avalanche sheds, refuge structures from typhoon flooding)

Defense facilities – Military installations

Some matters are not usually identified as infrastructure but are of significant importance such as facilities for storage

Infrastructure, Fig. 1 Early stage in construction of an elevated section of a coast road near Durban, South Africa (Photograph by the author)

Infrastructure, Fig. 2 A dam – a key infrastructure element in water storage and power supply, in KwaZulu Natal, South Africa (Photograph by the author) (e.g., warehousing) and mineral workings that are essential in underpinning construction and manufacturing.

Whereas much infrastructure is concentrated in urban areas, resources are largely exploited in the wider landscape, and infrastructure provides the essential connectivity nationally and internationally. Effective infrastructure depends on high levels of both public and private investment and medium to long-term planning.

Engineering geologists have important roles in all of these through ground investigation, sampling, and testing; identification of mineral and water resources; design of extraction, operations, and constructions; and identification, mitigation, and remediation of natural and anthropogenic hazards (Figs. 1 and 2).

Cross-References

- ▶ Bridges
- ▶ Dams
- ▶ Levees
- ▶ Nearshore Structures
- ▶ Reservoirs
- ▶ Tunnels
- ▶ Waste Management

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InSAR

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Synonyms

Radar interferometry; Synthetic aperture radar interferometry

Definition

SAR (Synthetic Aperture Radar) is a form of radar that is used to create images of objects, such as landscapes or other static targets (Massonet and Feigl 1998). SAR is a form of active remote sensing and can be used regardless of the atmospheric conditions (clouds or night) where the antenna transmits radiation that is reflected from the surface targets (or scatterers) within each SAR resolution cell and detected by the sensor (Ferretti et al. 2007). *Interferometry* is a family of techniques in which usually electromagnetic waves, acquired at different time, are superimposed in order to extract information about the waves (Bunch and Hellemans 2004). When using radar waves, the technique is called *SAR Interferomery* or *InSAR*.

Characteristics

InSAR represents a part of aerial photography and satellite imagery that is used by skilled terrain analysts and interpreters to identify (remote sense) Earth processes, environments, landforms, and materials, and to use the information to map and evaluate the physical and cultural terrain characteristics and conditions that adversely affect people and property (Mollard, 2013). By transmitting and receiving radar waves, the sensor detects two perpendicular oscillations: (i) the phase that explains exact distance to the surface targets in each resolution cell and (ii) the amplitude that represents the radiation intensity of the signal. Whereas the result of the first is an image of relative distances between the sensor and the targets, the second is the result of the target roughness and results in a grey-scale backscattered signal intensity image. In the resulted image, the bright pixels represent strong backscatter radiation (dominant scattering direction is towards the sensor) and are usually the result of rectangular shaped targets within the pixel (i.e., outcrops, buildings etc.), the dark pixels represent low backscatter radiation as the incidence waves are mirrored away (i.e., flat surfaces like calm water bodies), and the speckled patterns in the image represent pixels with high noise levels mostly related to the vegetation. A SAR interferogram is the difference of the interferometric phase values in a certain area, and it is a digital representation of change in surface characterization. The interferometric phase is a function of four factors: (i) topographic distortions, (ii) atmospheric effects, (iii) target displacement, and (iv) noise. These pose a challenge in SAR data processing that is usually performed by the InSAR data providers. The original values that range from $-\pi$ to $+\pi$ (as they correspond to phase variations) are converted to a map of elevation changes (TRE 2015).

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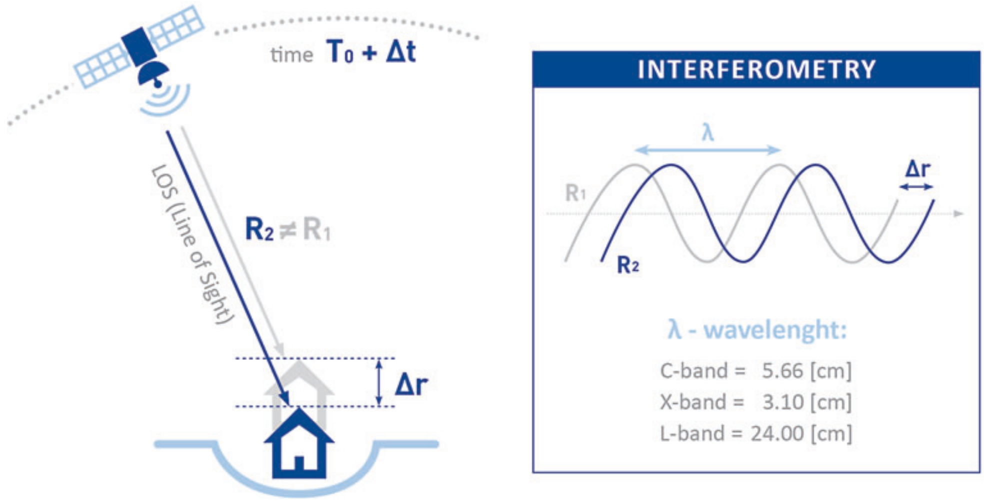
InSAR, Fig. 1 The basic principle behind the InSAR (TRE 2015)



The result of the phase difference between two images is extensively used in detecting slow ground displacements in the line-of-sight (LOS) of the radar signal (Fig. 1). InSAR is a good complementary tool to the traditional in situ tools (i.e., GPS measurements) (Pritchard 2006), as it provides a surface change data image. It can be used to measure variety of observables, such as topography, ground moisture changes, and surface deformations from glaciers, earthquakes, and volcanoes. Data obtained with InSAR, in combination with sophisticated location procedures, can help monitor nuclear tests, create better models of the elastic properties of Earth's interior, and assess seismic hazard (Pritchard 2006). In addition, InSAR is useful for monitoring surface deformations due to landslide activity, groundwater extraction and mining activities, pipeline deformations, and bad civil engineering practices.

From the end-user perspective, the limitations of the InSAR technique are related to (i) spatial limitations such as presence or absence of scatterers and the vegetation cover resulting in backscattered signal strength and (ii) observed phenomena dynamics where the displacement detection limit is defined by the half of the wavelength (i.e., for C-band with wavelength of 5.66 cm, which is 28 mm) and the frequency of SAR image acquisition of the same area (i.e., for ERS approximately 35 days).

Cross-References

- ▶ Aerial Photography
- ▶ Geohazards
- ▶ Laser Scanning
- ▶ Lidar
- ▶ Photogrammetry

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Instrumentation

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Synonyms

Construction performance; Geotechnical investigations; Monitoring; Risk; Site characterization

Definition

Measuring instruments that are used for *in situ* determination of soil and rock properties and instruments that monitor performance as part of a geotechnical investigation, study, or project (Dunnicliff 1993).

Introduction

Geotechnical instrumentation is essential for engineering projects that involve any degree of risk, be it to life, infrastructure, or financial in nature. The purpose of instrumentation is to provide a means to establish soil and rock properties and observe ground displacements due to natural processes such as landslide or construction activities. Selection and installation of monitoring instruments should be considered a project-specific task based on the anticipated ground movements and the ability of a given instrument to monitor a ground response.

Classification

Instruments can be classified into several different categories related to their intended application. Within each category are a number of different instruments that serve the same purpose, but the accuracy, resolution, and ability to automate will differentiate each type. Furthermore, as instrumentation is continually being researched and new applications or test methods developed, instruments can also be classified as research grade or conventional. Conventional instruments refer to tried and tested monitoring methods that are readily available to the practitioner, whereas research grade instruments are cutting-edge technologies that may require special installation techniques as well as advanced interpretation methods.

Limitations

This chapter refers to the type, application, and ability to monitor the desired ground response of an instrument and not the design or detailed electrical specifications of the various instruments. Furthermore, the installation and design intent is not discussed in detail herein. For these aspects, the reader should refer to Dunnicliff (1993).

Pore-Water Pressure/Groundwater Monitoring

Pore-water pressure measurements are essential in most geological engineering applications. These applications range from, but are not limited to, landslide hazards; embankment, tunnel, and supported excavation construction; and site dewatering or hydrogeological investigations. By far, pore-water pressure/groundwater level monitoring is the most commonly installed monitoring used today. The methods currently used are relatively inexpensive and easy to implement, readily interpretable, highly reliable, and are an essential minimum for site characterization.

As noted elsewhere for various instrument types, pore-water pressure/groundwater monitoring equipments are divided into systems that account for on-site hydrogeologic conditions. These conditions include whether the formation in question is considered to be a confined or unconfined aquifer or whether the pore-water pressure to be evaluated is located within the vadose zone. In addition to considering such factors, the application and formation of interest must be considered.

Observation Wells/Standpipe Piezometers

The presence of highly permeable formations such as sand, gravel, or fractured bedrock in either confined or unconfined aquifer scenarios permits the use of virtually any (positive) pore-water pressure method. These instruments can range from observation wells and open standpipes to sealed piezometers.

Open standpipes and observation wells are very similar in that they both consist of a well casing with a perforated zone installed into the formation of interest. Both are open to the atmosphere and operate under the premise that the water level in the piezometer will come to equilibrium with the surrounding groundwater level with time. The two differ in that an observation well is sealed at the ground surface to prevent the infiltration of surface water into the well pack, whereas a standpipe has the seal installed directly above the formation of interest. Standpipes therefore minimize the risk of interformation hydraulic communication, whereas observation wells do not. Figure 1 is a schematic illustrating an observation well and a standpipe piezometer.

The slotting of the open perforated zone can consist of either holes punched through the pipe or vertical, diagonal, or horizontal cut slots. Typically, most slots are machine cut to specified opening widths based on the formation into which they are to be installed as well as the desired use. Slot sizing is only done when the well is to be used for hydraulic testing and head losses through the screen must be minimized. Angled slots are typically only used during environmental investigations when light nonaqueous phase liquids (LNAPLs) are likely present and there is a risk that the contaminant may be caught between the solid portions of a slotted riser and therefore not sampled.

The open standpipe can have a ceramic porous stone as the perforated zone, as well as reduced diameter risers to the ground surface (Casagrande 1949). The purpose of both is to reduce the risk of fine migration and response time when installed into clay formations. The diameter of the riser is limited by the diameter of the method used to manually measure the water level within the standpipe. Installing

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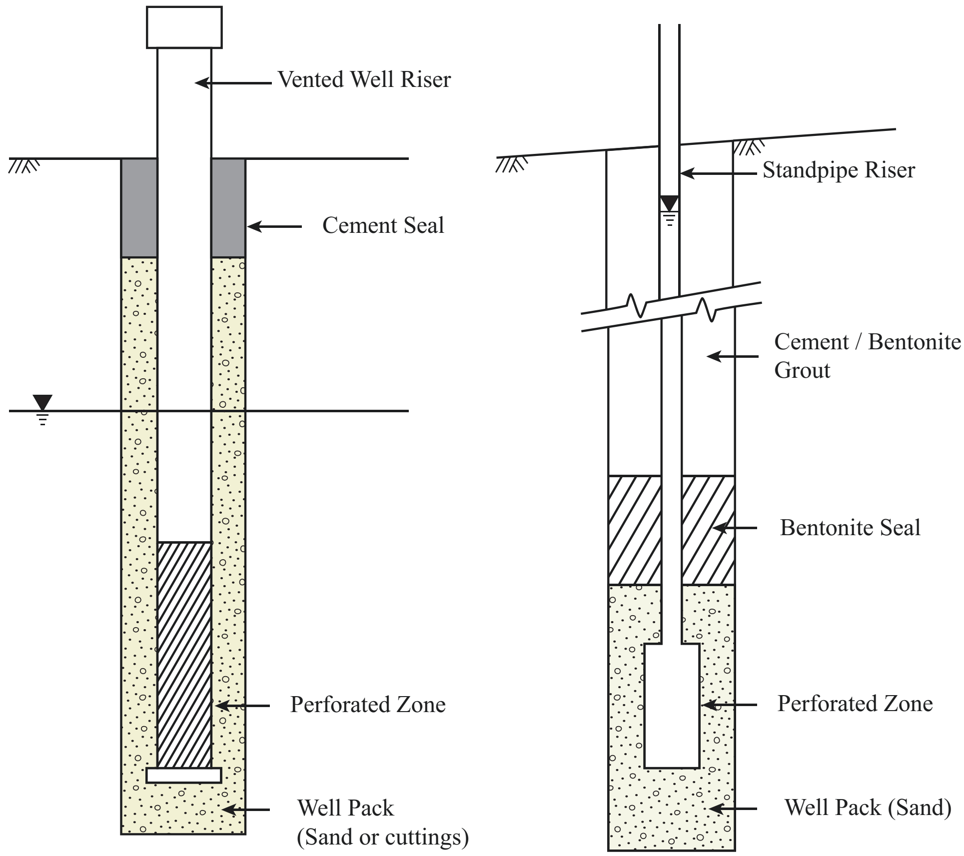
Instrumentation, Fig. 1 (**a**) An observation well and (**b**) a standpipe piezometer



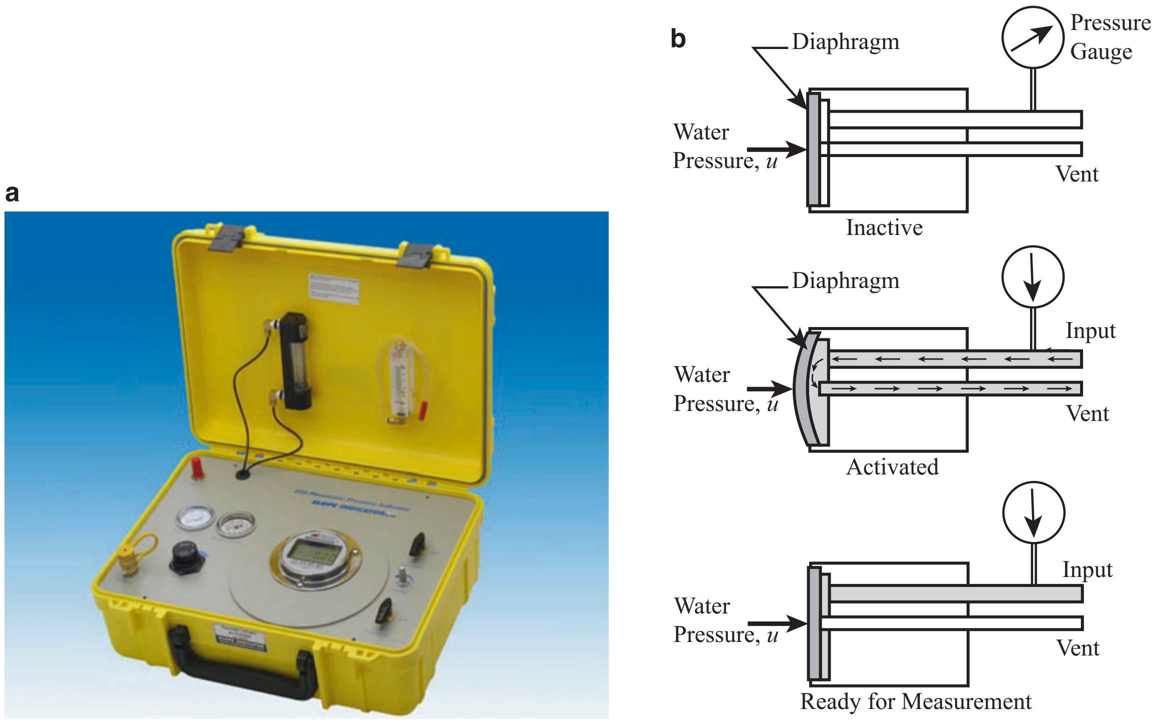
standpipes into soils that are subject to consolidation is not recommended as the riser sections may buckle or break. Furthermore, standpipes may not provide sufficient response time to loading to accurately assess the accumulation of excess pore-water pressures leading to foundation failure during embankment construction.

Pneumatic and Vibrating Wire Piezometers

When monitoring groundwater levels over longer-term applications, when the response time is critical in fine-grained soils, when multiple well points are required in the same borehole, or when consolidation settlements or landslide displacements are a concern, it is more practical to measure the pore-water pressure directly rather than the water level. This is typically achieved by installing either a pneumatic or vibrating wire piezometer (VWP) types that have a history of providing reliable measurements over a long period of time under a variety of adverse conditions. Recently, as the cost for electronics has become more affordable and field-based data logging systems more commonplace, VWPs have become the standard. Both pneumatic and vibrating wire piezometers are constructed as small well points, with porous stone at the end that is attached to a protected cable extending to the ground surface. Historically, the porous stone consisted of a high air entry value (HAEV) ceramic stone; however, due to difficulties in ensuring complete saturation of the stone, this method has more or less ceased *in lieu* of sintered steel.

Installation is similar to that of the standpipe piezometer in that the piezometer tip is installed into a borehole to the desired depth and sealed off in that formation. The relatively narrow diameter of the pneumatic tubes or signal cable (VWPs) makes it relatively easy to install a nest of several piezometers into a single borehole and still ensure that each tip is sealed into an isolated zone. Historically, pneumatic and vibrating wire piezometer tips were placed into sand packs wrapped in either burlap or a geosynthetic, lowered into the borehole, and backfilled with sand pack up to around 300 mm above the tip location. The sand pack was then sealed off with bentonite chips and the process repeated at the next monitoring interval. However, McKenna (1995) and Contreras et al. (2008) reported that the VWP tips can be fully grouted in place provided that the permeability of the cement-bentonite grout is within three orders of magnitude of the soil formation.

Pneumatic piezometers were very common in the 1970s to 1990s as they were relatively inexpensive and easy to install. The pore-water pressures measured from these piezometers were highly accurate with consistent repeatability. Pneumatics work on the premise of pressure equilibrium on a flexible diaphragm located within the piezometer tip. Pneumatic piezometers feature a piezometer tip consisting of the



Instrumentation, Fig. 2 (**a**) Pneumatic piezometer readout unit (with permission from Durham Geo Slope Indicator) and (**b**) schematic illustrating the use of a pneumatic piezometer

porous stone and the flexible diaphragm, which is connected to the lead cable composed of a pressure tube and a vent tube. A photo of a pneumatic piezometer tip and a schematic of their function are shown in Fig. 2a and b, respectively.

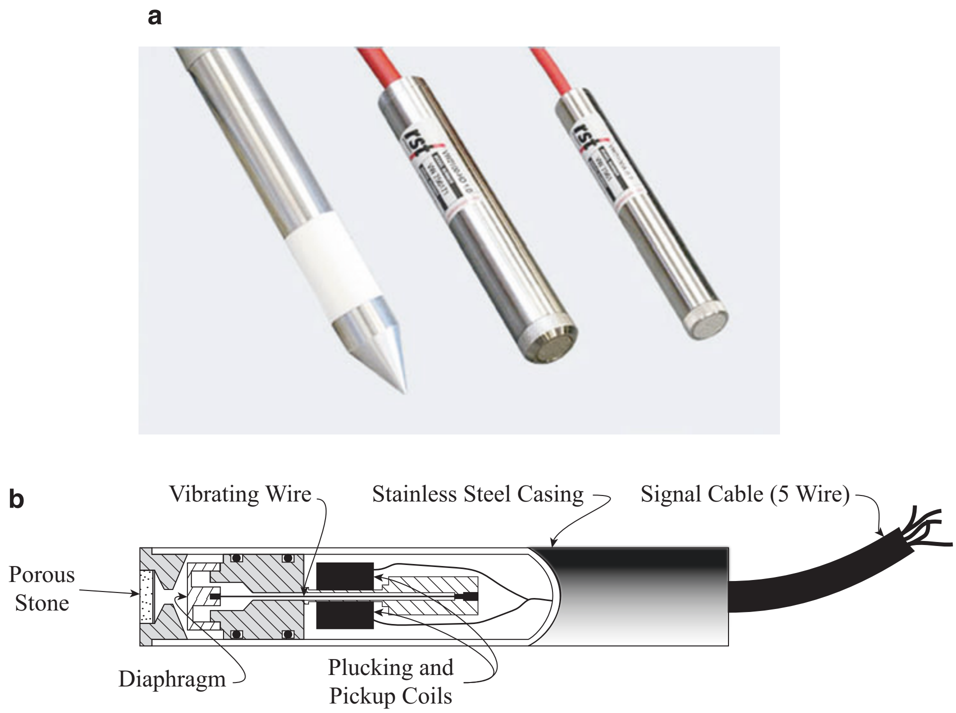
Pressurized gas (typically nitrogen) is injected into the pressure tube at the ground surface. The gas reaches the piezometer tip and forces the flexible diaphragm out. This direction is opposite that provided by the pressure of the groundwater. The movement of the diaphragm opens the flow of gas to the vent tube and permits the venting of gas at the ground surface. Once the system is venting, the flow of gas is stopped. The cessation of gas flow at the surface permits the pore-water pressure acting on the diaphragm to come to equilibrium with the pressure in the gas line, thereby forcing the diaphragm to a null position. The pressure of the gas at the ground surface at diaphragm closure is taken as being equal to the pore-water pressure acting on the porous stone.

The ease and accuracy of use of pneumatic piezometers is offset by the lack of capability for automation. Because gas must be injected into the system, an operator is required for a reading to be made. Furthermore, the equipment needed for measurement is not portable by today's standards, consisting of readout units with built-in nitrogen tanks weighing approximately 5–10 kg. Finally, the readout units require regular maintenance to ensure the gas regulators are compliant with safety standards. Moreover, the use of nitrogen requires the regular refilling of the internal tanks and therefore pneumatic piezometers have a built-in expected maintenance cost associated with their use.

Vibrating wire piezometers operate on the premise of a tensioned strand of wire installed to a flexible diaphragm open to the pore-water pressure in the ground. The measurement is taken by exciting the wire strand electrically and measuring the frequency of the wire during excitation or "plucking." As the pressure in the VWP changes, the diaphragm displaces and changes the degree of tension on the wire which, in turn, alters the frequency at which the strand vibrates when plucked. A calibration function is determined by the manufacturer by measuring the instrument frequency at a range of hydraulic pressures (typically the VWP pressure range is designed). The field pore-water pressure acting on the diaphragm of the VWP tip is then interpolated based on the calibration function. Most VWPs are equipped with onboard thermistors to allow for the measurement of temperature at the piezometer tip. The calibration and calculation of the hydraulic pressures should be corrected for changes in temperature at depth. An image of a typical VWP is shown in Fig. 3a, and a schematic of the internal structure of a VWP is shown in Fig. 3b.

Because VWPs are electronic in nature, they lend themselves well to automation and connection to dedicated data acquisition (DA) systems. This means that a single DA system can read and record pore-water pressure measurements of many instruments. Coupled with multiplexers, most DA

Instrumentation, Fig. 3 (**a**) Typical vibrating Wire Piezometer (with permission from RST Instruments) and (**b**) schematic showing the internal structure of a VWP (Adapted from USBR 6515-09)



systems are capable of monitoring up to 128 VWPs. One drawback associated with DA systems is the potential for damage by lightning strike to some or all of the attached instruments. Therefore, a DA system if used must be equipped with an onboard mercury gas tube designed to rupture when an electrical surge is encountered; the breaking of the mercury gas tube disconnects the multiplexers and ultimately all of the VWPs. In addition, the construction of a lightning rod and a direct ground at the monitoring station is highly recommended to minimize the risk of lightning strike altogether, as a single lightning strike can destroy hundreds of thousands of dollars of instrumentation in a split second.

If lightning is a concern or if the monitoring of pore-water pressures at all times is critical to the performance of the structure (i.e., an earth dam or tailings dike), then fiber optic piezometers can also be used. Fiber optic piezometers are not as common as VWPs due to the need for a readout unit or DA system that is not common to most engineering firms. Furthermore, the cost for both the instruments and DA systems has not yet reached a point that would warrant usage in day-to-day monitoring projects. Currently, most fiber optic piezometers are used in high budget, data-sensitive projects. In addition, the long-term functionality and stability of fiber optic piezometers are not well documented, though their construction materials and recent developments in fiber optic technology are quite promising and suggest they will be more cost efficient in the near future.

Fiber optic piezometers operate on the premise of Fabry-Perot interferometry. Similar to pneumatic and vibrating wire piezometers, fiber optic piezometers feature a stainless steel diaphragm that is open to the formation being measured. Pressure changes on the diaphragm vary the length of the Fabry-Perot cavity and thereby alter the light wavelength within the piezometer. Because the piezometer cables are constructed of fiber optic cable, they are immune to lightning strikes and electrical surges from a lightning strike on the DA system. Therefore, a direct hit by lightning to the DA system in the field will destroy the DA system but leave all of the instruments intact and operational. This essentially saves the high cost of installation and instrument replacement.

Negative Pore-Water Pressure/Water Content Instruments

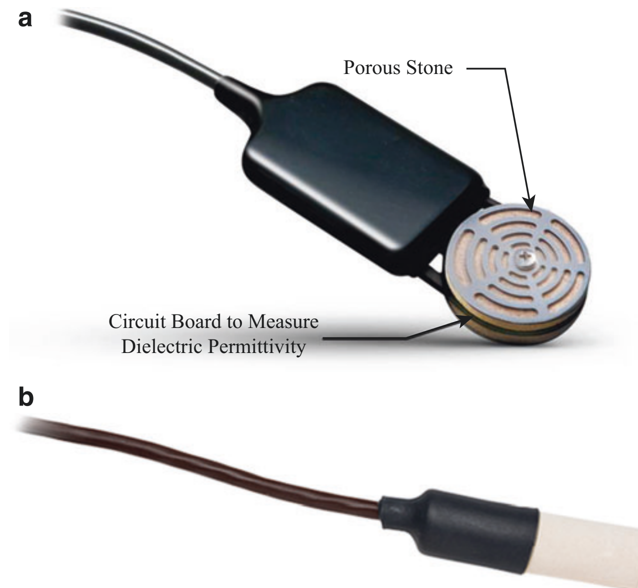
For many years, attempts have been made to develop reliable methods for monitoring negative pore-water pressures in soils. Most developments have come from soil scientists (agriculture) as opposed to geotechnical engineers. Most engineering methods focus on measuring the matric suction of a soil instead of the total or osmotic suction. This is because matric suction is the primary driving factor contributing to an increase in effective stress. Furthermore, loss of matric suction over time can lead to progressive softening (McLernon, 2014; Carse, 2014) and failure of slopes and cuts. The measurement of matric suction, however, is hampered by the inability to directly measure suction for any long period of time to pressures less than atmospheric. The formation of air bubbles (cavitation nuclei) affect the vacuum and therefore measurement of the water potential ceases. For this reason, a

Instrumentation, Fig. 4 (**a**) Standard tensiometer with a vacuum gauge and refill reservoir (with permission from Soil Moisture Inc.) and (**b**) rapid response mini tensiometer (With permission from Meter Instruments)

number of indirect methods have been developed. In this section, direct measurement methods of matric suction will be discussed first, followed by indirect methods.

The most common direct method for measuring the matric suction in a soil is to use a tensiometer. Tensiometers were initially developed to help farmers understand when they needed to water their fields by allowing them to accurately monitor the wilting point of their crops. With Bishop's (1959) use of matric suction in the effective stress equation, this value has become important for soil engineers. A tensiometer is a simple instrument consisting of a long tube (typically clear acrylic) with a HAEV porous stone on one end and a vacuum gauge or transducer on the other. The tube is filled with water and the porous stone placed into the soil. The porous stone must be in intimate contact with the soil for accurate measurements to be taken. This suggests that either a purpose-drilled hole be constructed or the instrument be embedded in the ground. In most cases, a water reservoir at the top of the tube permits the flushing of the porous stone when the pressure in the tensiometer is relieved. The vacuum gauge can be replaced with a vacuum transducer and connected to a dedicated DA system for data logging over time. Figure 4a and b are photos of a standard tensiometer and a smaller diameter, quick response tensiometer, respectively.

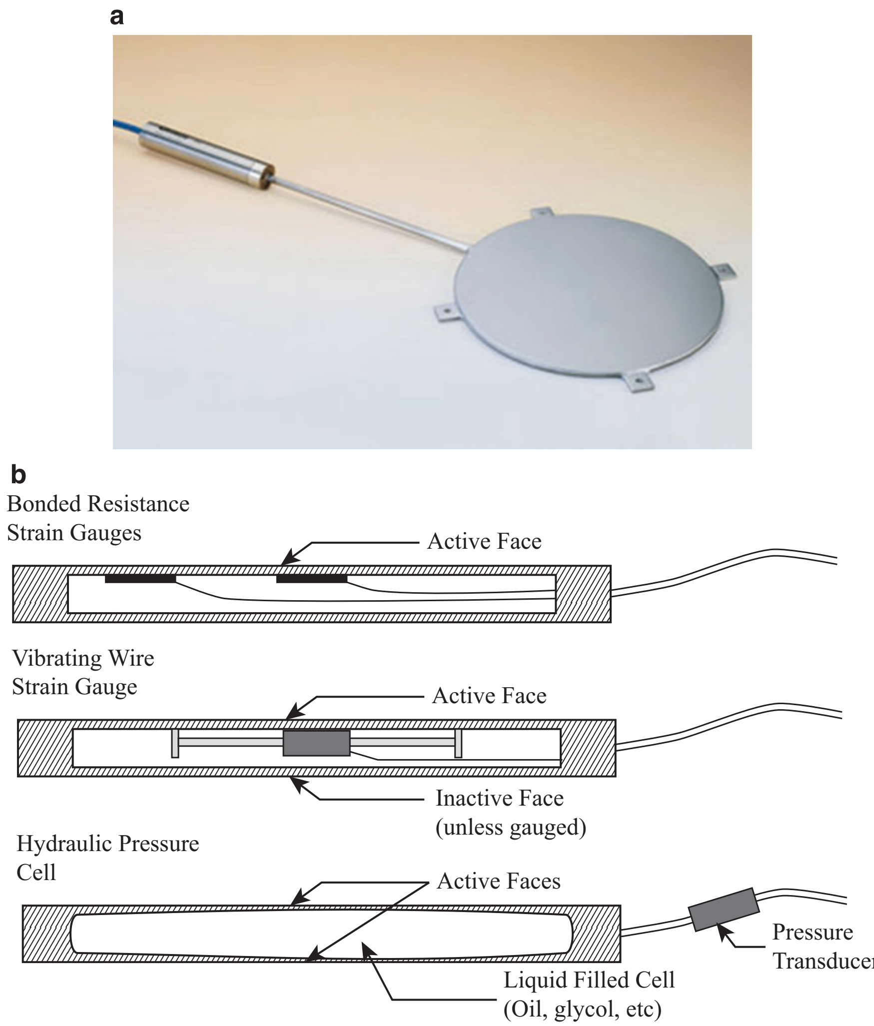
As stated elsewhere, one of the main drawbacks of tensiometers is that they are ineffective at pressures lower than 1 atm. In reality, this minimum pressure is dictated by the elevation and temperature at which the test is being



Instrumentation, Fig. 5 (**a**) High tension matric suction transducer using dielectric permittivity (with permission from Meter Instruments) and (**b**) high tension matric suction transducer using thermal conductivity (With permission from Campbell Scientific)

conducted. In most cases, the minimum pressure does not exceed −80 kPa. Another drawback of tensiometers is that the porous stone must be completely saturated for the method to be effective. The most effective way of saturating the stone is not to place it in water overnight but rather to cycle the stone through wetting and drying cycles. This process can be sped up using a hair drier to dry the stone and then flushing the stone by releasing the internal pressure. In addition, a couple of drops of detergent can be added to the flush reservoir, as this dispersant reduces the surface tension both in the stone and along the side walls of the tensiometer itself.

Attempts have been made to produce a high pressure tensiometer to measure matric suctions less than 1 atm (Ridley and Burland, 1993; Guan and Fredlund, 1997) with little success. To overcome this, recent developments have stopped attempting to measure the tension directly in the soil mass and instead determine the water content of a porous stone by measuring the dielectric permittivity. Once the water content of the porous stone is determined, if the soil water characteristic curve (SWCC) or soil water retention curve (SWRC) is well understood, then matric suction of the porous stone can be measured. Assuming that the matric suction of the soil is in equilibrium with the porous stone with which it is in contact, then matric suctions far less than 1 atm can be reliably measured. The range of the instruments are limited by the SWCC of the porous stone in that the minimum negative pore-water pressures that can be measured coincide with the air entry value (AEV) of the porous stone. Figure 5a is an image of a typical high tension matric suction instrument.



Instrumentation, Fig. 6 (**a**) Typical flat plate earth pressure cell (With permission from GeoKon Instruments) and (**b**) schematic illustrating the operating principle of an earth pressure cell

Another method for indirectly measuring the matric suction uses a thermal conductivity sensor. The sensor is described by Sattler and Fredlund (1989) and works on the principal of heat dissipation. The sensor consists of a thermal element surrounded by a ceramic porous stone. The thermal element is capable of heating the core of the sensor for set periods of time while measuring the rate of heat dissipation following cessation of heating with time. Once the sensor is installed into the ground, the water content of the soil will come to equilibrium with the porous stone of the sensor. The rate of heat dissipation is directly related to the water content of the porous stone. Increased rates of dissipation are related to increased water content within the stone. Because the sensor essentially measures the *in situ* water content, the SWCC of the porous stone must be determined, as was the case for the dielectric permittivity sensor. Once the water content and SWCC of the porous stone are known, then the matric suction of the soil can be obtained.

Total Stress in Soil

Accurately and consistently measuring total stress in soils is typically very difficult. The methods currently available are well established but prone to changing boundary conditions depending on their use and application. Most Earth pressure cells use sealed oil reservoirs and VWPs to measure stresses on a plate as they accumulate from external loads. The vibrating wire transducers are typically installed on arms that extend away from the plate to minimize the influence of the transducer on differential compaction around the cell. Other Earth pressure cells can measure stress by determining the strain on the surface of the cell plate using high precision bonded strain gauges. The use of strain gauges permits the measurement of displacement (and subsequently stress) on either one or both faces of the cell, whereas a vibrating wire pressure cell measures the change in pressure within the cell cavity alone. Figure 6a shows a typical Earth pressure cell, whereas Fig. 6b provides a schematic of a typical cell.

All pressure cells, whether embedded or in contact with a structure, must have an adequate sensing area, with smaller cells making it more difficult to achieve reliable values. The cells also should have minimal sensitivity to nonuniform soils and a method of installation that does not dramatically impact the *in situ* stresses of the soil. Typically, cells are placed into hand dug trenches and backfilled using hand tamping, or cast into concrete facing where possible, or affixed to a structural face where casting in place is not an option.

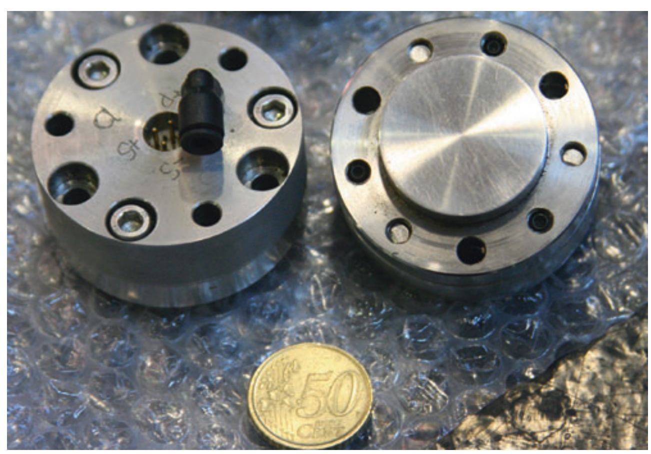
Pressure cells are installed either within fill materials such as during embankment construction to measure the applied loads leading to consolidation, or in contact with structures such as retaining walls, or on the outside of sprayed concrete liners for tunnels. The primary reason for installation of Earth pressure cells is to confirm loading assumptions made during the design stage. Finally, most commercially available Earth pressure cells are designed to measure static loading alone and are not capable of measuring seismic or dynamic loading. If measurement of dynamic load is required, then the cells must be designed to respond quickly and the plucking frequency of the VWPs set accordingly.

As stated previously, Earth pressure cells are wrought with issues associated with their boundary conditions. Weiler and Kulhawy (1982) as well as Dunnicliff (1993) summarize the documented errors associated with the use of Earth pressure cells within embankments and fills. The major issue associated with Earth pressure cells is related to the stiffness differential between the cell and the surrounding soil. If the total stress cell is stiffer than the soil, it will effectively attract stress and over-register the measured value. Moreover, the compaction and soil stiffness directly surrounding the cell can be an issue as the presence of the cell requires different compactive effort relative to the surrounding soils. This free-field stress differential is minimized by minimizing the thickness of the cell itself; however, reducing the thickness of the cell does not mitigate the problem as a thinner cell tends to become flexible and runs the risk of deforming such that the soil over the cell can lose contact due to arching, resulting in under-registration of the Earth pressure. Readers are referred to Selig (1964) for more details. Calibration of the pressure cells for real world compaction states is also extremely difficult (Bozozuk, 1970). Furthermore, calibration in the field can also be extremely expensive and difficult to effectively perform. For this reason, most practicing engineers rarely use stress cells as a means to calculate stress within or below an embankment; rather, this is usually done via back calculation using numerical modelling fitted to field displacement monitoring and measured constitutive models.

Earth pressure cells placed in contact with structures are not as wrought with problems as those embedded in embankments, but they are also not entirely without problems. With a contact cell, the stiffness of the cell as well as temperature fluctuations can influence the results. Thick walled hydraulic cells are not typically recommended as they cannot be effectively embedded within the structure and therefore can protrude from the face. In recent years, cell thicknesses have been dramatically reduced to minimize this issue. With this in mind, the stiffness of the cell must be considered; when it is affixed to a structural element (concrete wall) or tunnel liner, the cell must have stiffness less than that of the structural element. In addition, the surface on which the cell is installed must be flat and planar. Irregular surfaces can result in significant error. Finally, temperature should be measured at the time of the pressure reading. All vibrating wire pressure results should be corrected for temperature, which has a strong effect on hydraulic cells (by influencing the volume and therefore the pressure of the fluid) as well as on the plate itself (mainly for strain gauge cells where thermal contraction / expansion can result in misleading measurements).

Recently, Talesnick (2005) and Talesnick et al. (2011) developed a method for measuring Earth pressures in contact with structures that minimizes the effects noted above. By developing small sensors that are pressure regulated by a computer terminal, issues associated with stiffness and temperature are minimized. Talesnick (2005) demonstrates that the pressure plate is maintained in a null position at all times by measuring the strain on the surface of the plate at 1 second intervals and adjusting the pressure automatically using pressurized gas on the opposite face. Though still in its early stages, feedback on this Earth pressure system appears positive; however, the cost of the monitoring system is currently quite high. A photo of two (functional) miniaturized Talesnick cells is shown in Fig. 7.

Because all of the methods for measuring the change in pressure are electronic in nature, all of the pressure cell instruments are well suited for continuous data logging using dedicated DA systems. Importantly, the use of strain gauges to measure the strain of the load surface may require a



Instrumentation, Fig. 7 Compensation type contact earth pressure cell structural DA system with wiring and logging that may be considerably different than those sold by most geotechnical instrumentation suppliers.

Measurement of Deformation

Perhaps the broadest groups of instrument diversity are those related to deformation monitoring. Engineers measure deformations very well as it is an obvious outcome of construction and natural processes. Therefore, many methods have been developed to help understand what is taking place when changes to the effective stress occur in the ground. Most of the methods used today are extremely inexpensive and easy to employ with a high degree of reliability. Dunnicliff (1993) provides a complete outline of the various deformation instruments and their applicability.

Instrumentation of deformations spans applications from simple survey methods using a level and rod to lateral deformations of excavations and slopes, to convergence of tunnels or rotation of buildings. Recent developments have also started implementing remote sensing techniques as opposed to discrete point measurements to help quantify displacements over large regions. In addition, many methods have been advanced in recent years to utilize automation to provide ongoing, low-cost monitoring systems.

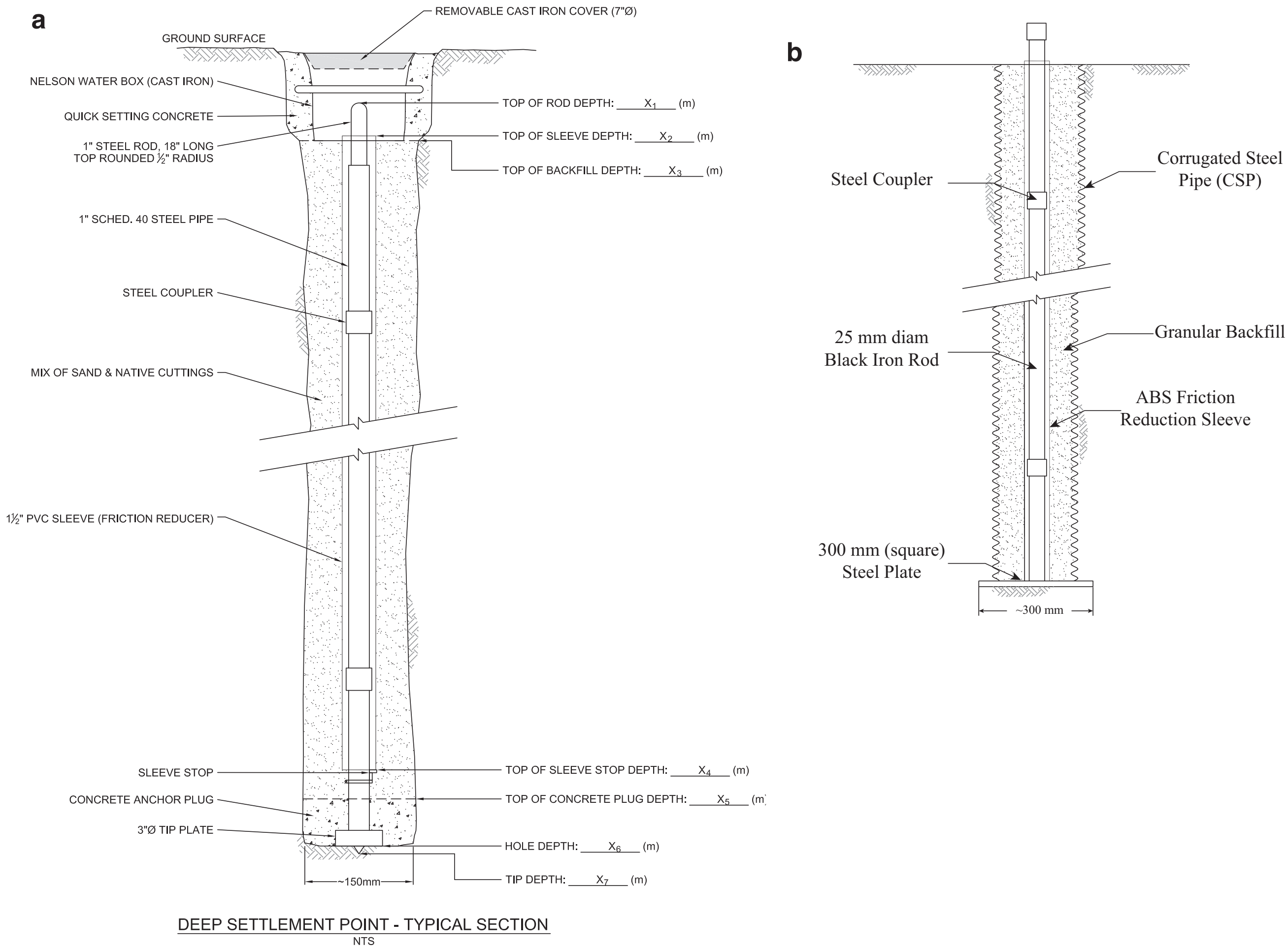
Survey Instrumentation

One of the most commonly used methods for vertical and horizontal deformation monitoring is survey methods. Survey methods include measurement of vertical displacements using a survey level and rod through to total deformations using a total station and prism. Each will be discussed separately. Please note that discussion of remote sensing methods such as light detection and ranging (LiDAR) and aerial photogrammetry will be left for the remote sensing section despite these technically being survey methods.

Survey methods have been available since the start of engineering and require a trained land survey crew in addition to well positioned, fixed benchmarks. Most construction surveyors are not necessarily familiar with the precision required for informed monitoring programs, nor are they necessarily fully aware of the limitations of their equipment or understand the importance of highly accurate, repeatable measurements. The error associated with the use of total stations can be on the order of +/− 2 mm. In most circumstances for tunnel construction, this error can constitute over 10% of the total displacement. This may be sufficient in some cases where the risk to surface damage is low, but different methods must be used if precise data are required to make informed decisions with respect to damage potential. Most precise levels can record measurements on the order of +/− 0.25 mm and, in the correct conditions, this error can be reduced further. In cases where precise measurements are required but the deformations are not solely vertical, total stations can be used as long as they are supplemented and calibrated using more precise methods at discrete locations.

Settlement monitoring is generally carried out for embankment construction on soft, compressible soils; for tunneling projects completed in urban environments; and within the zone of influence of deep excavations. The instrumentation generally consists of black iron or steel rods installed into boreholes that extend below the frost line. In regions where there is little to no frost penetration, the use of concrete nails or other surface nails may be used to establish a survey point. The cap of the survey rod should be rounded to permit repeatable measurements. The base of the rod should be encased in concrete to hold the rod in place or be fitted with a borehole-sized plate. Also common is use of the drill rig to push the rod slightly into the native soil prior to placing concrete. A friction-reducing sleeve consisting of either polyvinyl chloride (PVC) or acrylonitrile butadiene styrene (ABS) pipe is slid over the survey rod, with the upper 100 to 150 mm of the rod protruding through the collar of the friction-reducing sleeve. The diameter of the friction-reducing sleeve should be large enough so that the survey rod and any couplers can slide freely within the sleeve at all times. Care should be taken so as to keep the friction-reducing sleeve on top of the concrete plug (if used). This ensures that the settlement rod displaces from the ground below it and remains frictionless. Placing a thin layer of well sand on top of the concrete is sometimes beneficial to maintain separation. At this time, the borehole may be backfilled with either sand or a cement-bentonite grout. The survey rod should be maintained some distance below the ground surface and protected from vehicular and pedestrian traffic with an appropriate cover. Figure 8a is a schematic of a typical borehole type settlement rod. The presence of frost must be considered when installing these instruments, and the idea that one can reasonably predict the associated frost heave during winter months is naïve and potentially dangerous. Heave not related to construction activities has the effect of masking deformations that will be manifested upon spring thaw and, by that time, there is no ability to adjust construction methods to control settlements.

When the rod is to be placed within a fill or embankment, a base plate that is sufficient in size should hold the rod vertically in place until backfilling starts. The top of the rod typically consists of couplers to permit the addition of more rods as the height of the embankment increases. Any couplers used should be tightened with pipe wrenches and thread lock to minimize the risk of coupler movement. Hand digging a small trench typically occurs prior to placement of the settlement plate to ensure relatively undisturbed foundation conditions. As with the borehole rod, a friction-reducing sleeve is placed over the rod. Due to the significant truck traffic

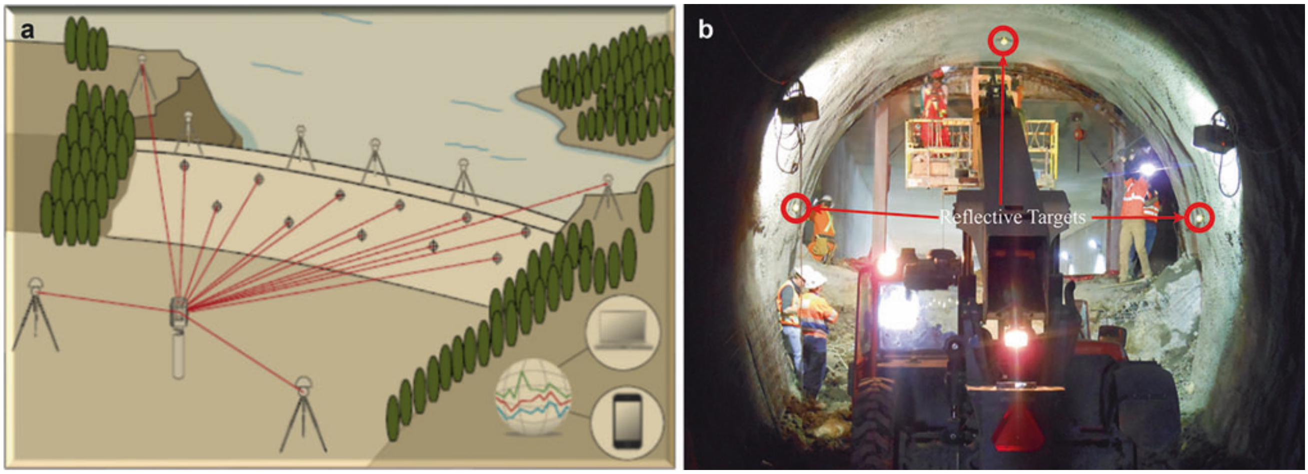


Instrumentation, Fig. 8 (**a**) Typical borehole settlement rod and backfill detail and (**b**) typical settlement rod placed within an embankment fill

typically occurring during the placement of embankment fills, the settlement rods should be protected as best as possible. Use of corrugated steel pipe (CSP) culvert sections is common. The CSP is slid over the friction-reducing sleeve, and then the annular space between the friction sleeve and the CSP backfilled with granular material. Care should be taken during backfilling of the CSP to ensure that no granular material enters the friction sleeve. As the fill increases around the settlement rod, additional rods are threaded into the couplers and additional friction sleeves and CSPs are added as needed. Figure 8b is a schematic of a typical embedded type settlement rod.

Monitoring is manual and requires the use of a precise level and survey rod. In cases of settlement of an embankment where the risk of damage to nearby structures is low, then the points may be surveyed using a total station and reflective prism. In cases where there is little infrastructure around and little concern with pedestrian traffic or vandalism, each of the settlement rods can be fitted with a reflective prism and the points surveyed continuously using an automated total station. Using proper reflective prisms where settlements are critical is important because the accuracy of a reflective prism relative to a reflective (sticker) target is substantial.

A total station must be employed when surveying is used to monitor the ground deformations of temporary support, such as a soldier pile wall or a sprayed concrete liner in a tunnel. In this case, reflective targets are placed either within or onto the surface of the structure to be monitored and the points surveyed on a regular basis. The precision of the surveys will not be high but is typically suitable for most applications. Confirming the position of the setup point on a regular basis is important, particularly in tunnel survey monitoring. Tunnels rarely ever have “fixed” points that can be used as a reference and, therefore, regular loops to reposition back sights are essential for accurate monitoring. Because the points are positioned on the structure, monitoring generally does not obstruct most construction activities and so is quite attractive to most contractors. However, the accuracy is limited and therefore supplementing the measurements with a more precise monitoring method is beneficial so that all



Instrumentation, Fig. 9 (**a**) Deformation monitoring of structures using a total station and (**b**) convergence monitoring points in a tunnel

measurements can be calibrated on a regular basis. Figure 9 illustrates the use of survey methods in (a) an excavation or dam and (b) within a tunnel for convergence of the temporary sprayed concrete liner.

Extensometers

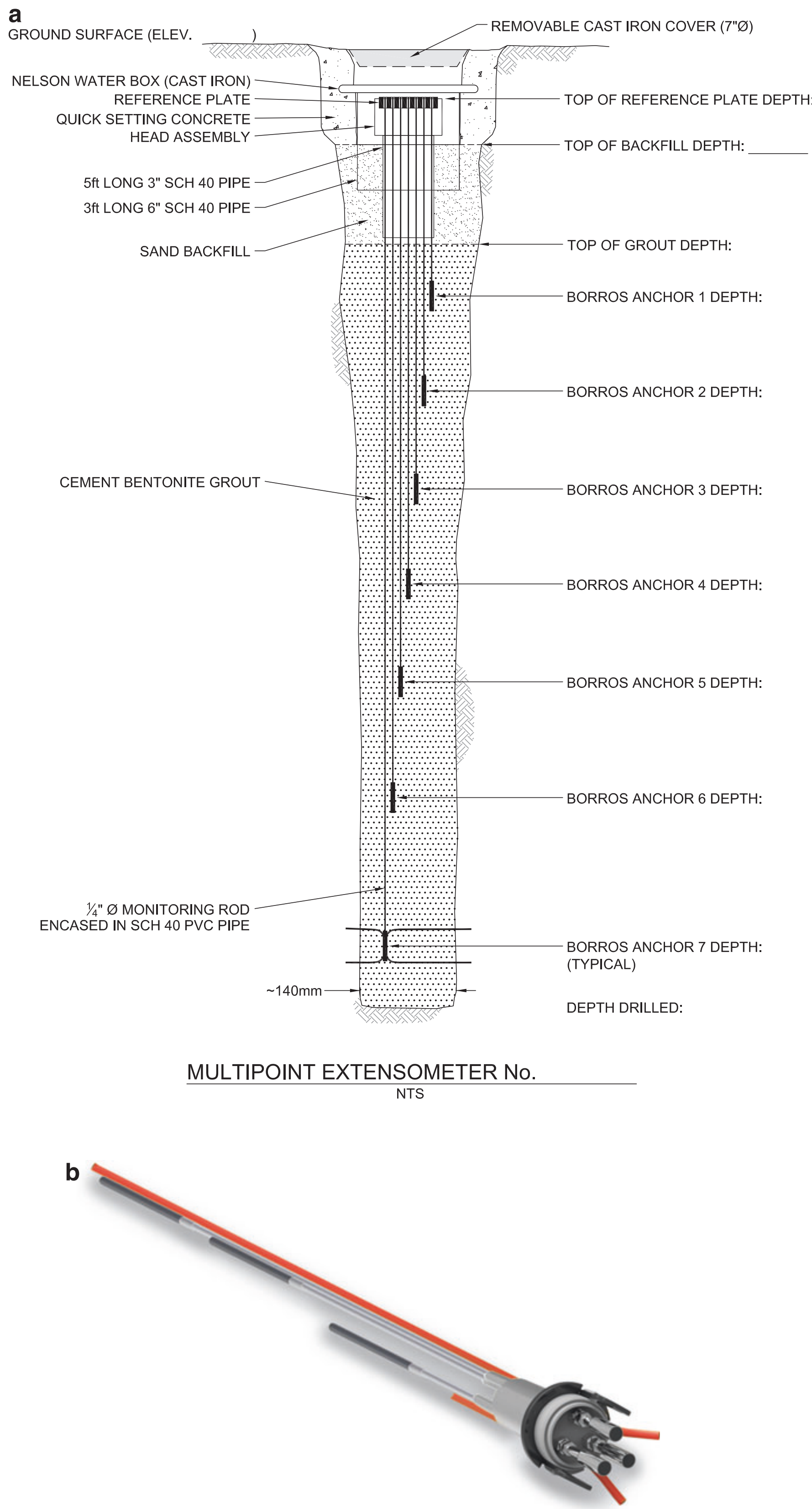
Extensometers measure displacement relative to a set datum or length (strain). They are commonly used for settlement monitoring as well as convergence in tunnels, although they can also be used for monitoring crack width. Extensometers are usually quite accurate with a high degree of precision. They have very few installation issues, provided an experienced technician or engineer does the installation. Understanding the soil or rock into which they are to be installed is very important due to implications with respect to the method of fixation to the surrounding soil.

Borehole extensometers are the most commonly used extensometers because they can accommodate multiple monitoring depths in one borehole. The diameter of the borehole is dependent on the number of monitoring points required. The more points to be installed into a single hole, the larger the diameter. The instruments are constructed as a series of small diameter stainless steel rods protruding from a head located at the collar of the borehole. Each rod is cut to a different length and the end installed into the borehole is attached to an anchor to affix the rod to the soil at a discrete location. Most borehole extensometers use burros-type spider anchors. In bedrock, the anchors are often fully grouted in place and the anchors omitted. The displacements are measured by using either a vibrating wire strain gauge at the head or a caliper. Because vibrating wire technology can be used, borehole extensometers are readily automated. All displacements are calculated as the difference between the initial reading and subsequent readings. Figure 10a is a schematic of a typical borehole extensometer, and Fig. 10b is a photo of a rock borehole extensometer.

Because the rod lengths are manufactured, considerable thought must be given to the depths of interest during the design phase. Furthermore, it is integral to understand the ground displacement fields into which the extensometer will be installed. For instance, a vertical, multipoint extensometer installed into a borehole to monitor vertical settlements associated with tunneling will not be accurate if the head (surface point) is not surveyed on a regular basis. This is because the head will also settle with the anchors and the settlement of the head will effectively hide the settlements of the lower points. Surveying is not required if the base anchor is installed into a competent bedrock and is below the zone of influence.

Convergence monitoring is carried out using a tape extensometer (TE). The use of tape extensometers is becoming increasingly rare as survey methods become more reliable. Tape extensometers are relatively labor intensive, do not lend themselves to automation, and have the potential to obstruct construction activities. However, the accuracy and resolution of a TE measurement far exceeds that achievable by any other method. Typical error associated with TE readings is on the order of +/− 0.1 mm but is dependent on the operator.

A TE works on the premise of measuring the change in chord length across a given opening (usually a tunnel diameter). Expansion nuts are installed into the liner and eye bolts are threaded into the expansion nut using thread lock. It is important that the eye bolts not rotate once the initial measurement has been recorded as this will affect all subsequent readings. A tape is run from one bolt to another, across the opening, and tensioned using a spring-loaded mechanism. Once the tension is set, the measurement is taken as the combination of the tape distance plus a fine measurement



Instrumentation, Fig. 10 (**a**) Typical multipoint borehole extensometer installation detail and (**b**) a borehole extensometer (With permission from SISGEO)

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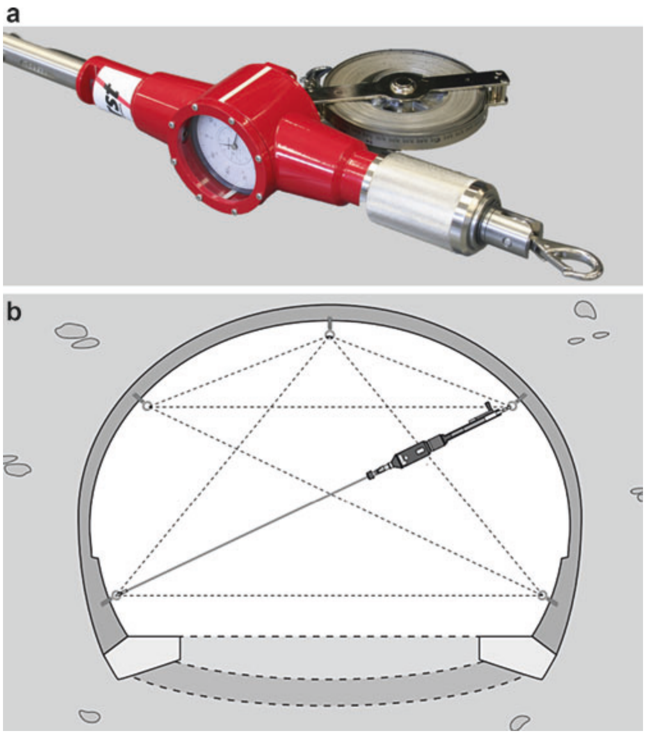


Instrumentation, Fig. 11 (**a**) Typical tape extensometer (image courtesy of RST Instruments) and (**b**) in-tunnel measurement of displacement (Image courtesy of SISGEO)

using a built-in caliper. Another benefit to using a TE is that any error is readily observed by simply checking a chord length from the other direction and noting the difference between two readings.

Figure 11a is a photo of a typical tape extensometer and Fig. 11b shows its intended use.

Tape extensometers can also be used to monitor the growth of cracks in rock excavations, though their accuracy is somewhat diminished by the factors associated with wall movement. To monitor the growth of cracks, an anchor is grouted or epoxied into the rock face on either side of the crack. The TE is then installed between the two anchors and the initial length recorded. Subsequent readings show the growth or closure as relative displacements. Use of an instrumented tensioned wire or rod as opposed to a manual TE is becoming increasingly more common. By affixing strain gauges to the wire or rod, the growth of the crack can be measured to an extremely precise degree.

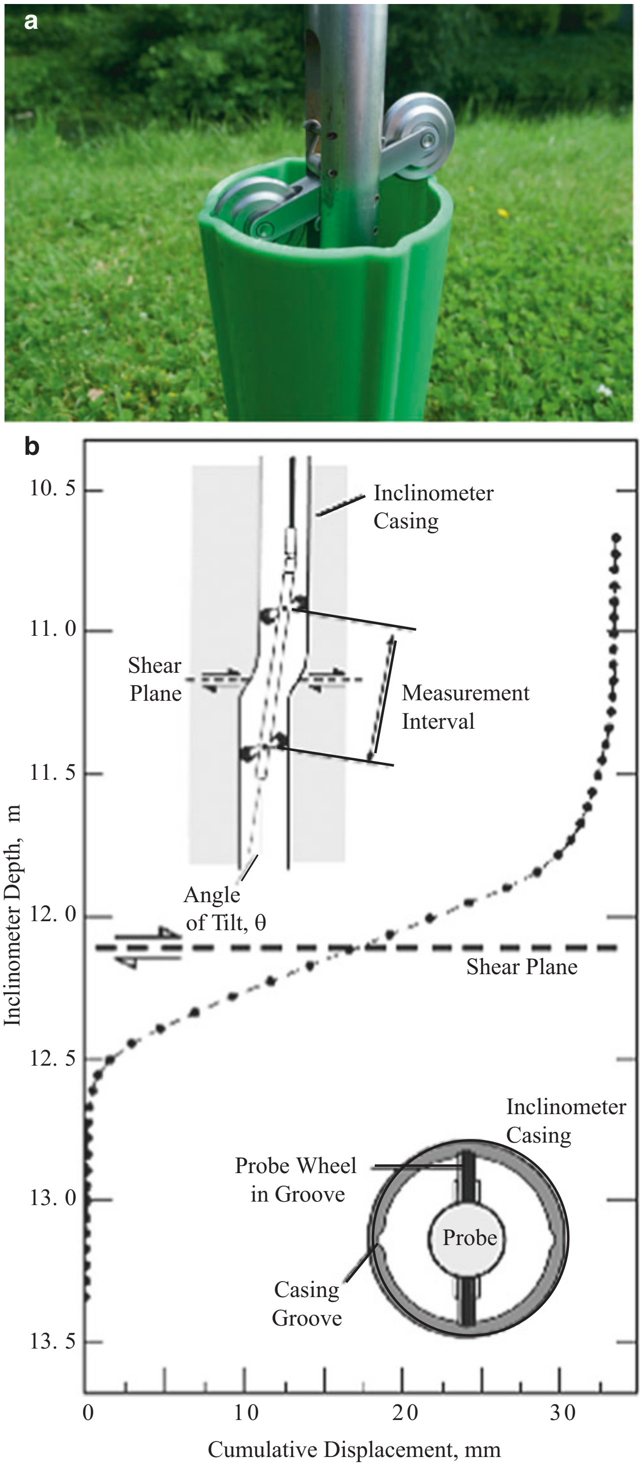
Tiltmeters

Tiltmeters are typically used to monitor rotations of structures as a result of nearby construction activities or from lateral loading such as walls and dams. They are extremely accurate in that they tend to use accelerometers or vibrating wire transducers. Manual tiltmeters exist but are becoming less common as they cannot be automated. Measurements are taken by placing the tiltmeter in the exact location and position it was initially. This is done using reference plates that are bolted or epoxied to the structure being monitored. When measured using readout units, it is important to take a reading and then rotate the unit 180° and take another reading to check the error (*check-sum*). The tiltmeter must be installed onto a structural element such as a foundation wall or column but not to a façade that is prone to movement relative to the structure. Tiltmeters are also highly susceptible to temperature fluctuations, which not only affect the sensor but also the structure to which it is attached. Installing a tiltmeter onto the foundation of a building that is exposed to direct sunlight is not recommended. Thermal expansion of the structure will result in false displacements as indicated by cyclic fluctuations associated with the heat of the day and cooling at night. Calibration of this type of displacement is also very difficult as it requires separation of the thermal properties of the instrument and the structure. For this reason, tiltmeters should be installed into the basement of structures or in shaded areas where possible.

Initially, tilt meters were quite large and cumbersome to use; however, the units have become relatively small and unobtrusive with the recent development of micro-electro-mechanical systems (MEMS) technology. MEMS technology still uses accelerometers and simply measures the change in location of one sensor to another (fixed) sensor with rotation.

Inclinometers

Inclinometers are used by just about every geotechnical and geological engineering company throughout the world. They are typically used to monitor the movement of slopes or excavations. Measurements are taken normal to the axis of a pipe installed into the ground by inserting a probe to fixed depths. Similar to an extensometer, the readings are compared to a datum usually set as the first sounding. Inclinometers are extremely accurate and precise provided that care is taken during installation of the casing and with the instrument itself. They are effective at identifying the location of slide planes as well as indicating the rate of movement when displacements measured over a period of time are compared. Measurements are reported as cumulative and incremental displacements in both the major (A) and minor (B) directions. Cumulative displacements represent the increase in movement for each monitoring point relative to the fixed base, whereas incremental displacements indicate the displacements relative to one point from the next. Most engineers commonly use the cumulative displacement, as total displacement is usually specified in most construction applications. However, one can argue that the incremental displacement is more critical to report as it indicates the location of the displacement and therefore illustrates where remedial action is required. A photo of a typical inclinometer casing and probe is shown in Fig. 12a,



Instrumentation, Fig. 12 (**a**) Inclinometer casing with probe partially in (with permission from Gouda GeoEquipment BV); and (**b**) typical cumulative data (After Eberhardt and Stead 2011)

and a plot of cumulative data versus depth is shown in Fig. 12b.

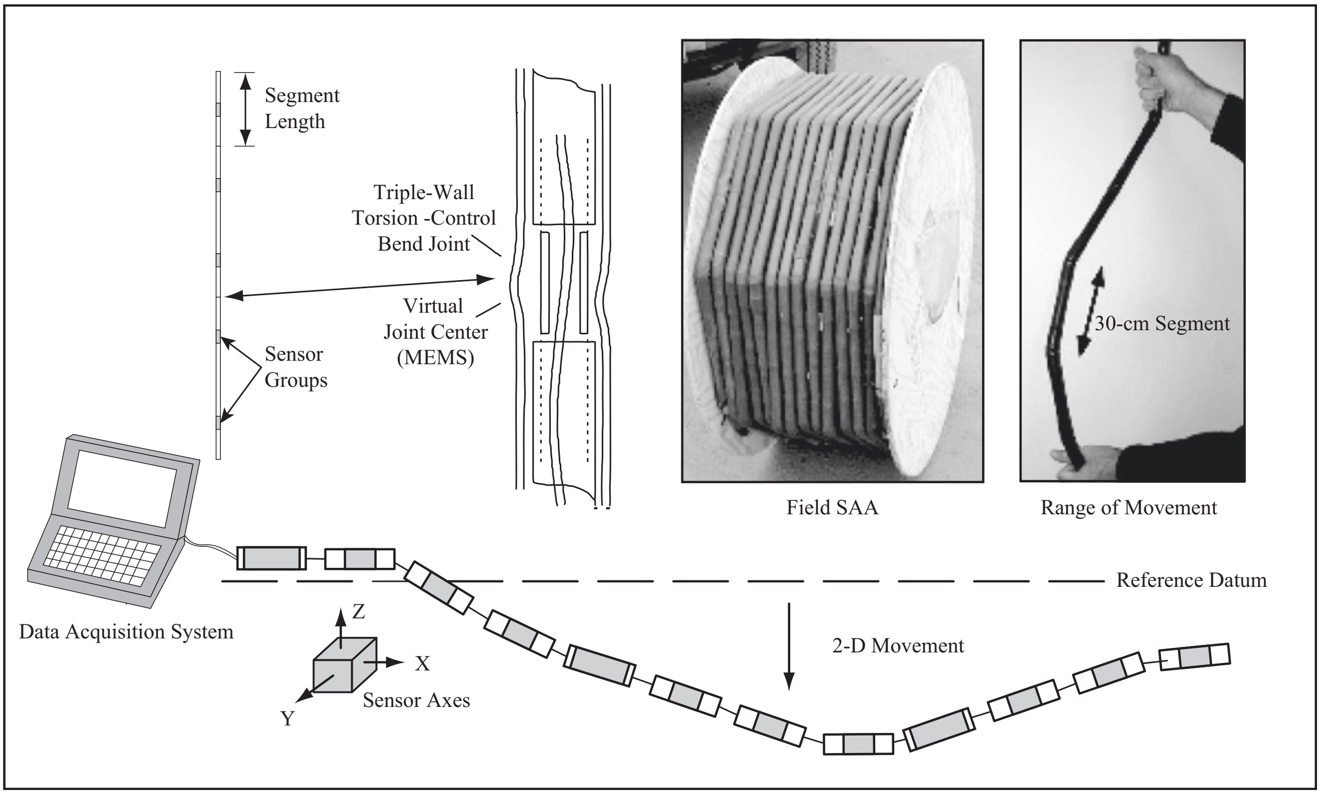
Because inclinometers are typically used in slopes that are moving, they run the risk of shearing and long-term damage. Minor shear displacements can result in sufficient angular distortion of the casing such that the probe can no longer traverse to the base rendering the instrument destroyed. Recent developments using the MEMS technology described above have permitted the construction of continuous chains of MEMS accelerometers, marketed as shape accel arrays (SAAs), resulting in an in-place inclinometer that is readily automated using a DA system. These SAAs reduce the error associated with probe functionality and can also sustain considerably more shear displacement than a conventional inclinometer casing. Furthermore, SAAs are fully recoverable and can be used on subsequent projects as needed. The convenience of SAAs is, however, offset by their relative cost. Most inclinometer casings are installed to depths greater than 20 m below the surface. An SAA to 30 m will have considerably higher up-front costs associated with purchase and installation than a conventional inclinometer, but the up-front costs may be offset by automation versus manual readings. An image and schematic of the use of an SAA is shown in Fig. 13.

Because SAAs have the ability to flex at the sensor intervals, they are not solely confined to vertical or horizontal arrangements. They can be also used around shapes such as tunnel openings to monitor the convergence of the tunnel with time. Since all points are moving into the tunnel opening, there is no fixed point to reference the change in shape. Because of this, it is important that at least one point is surveyed on a regular basis in order to provide a “fixed” point and a reference for the total displacement of the SAA. This can be achieved by simply installing a reflective prism and surveying with a total station on a regular basis. A photo of this arrangement is shown in Fig. 14.

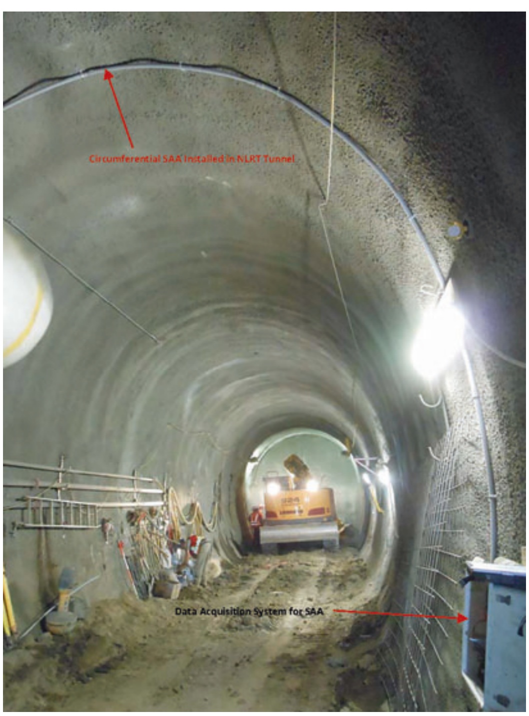
Other in-place, inclinometers (IPIs) consist of standard probes manufactured to desired lengths that are connected together in a chain. This chain is inserted into a standard inclinometer casing and connected to a DA system. IPIs are very effective and slightly cheaper than SAAs. They are also recoverable provided that the displacements of the inclinometer casing were not sufficient to “wedge” a given segment into the casing.

Inclinometers (standard, SAAs, and in-place) are prone to error associated with installation and operator error. Most problems occur during installation and are related to poor casing alignment, casing diameter, backfilling procedures, and casing spiral. Inclinometers installed through embankments prone to consolidation are also susceptible to buckling as the casing is dragged down with the embankment.

Casing alignment errors occur when the person installing the casing is “eyeing” the principal direction of slope or wall movement. This can result in error in calculation of the actual cumulative displacement and rate of displacement. More common is to align the principal A direction to north using



Instrumentation, Fig. 13 Use of SAA (Adapted from Eberhardt and Stead 2011)



Instrumentation, Fig. 14 Use of an SAA to monitor tunnel convergence

a geologic compass and resolve the vectors between the A and B directions.

Casing diameter plays a role in the effectiveness of the measurements by reducing the play of the probe in the casing as well as accommodating larger displacements prior to shearing.

Borehole backfilling is also a common problem for most field personnel. Because the casings are fully sealed and installed into relatively deep boreholes, considerable buoyancy effects often occur. The natural tendency is to place a weight on the top to counter these effects, but this can result in buckling of the casing. It is best to first fill the casing with water and then slowly (and carefully) lower either a chain or drill rods into the casing. Care must be taken when lowering drill rods into the casing so as not to crack the end cap. By providing weight on the base, the casing is forced into extension and remains straight at all times. Furthermore, with respect to backfilling, the relative stiffness of the soil being monitored must also be considered. A soft backfill (relative to the host soil) will allow displacements to occur into the backfill that may not necessarily be registered by the casing. This is also true for the use of sand backfill, which is generally poured from the ground surface into the annular spacing, resulting in a loose density that can adjust as displacements take place. For these reasons, only a stiff cement-bentonite grout should be used for backfilling. A stiffer column of backfill may to a certain degree mask the exact location of the ground movements, but the total displacement will be illustrated in the cumulative displacement chart. Most inclinometer casing manufacturers have recommended backfilling procedures and grout mixtures. The last concern with respect to installation comes from an inability to found the inclinometer casing into a stable, unaffected stratum. Because all readings are relative to a fixed point (typically the base of the casing), it is essential that the fixed point not move to ensure accurate readings. Any movement of the inclinometer base will hide actual movements. This is primarily a problem in very deep-seated slope failures that may be driven by weak layers in the bedrock or during the instrumentation of soldier piles. For this reason, inclinometers should be constructed a minimum of 3 m into a competent bedrock with the competency confirmed using rotary coring methods.

Errors associated with probe usage stem from the relatively delicate nature of the instrument in general. Users must be careful at all times to ensure that connector pins are aligned correctly and that the probe is protected from shock. Wheels should be regularly inspected for tightness and the probe sent for regular calibration at least once per year. The sensitivity of the probes must be considered when planning a monitoring program. No two probes are alike and therefore the same probe must be used throughout the life of a given casing. Therefore, sending the probe(s) to be used on a long-term project to the manufacturer for calibration and maintenance prior to the start of the project is recommended.

Summary

The selection of instrumentation used on any project should be a function of the soils or bedrock (geology) encountered at the site; the anticipated changes in pore-water pressure, stress or displacements, and the level of risk associated with failure. Other factors that must be considered include the working range (pressure / strain / temperature) of the various instruments, ability to measure the desired response of the ground, and the ability of the field staff to install and read the instruments correctly. Finally, when considering risk, the selection of instruments must take into account the response time of the instrument as well as whether they are capable of being automated and report to all stake holders in a timely and efficient manner.

Cross-References

▶ Aerial Photography
▶ Bedrock
▶ Boreholes
▶ Borehole Investigations
▶ Casing
▶ Cement
▶ Clay
▶ Cohesive Soils
▶ Collapsible Soils
▶ Compaction
▶ Compression
▶ Cone Penetrometer
▶ Consolidation
▶ Dams
▶ Deformation
▶ Designing Site Investigations
▶ Dewatering
▶ Effective Stress
▶ Engineering Properties
▶ Excavation
▶ Expansive Soils
▶ Extensometer
▶ Fluid Withdrawal
▶ Foundations
▶ Geohazards
▶ Geostatic Stress
▶ Ground Anchors
▶ Groundwater
▶ Grouting
▶ Hydrocompaction
▶ Inclinometer
▶ Landslide
▶ Laser Scanning
▶ Lidar
▶ Monitoring
▶ Noncohesive Soils
▶ Normal Stress
▶ Photogrammetry
▶ Piezometer
▶ Pore Pressure
▶ Pressure
▶ Rock Properties
▶ Shear Strength
▶ Shear Stress
▶ Site Investigation
▶ Soil Properties
▶ Strain
▶ Strength
▶ Stress
▶ Subsidence
▶ Surveying
▶ Thermistor
▶ Tiltmeter
▶ Tunnels

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International Association of Hydrogeologists (IAH)

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Definition

A professional association for scientists, engineers, water managers, and others working in the fields of **groundwater resource planning**, management, and protection.

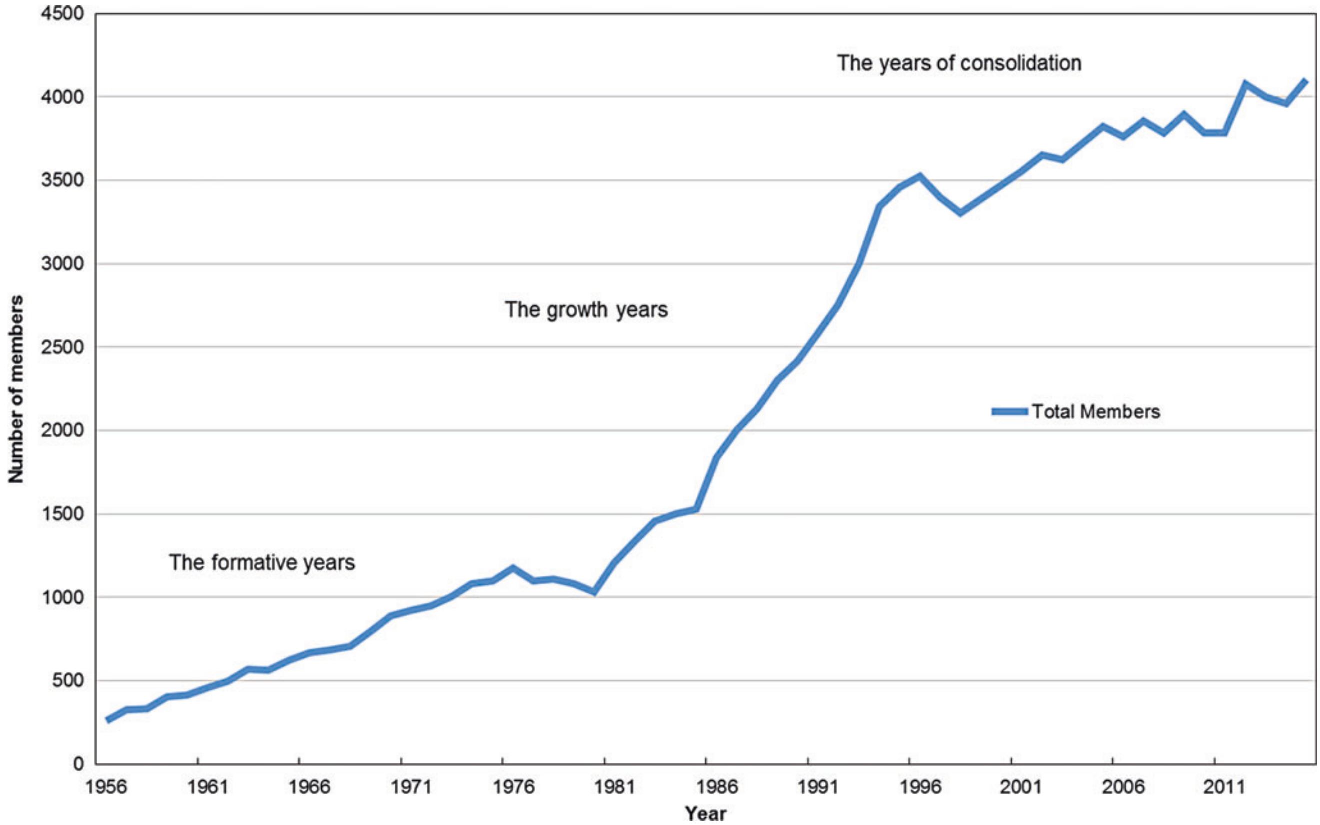
The International Association of Hydrogeologists was founded in 1956 at the 20th International Geological Congress. IAH celebrated its 60th anniversary in 2016. The association held its first Congress in Paris in 1957 and its 43rd in Montpellier in 2016 (Struckmeier et al. 2016).

Like all professional associations, from the beginning its governing council wished to publish the results of relevant research in books and journals. Book publication began in the 1960s with monographs and proceedings from congresses and continues to this day. The ambition for a scientific journal came to fruition in 1992 when ***Applied Hydrogeology*** began publication. The title was changed to ***Hydrogeology Journal*** in 1995 and publication moved to Springer in 1997, since which time it has grown into the present high quality journal of eight issues per year with around 130 articles and nearly 2000 pages and a steadily increasing impact factor.

Much of the scientific work of the association is undertaken by its Commissions and Networks. One of the earliest (established in 1959), longest running, and most productive was that devoted to **Hydrogeological Maps**, and the International Legend marked an important contribution which still serves as a model throughout the world. Other long-established commissions on **Mineral and Thermal Waters** and on **Karst** both date from 1968 and have been major contributors to IAH's congresses and book publications. More recent Commissions for **Groundwater Protection**, **Urban Hydrogeology**, Managing **Aquifer Recharge,** and Transboundary **Aquifer Management** were established and even newer Commissions on Groundwater and Climate Change, Groundwater and Ecosystems, and Groundwater and Energy reflect the changing directions of hydrogeological science (https://iah.org/).

From small beginnings, IAH has grown into a worldwide association which now has around 4100 members in 130 countries and 45 national chapters. As shown in the accompanying figure, membership grew quite slowly during the 1960s and 1970s and then more rapidly (Fig. 1, Struckmeier et al. 2016). Membership more than doubled during the 1980s as the importance of mapping and investigating groundwater resources and developing new supply sources became more widely recognized and the number of groundwater professionals increased. Congress themes, book titles, and IAH commissions formed during this time largely reflected this emphasis. More recently, the increasing importance of managing groundwater resources and protecting aquifers from pollution has produced evolving scientific directions in hydrogeology which are reflected in the titles of IAH's congresses and books and in the present IAH mission *"to further the understanding, wise use and protection of groundwater resources throughout the world."*

To reflect the changing management and administrative requirements of a growing global association, IAH became



International Association of Hydrogeologists (IAH), Fig. 1 IAH membership growth, 1956–2015

incorporated as a company and registered as a charity in the UK in 2000. The association is governed by an elected council of 13 and administered by a small Secretariat based in the UK. IAH has been a member of the International Union of Geological Sciences since 1964 and is a member of the World Water Council, Global Water Partnership and UN-Water, and long-term partner in UNESCO's International Hydrological Programme, all of which help toward achieving the association's present mission.

Cross-References

- ► Aquifer
- ► Groundwater
- ► International Association of Engineering Geology and the Environment (IAEG)
- ► International Society for Rock Mechanics (ISRM)
- ► International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)
- ► Karst

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International Society for Rock Mechanics (ISRM)

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Definition

A nonprofit scientific association devoted to all studies relative to the physical and mechanical properties of rocks and rock masses and to their applications to engineering.

The International Society for Rock Mechanics was founded in Salzburg in 1962. Its foundation is mainly owed to Prof. Leopold Müller who became the first ISRM president.

The Society is a nonprofit scientific association, supported by the fees of the members, by revenues from technical publications, by unrestricting grants, and by other sources of revenue as approved by the board.

Membership of the Society consists of individual members affiliated through national groups, corporate members, and corresponding members. In 2017 the Society had 8,000 members and 61 national groups.

The field of rock mechanics is taken to include all studies relative to the physical and mechanical behavior of rocks and rock masses and the applications of this knowledge for the better understanding of geological processes in the fields of engineering.

The main objectives and purposes of the Society are:

- To encourage international collaboration and exchange of ideas and information between Rock Mechanics practitioners
- To encourage teaching, research, and advancement of knowledge in rock mechanics
- To promote high standards of professional practice among rock engineers so that civil, mining, and petroleum engineering works might be safer, more economic, and less disruptive to the environment

The main activities carried out by the Society in order to achieve its objectives are:

- Holding international congresses at intervals of 4 years
- Sponsoring a coordinated program of international symposia, regional symposia and specialized conferences on topics in rock mechanics and rock engineering, organized by national groups of the Society
- Operating commissions to study and report on matters of concern to the Society
- Encouraging the preparation of internationally recognized nomenclature, codes of practice, standard tests and procedures
- Promoting international cooperation by distributing news to members, publicizing bibliographic and other information services, books and periodicals, and new products and services pertaining to rock mechanics and rock engineering
- Cooperating with international bodies whose aims are complementary to those of the Society
- Awarding prizes, namely the prestigious Rocha Medal for an outstanding doctoral thesis, every year, and the Müller Award in recognition of distinguished contributions to the profession of rock mechanics and rock engineering, once every 4 years

The Society is governed and administered by a council, a board, and a secretariat. The council is the supreme body of the Society. The board administers the affairs of the Society, with the assistance of the secretariat, and in accordance with policies established by the council. The current president of the board is Dr. Eda Quadros, from Brazil.

The ISRM Secretariat has been headquartered in Lisbon, Portugal, at the Portuguese National Laboratory for Civil Engineering – LNEC - since 1966, date of the first ISRM Congress, when Prof. Manuel Rocha was elected as president of the Society.

The ISRM Suggested Methods, produced under the coordination of the Commission on Testing Methods, represent a major contribution to the rock mechanics fraternity (Hudson and Ulusay 2007; Ulusay 2015).

The 50th Anniversary Commemorative Book 1962–2012 (Hudson and Lamas 2012) provides full details about the history and the activities of the Society.

Cross-References

- ▶ International Association of Engineering Geology and the Environment (IAEG)
- ▶ International Association of Hydrogeologists (IAH)
- ▶ International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)
- ▶ Rock Mechanics

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International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

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Definition

The International Society for Soil Mechanics and Geotechnical Engineering.

ISSMGE was founded in 1936 at the First International Conference on Soil Mechanics and Foundation

Engineering, held at Harvard University in Cambridge, MA, USA, at which Karl Terzaghi, widely regarded as "the father of soil mechanics," gave a plenary presentation, and where 2 years later, he would become a member of the civil engineering faculty. The name ISSMGE was adopted in 1997 to more accurately reflect the activities of the society. ISSMGE is a professional body representing about 90 member societies and approximately 20,000 individual members who are practicing engineers, academics, and contractors worldwide who actively participate in geotechnical engineering.

ISSMGE (2016) is a member organization of the Federation of International Geo-Engineering Societies (FedIGS 2016), along with International Association for Engineering Geology and the Environment (IAEG 2016), International Society of Rock Mechanics (ISRM 2016), and International Geosynthetics Society (IGS 2016).

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J

Jacking Test

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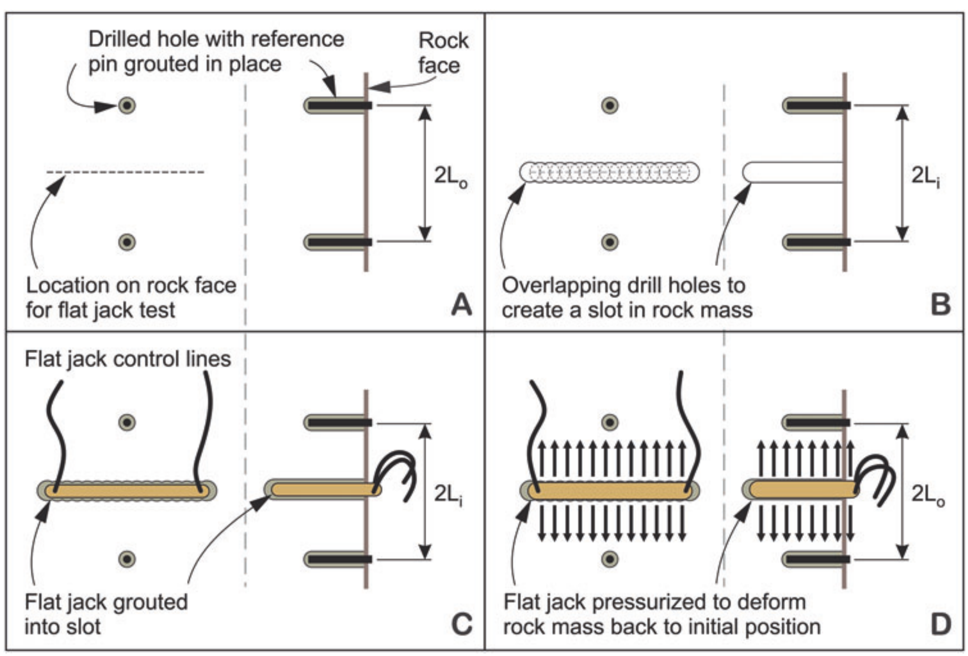
Definition

A jacking test in rock mechanics is used to apply a stress and measure the resulting rock-mass deformation.

Jacking test results are used for estimating rock mass performance and its influence on engineered works, such as concrete dams and pressure tunnels, and for estimating *in situ* stress conditions for design and construction management. If the stress-strain behavior complies with Hooke's Law, then the slope of the stress-strain relationship is the *modulus of elasticity*; however, if it does not comply, then the slope is the modulus of deformation (Goodman 1980). Four types of jacking tests are performed for rock mechanics applications: plate jacking, borehole jacking, radial jacking, and flat jacking (ASTM 2016).

Plate jacking tests use circular, steel plates, hydraulic rams, and suitable reactions, with stable frames for attaching deformation monitors, typically linear-variable-differential transformers (LVDTs). Measured values are the radius of the plate (*pr*), the *stress* applied by the plate (*ps*) (load divided by plate area), and the mean displacement of the plate (*pd*) corrected for any rotation (Kavur et al. 2015). The modulus of elasticity (*E*) may be calculated by assuming that the rock is a homogeneous infinite half-space of elastic isotropic material, and specifying a value for *Poisson's ratio* (*v*) using



Jacking Test, Fig. 1 Four steps of flat jack test. (**a**) Installing reference pins and measuring distances. (**b**) Drilling overlapping hole to create a slot and measuring the corresponding pin distances. (**c**) Installing the flat jack and fixing it in place with neat cement grout. (**d**) Pressurizing and depressurizing the flat jack while measuring corresponding pin distances

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$$pd = \frac{C\,ps(1 - v^2)pr}{E}; E = \frac{C\,ps(1 - v2)pr}{pd} \qquad (1)$$

where C is a boundary-condition constant ($C = \pi/2 = 1.57$ for a perfectly rigid plate; $C = 1.70$ for a flexible plate).

A **borehole** jacking test uses a tool with diametrically opposed curved platens that fit into a ~ 76 mm diameter borehole. A two-ram jack presses the platens into the borehole wall, LVDT sensors record platen movement, and a gauge records pressure. A radial jacking test is similar to the borehole jacking test except it is performed inside a suitably sized tunnel, drift, or adit. Measurements are made in diametrically opposed pairs around the circumference of the excavation.

A flat jack consists of two sheets of steel welded around the perimeter, filled with oil that can be pressurized. The test consists of four steps (Fig. 1): (1) installation and measurement of one or more pairs of pins that will be on opposite sides of a slot for the flat jack; (2) drilling overlapping holes to create a slot for the flat jack; (3) installing, seating, and cementing the flat jack; and (4) monitoring the distance between measurement-pin pairs while the flat jack is pressurized and depressurized. Flat jack tests are performed in slots at different orientations to allow in situ stresses to be estimated by recording the orientations of the slots and measurement-pin pairs, and the pressure required to deform the rock to its preslot position.

Cross-References

- ▶ Deformation
- ▶ Hooke's Law
- ▶ Modulus of Deformation
- ▶ Modulus of Elasticity
- ▶ Rock Mechanics
- ▶ Strain
- ▶ Stress

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K

Karst

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Definition

Terrain, mostly on limestone, distinguished by underground drainage and dissolution cavities. Natural ground cavities and the sinkholes that are formed largely when soil cover is washed down into them constitute the main karst geohazards.

Introduction

Karst terrains provide construction engineers with some of the most variable and unpredictable ground conditions found anywhere. Karst is distinguished by the solubility in water of its bedrock (mainly limestone, but also gypsum and salt) and is defined by its underground drainage through fissures and caves (Jennings 1985; Ford and Williams 2005). Consequently, karst ground can vary, on scales of meters or hectares, between strong, solid limestone, and voids that are either open or filled with soft sediment. Areas of limestone karst occur in all parts of the world, with the karst geohazard of particular importance in southern China, Slovenia, Croatia, and the eastern USA (Doctor et al. 2015), whereas it is of lesser significance within most of Great Britain (Cooper et al. 2011).

The scale of the karst geohazard is greatest in areas with warmer and wetter climates, where dissolution levels are higher in waters enriched with biogenic carbon dioxide. An engineering classification of karst therefore reflects this climatic influence in its recognition of five classes of karst ground conditions (Waltham and Fookes 2003). These classes can only represent generalities in their ground conditions, and the geohazard is normally best assessed by identifying the scale of the three key factors of significance to construction works. These are sinkhole development (mainly in the soil profile), pinnacled rockhead, and ground cavities (Waltham 2016; Waltham et al. 2005; Fig. 1).

The Sinkhole Hazard, and the Means of Reducing It

Sinkholes (or dolines in most geomorphologists' parlance) are the ubiquitous feature of karst terrains, formed at points where water sinks underground. They can form by dissolution and collapse of the bedrock, but dissolution is extremely slow and collapses are rare. By far the greatest geohazard is presented by the huge numbers of sinkholes developed or developing within soil profiles that overlie fissured limestone. These are known as subsidence sinkholes and are developed where the soil cover is washed downward by any form of drainage flow into the limestone fissures, in a process known as suffosion (Waltham et al. 2005).

Individual subsidence sinkholes are commonly 2–50 m across and 1–15 m deep, typically with a diameter that is less than three times the soil depth. Within the bedrock, the cavity that is the cause of the sinkhole, and is the outlet for the water and soil, could be a fissure just a few centimeters wide or a shaft a meter or more across at a fissure intersection. Of the two types of subsidence sinkhole, a dropout sinkhole develops in a cohesive soil that can bridge over a soil cavity before its sudden collapse, whereas a suffosion sinkhole develops more slowly by continuous slumping. Many sinkholes fit between these extremes and develop in stages lasting hours or weeks that increase depth and diameter.

Though many new subsidence sinkholes occur during major rainfall events, it is well documented that the vast majority of new sinkholes are caused by man's activities (Newton 1987; Waltham et al. 2005; Waltham 2009; Guttiérez et al. 2014). New failures are caused either by increased inputs of water, normally by inadequate, changed, or broken drainage systems, or by water table decline that has

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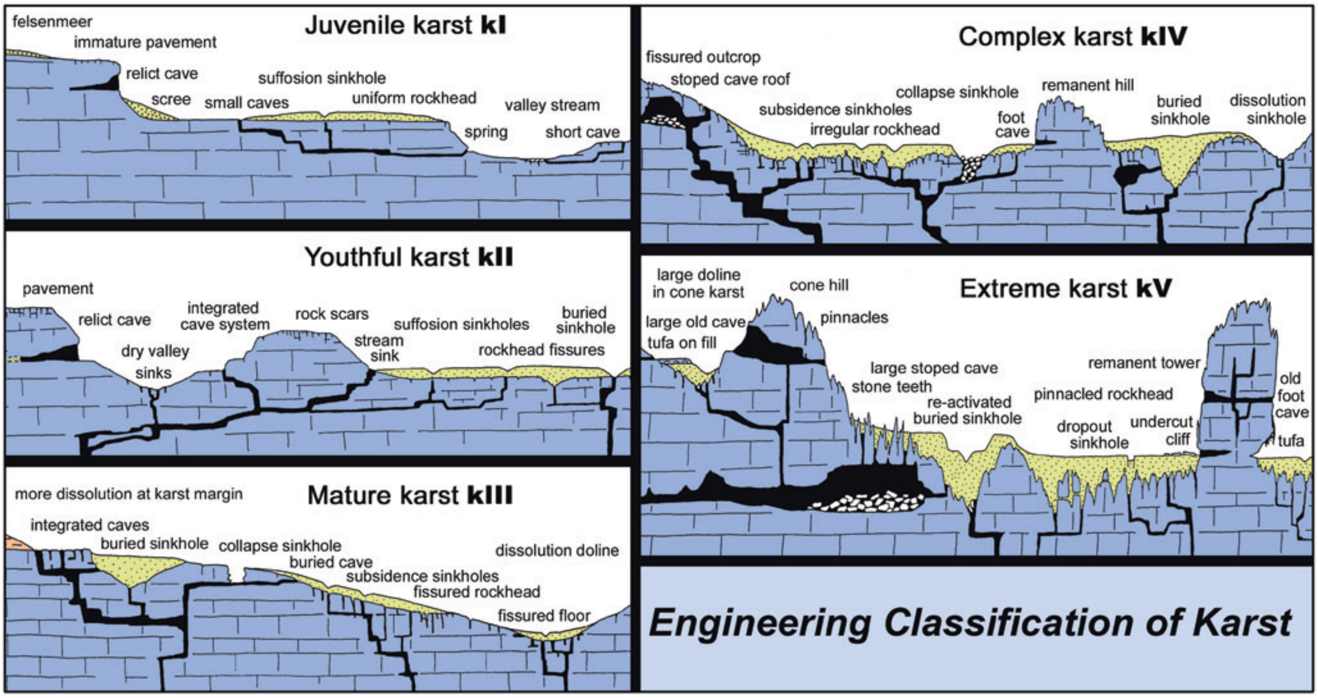


Karst, Fig. 1 Variation in karst morphology broadly recognized in an engineering classification that recognizes increasing sizes and numbers of caves, sizes and numbers of sinkholes, frequency of new sinkhole events, topographic relief, and rockhead relief in the increasingly more mature karst terrains (After Waltham and Fookes 2003)

a comparable drawdown effect or can induce failure by the loss of buoyancy support. A fissure large enough to swallow soil takes thousands of years to be formed by rock dissolution, but a new input of water, failure of a soil arch, or washing out of a choke can cause a new sinkhole to develop in the soil profile within hours or days. Prediction of new sinkhole locations is impossible, except to recognize that most will occur where there is a new water input to the soil cover. Short of stripping away the soil cover, open fissures, and potential sinkhole sites cannot be determined by any practicable level of ground investigation.

A stable situation of rainfall filtering through natural ground into myriad fissures in underlying limestone is easily disturbed when a built structure concentrates run-off into a few perimeter points, each of which then becomes a potential site for a new sinkhole. Roads (with their marginal run-off), railways, buried pipelines (with their granular pipe seating along their floors), and any unlined ditches all constitute effective diversions of drainage. The most cost-effective means of minimizing the sinkhole hazard is thorough control of surface water, to ensure that as little of it as possible can ever collect at points where it can sink underground and wash any soil cover into karstic cavities within underlying limestone (Waltham 2016).

Careful design of good drainage is essential in karst terrains. Built drains, as efficient and comprehensive as practicable, should ideally carry run-off water away from the site. Retention ponds that lose water into the soil cover are only appropriate sited away from structures; a guideline minimum distance is double that of the local soil thickness to allow for the flared sides of any new sinkhole and also some lateral flow along rockhead. Soakaway drains are best avoided in karst, or can be cased into bedrock in order to avoid flow through the soil cover.

Inevitably, almost any construction project disturbs soil drainage and accounts for many new sinkholes during or soon after the period of site activity. The hazard can only be reduced by *ad hoc* drainage control that is site-specific and primarily avoids locally increased infiltration to the soil.

Subsidence sinkholes are commonly induced by any water table decline that increases downward flow of soil drainage. This can induce clusters of new sinkholes across wide areas, especially where the water table declines past the rockhead, so that minimal, lateral groundwater flow is replaced by focused, downward flow at the critical points of soil loss into bedrock fissures. The two main reasons for water table decline are excessive abstraction for water supply and dewatering around mines and quarries (Fig. 2). The only means of reducing new sinkhole occurrences is to allow the water table to recover, and that is normally constrained by wider economic considerations.

Sinkhole remediation is rarely easy. Simple backfilling is invariably followed by reactivation when the fill is itself washed downward. Stability can be achieved by choking the sinkhole with blocks of rock too large to move downward, though this commonly requires exposure of the bedrock fissures to be successful, and is preferably accompanied by diversion of immediate drainage.

Karst, Fig. 2 Failure of a road in Pennsylvania where a subsidence sinkhole developed as soil was washed down into a fissured karst limestone after the local water table had been lowered by pumped drainage of a nearby quarry

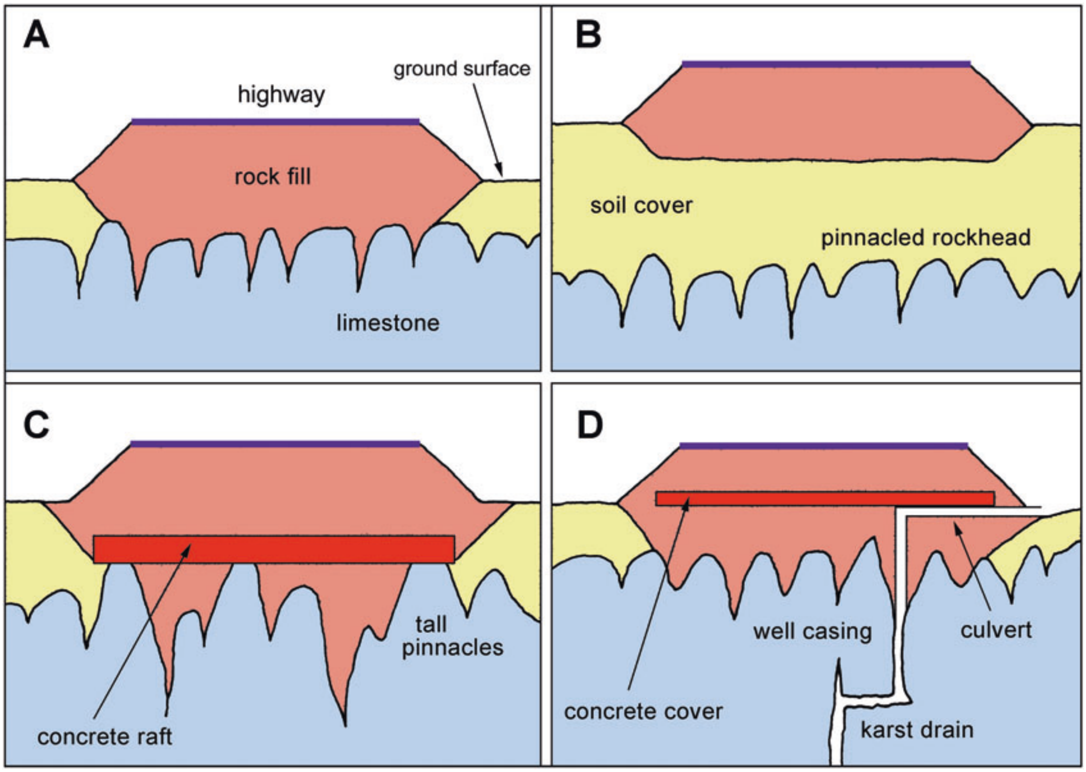
Foundations on Pinnacled Rockhead, and the Means of Ensuring Stability

The surface profile of limestone bedrock within in a karst terrain is typically extremely irregular. A glaciated karst may have only open fissures between flat expanses of limestone pavement, but a tropical karst is normally fretted into columns, gullies, and loose blocks on a spectacular scale (Waltham and Fookes 2003). When buried by a deep soil, rockhead morphology is totally unrecognizable from surface observation alone. In areas of pinnacled rockhead, boreholes only meters apart may reach bedrock at depths varying by tens of meters, where they meet either the crest of a pinnacle or the floor of a dissolutionally widened fissure at its side.

Structures can be founded on buried pinnacles (Sowers 1996; Waltham et al. 2005). The proviso is that pinnacles must be proven to be large enough and stable and are not detached blocks with lateral support only by soft soil. Assessment may require partial exposure, generally accompanied by multiple boreholes. Concrete rafts or beams can bridge between the pinnacles of strong limestone (Fig. 3), and coarse aggregate mats formed over a mixture of soil and rock can achieve stability by distributing loads across multiple pinnacles (Lei and Liang 2005). The use of driven piles requires considerable care in pinnacle karst as they may be bent, deflected, or inadequately seated on steeply inclined rock surfaces on the sides of buried pinnacles.

Unseen Cavities, and the Means of Detecting Them

By definition, karst terrains contain caves, cavities, voids, and fissures. These are filled with air, water, or soft sediment, and have virtually zero bearing capacity. They may represent potential sites for the development of subsidence sinkholes where they can receive debris and sediment that is washed into them from the soil profile. They can also evolve into collapse sinkholes where the rock arches or bridges above them are weathered and eroded to the point of failure. Though collapse sinkholes are a feature of many karst terrains, their natural development is over geological



Karst, Fig. 3 Design concepts developed for construction of a motorway across limestone karst in southern China. A = coarse rock fill placed on exposed bedrock where soil cover is <3 meters thick. B = mattress of coarse rock fill placed on a thick soil cover. C = concrete raft that spans wide soil-filled fissures and hollows between footings on stable limestone pinnacles. D = culvert installed to maintain drainage into the natural sink within a karst depression (After Lei and Liang 2005)

Karst, Fig. 4 A new sinkhole nearly 50 meters across in central Turkey, which destroyed a road and a house when the ground dropped by more than a meter due to failure of a cave roof at considerable depth

timescales, so that new collapses are extremely rare (Fig. 4). Collapse induced by loss of buoyancy support, due to water table decline, can occur (Doğan and Yilmaz 2011), but is very unusual. The main geohazard is created by inadvertent structural loads being imposed on rock that bridges over an unseen cavity.

Natural caves can occur at any depth and to any size within karst terrains, but only the larger caves at smaller depths are relevant to engineering works on the ground surface. The load-bearing capacity of the rock roof over caves varies enormously, as it depends on cave width, cover thickness and the mass strength of the fractured rock (Waltham and Lu 2007). A very rough guideline is that a roof thickness greater than half the cave width is stable for most structural loading in strong limestone, but each situation requires individual assessment.

Karst, Fig. 5 Construction on a concrete slab bridging between strong and stable limestone pinnacles without the inconvenience of a soil fill to obscure the ground conditions in deeply dissected limestone karst in the Philippines

The only feature predictable about caves is that they are unpredictable, and the only certain means of proving intact rock beneath a built structure in a karst terrain is with boreholes. A guideline is that these should probe to a depth in intact rock equal to the likely cave width, and that dimension can only be roughly assessed through local knowledge of the karst (Waltham 2008).

Geophysics offers some prospect in cavity searches (Waltham et al. 2005). Microgravity surveys are probably the most reliable because a cave creates a clear negative anomaly even if it is filled with water, breakdown or sediment. The disadvantage of microgravity is the high cost, as closely spaced recordings are required for realistic interpretation of the data, so that the method is applicable only at limited numbers of sites. Resistivity surveys can be economically viable over large sites or along transport corridors. However, they suffer from the fact that a cavity filled with clay or water creates a negative anomaly whereas a dry, open cavity creates a positive anomaly. Consequently, ground with both open and filled fissures tends to cancel out its own anomalies, and there have been many cases where the surveys have proved to be unhelpful or even misleading. Resistivity modelling in 3D increases the cost but on a small site can provide results that are more reliably interpreted than 2D surveys. Cross-hole seismic tomography has also produced useful results where a borehole network is available. Depth limitations on ground penetrating radar generally restrict its use in karst.

Any cavity of significant size found with inadequate rock cover beneath a site for load-bearing foundations requires attention. Filling with mass concrete can be the simple remedy, but may be difficult where laterally extensive cavities can swallow huge quantities of injected fill or where a cave's floor of soft sediment cannot adequately support the placed fill. Bridging a cavity with beams that span between solid footings (Fig. 5), or with piles that reach sound rock beneath a cave can be successful in specific cases. In some situations relocation of the structures, to avoid known cavities, can be the best or even the only option.

Summary

The main geohazard in karst is created by the development of new sinkholes within the soil cover, largely when and where the drainage has been disturbed. They are related to ground cavities, which form a second, though smaller, geohazard in their own right. There can be no rigid rules concerning the scale, methods, and detail of ground investigation on karst. Each site on cavernous ground is different and requires individual assessment to a level that provides sufficient confidence that built structures will retain integrity. Construction projects on karst should proceed only when the extremely variable ground conditions are fully appreciated.

Cross-References

▶ Biological Weathering
▶ Dissolution
▶ Evaporites
▶ Geohazards
▶ Limestone
▶ Sinkholes
▶ Subsidence
▶ Voids

K

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L

Lacustrine Deposits

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Synonyms

Lake deposits

Definition

Lakes are dynamic and complex systems, whose depositional processes and sedimentation are affected by climatic and tectonic events (Scott et al. 2012 and the references herein). In addition, the great physicochemical variability among lakes, in terms of origin, size, morphology, catchment size, and water biochemistry influence the nature and the rate of lacustrine deposits' formation (Schnurrenberger et al. 2003). The lake system, in comparison with other depositional environments such as rivers, is characterized by low energy (Rust 1982). This depositional condition allows the fine particles to settle out with a variable organic component, making lacustrine sediments prevalently characterized by silt, fine sand, and clay mixtures. Nevertheless, near the margin, the lake system presents higher-energy depositional environments, such as alluvial and fluvial-lacustrine features (i.e., alluvial fans, alluvial floodplain and deltas). For these reasons, sandy and gravelly interbedded deposits can occur within the finer sediment bodies, especially in the proximal areas of a lake system, where the abovementioned boundary conditions are present (Rust 1982; Scott et al. 2012). Due to the high variability of physicochemical properties of the lake systems distributed worldwide, many lacustrine deposits vary from meters to hundreds of meters in depth comprising fine soils that range from sand to clay including organic clay. The mechanical properties of such soils are quite variable, depending on the dominant soil fractions, the type of clay mineral content and the organic matter. Commonly, such soils show high compressibility and low mechanical resistance that can induce subsidence under static loading (that is a plastic progressive settlement) and liquefaction under dynamic loading when the groundwater is shallow.

Engineering Properties of Fine Deposits in Lacustrine Environment

In terms of engineering geological characterization, the focus must be put on the properties of fine and very fine-grained soils (clay and silt), although frequently sand and locally gravel can be interbedded within the lacustrine soils. Thus, hereinafter, the most relevant parameters that affect the main properties of fine soils (Table 1) are introduced and their reference values provided.

Fine soils are characterized by grain dimensions less than 0.06 mm. Lacustrine deposits therefore are commonly made up of clay and silt mixtures that can be both inorganic and organic. Although silt consists of very small grains, it is characterized by grains not flaky particles in contrast to clay. This implies that silt develops fine deposits with granular character. Nonetheless, when silt is mixed with clay it can be described through the same parameters as clay, whereas when it is mixed with sand and gravel it contributes to reduce permeability because the small silt grains find places within the gravel and sand voids thereby increasing the packing. Nevertheless, silt can be described through granular soil parameters.

For sand and gravel, the most relevant property is the relative density (D_r). It is the index that quantifies the degree of packing between the loosest and the densest possible state of coarse-grained soils. It is defined as follows:

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Lacustrine Deposits, Table 1 Grain size dimensions of soils and ranges of values of natural γ unit weight and the void ratio e

Term	Particle size (mm)	Equivalent soil grade	γ (kN/m^3)	Void ratio e
Coarse-grained	2–60	Gravel	18–23	0.3–0.7
Medium-grained	0.06–2	Sand	16–21	0.3–1.0
Fine-grained	0.002–0.06	Silt	16–21	0.5–1.0
Very fine-grained	<0.002	Clay	14–21	0.4–2.3
		Peat	10–13	3.0–19.0

Lacustrine Deposits, Table 2 Medium to coarse-grained size classification based on D_r and corresponding range of values of the blow count of the Standard Penetration Tests N_{spt} and the friction angle ϕ

D_r	Description	N_{spt} (30 cm)	Friction angle (deg)
0–15	Very loose	0–4	25–28
15–35	Loose	4–10	29–32
35–65	Medium dense	10–30	33–35
65–85	Dense	30–50	36–40
85–100	Very dense	Over 50 up to 100	41–45

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \quad e = \frac{V_v}{V_s} \tag{1}$$

where e_{max} is the maximum void ratio (loosest condition), and e_{min} is the minimum void ratio (densest condition), and e is the measured void ratio. The void ratio (e) is defined as the ratio between the void volume and the solid component. Table 2 shows the relative density range values for the loose or dense state of the soil (Terzaghi and Peck 1948) as well as some other typical ranges of values of physical and mechanical properties. In lacustrine soils, D_r values are less than 65 but can vary from site to site.

Furthermore, the main relevant resistance parameter of granular soils (gravel, sand, and silt) is the shear strength according to the Mohr-Coulomb failure criterion for cohesionless soils (Coulomb 1776):

$$\tau_f = \sigma' \bullet \tan(\varphi') \tag{2}$$

where σ' is the effective stress and φ' is the internal friction angle. Suggested φ' values are listed in Table 3.

In contrast, when the fine fractions are considered (silt and clay), they are classified based on the Atterberg limits and the Casagrande plasticity chart (Fig. 2). Clay soils (whose particles size is less than 2 μm) have flaky particles to which water adheres, thus the plasticity is relevant together with the consistency limit in order to estimate the swelling (relevant phenomena in expansive clay) and the compressibility (heavily affecting organic clay and quick clay) capacity under static conditions. In the dynamic behavior of fine soils, the plasticity index affects the shape of the Damping function with the shear strain D(γ) and the Shear Modulus Reduction Curve $G(\gamma)/G_{max}$. The Atterberg limits are strictly related to the mineral content of the clay fraction and consists of threshold values of water content between different physical states of the soil (Table 4):

Lacustrine Deposits, Table 3 Typical values of φ′ for granular soils

Soil type	φ′ (deg)
Gravel with some sand	34–48
Sand with rounded grains	
Loose	27–30
Medium	30–35
Dense	35–38
Sand with angular grains	
Loose	30–35
Medium	35–40
Dense	40–45
Silt	26–35

Lacustrine Deposits, Table 4 Typical limit liquid, plastic limit, and shrinkage limit values of different clay minerals

Soil prevalent mineral component	Liquid Limit – LL (%)	Plastic Limit – PL (%)	Shrinkage Limit – SL (%)
Montmorillonite	100–900	50–100	8.5–15
Nontronite	37–72	19–27	
Illite	60–120	35–60	15–17
Kaolinite	30–110	25–40	25–29
Hydrated Halloysite	50–70	47–60	
Attapulgite	160–230	100–120	
Chlorite	44–47	36–40	
Allophane (undried)	200–250	130–140	

1. The liquid limit w_{LL} marks the passage from plastic to liquid state.
2. The plastic limit w_{PL} marks the passage from semisolid to plastic state.
3. The shrinkage limit w_{SL} marks the passage from solid to semisolid state.

The range of water contents over which the soil deforms plastically is known as the plasticity index:

$$I_p = w_{LL} - w_{PL}$$

Another important parameter especially when silt and clay mixtures are detected is the activity A, that is:

$$A = \frac{I_P}{\text{Clay fraction } (\%)}$$

This latter ratio A was introduced by Skempton (1954), who noted that for soils with a particular mineralogy the plasticity index is linearly related to the amount of the clay fraction. The proportion of clay mineral flakes (<2 μm size) in a fine soil affects its current state, particularly its tendency to swell and shrink with changes in water content. The degree of plasticity related to the clay content is called the activity of the soil. Thus, clay can be classified according to its I_P and A values as shown in Table 5. The A values of a soil are influenced by the compositional minerals. Some reference values for A can be suggested for the commonest clay minerals: Smectite, 1–7; Illite 0.5–1; Kaolinite 0.5; Allophane 0.5–1.2.

Lacustrine Deposits, Table 5 Soil classification according to the activity index A and the plasticity index I_p

I_P	Soil property	A	Soil property
0–5	Not plasticity	<0.75	Inactive
5–15	Low plasticity	0.75–1.25	Normal
15–40	Intermediate plasticity	>1.25	Active
>40	High plasticity		

Lacustrine Deposits, Table 6 Some guide values of the consistency index *Ic*, the undrained shear strength s_u measured through the field vane test and blow count numbers N_{spt} from Standard Penetration Test

I_c value	Soil classification	Undrained shear strength s_u (kPa)	N_{spt}
<0.25	Very soft	<12	0–2
0.25–0.50	Soft	12–25	2–4
0.50–1.00	Medium	25–50	4–8
1.0–2.0	Stiff	50–100	8–16
2.0–4.0	Very stiff	100–200	16–32
>4.0	Hard	≥200	Over 32

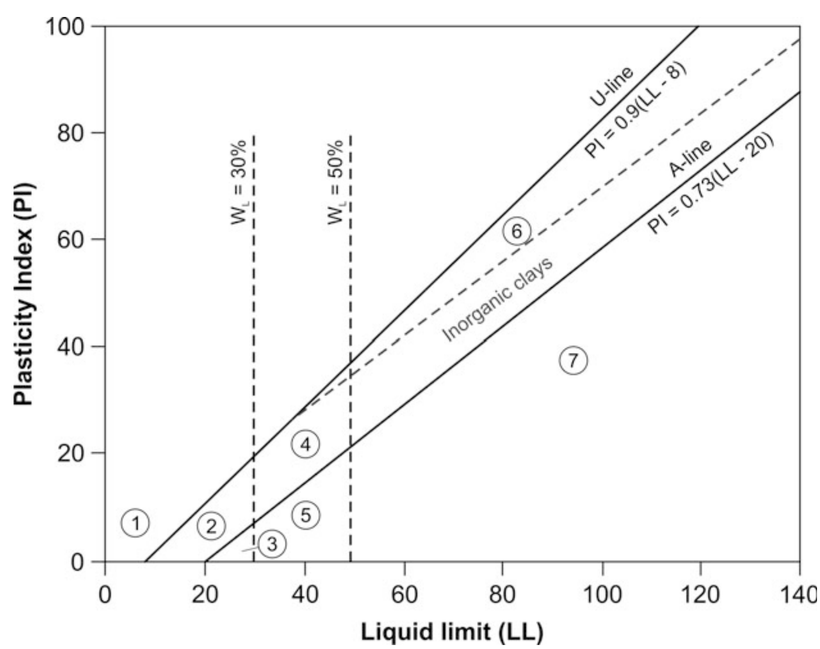
Another useful parameter to describe the mechanical properties of lacustrine soils is the consistency index I_c:

$$I_c = 1 - I_L \qquad \text{where} \qquad I_L = \frac{w - w_{PL}}{I_P}$$

In Table 6, the ranges of I_c values are reported together with the undrained shear strength s_u that represents the most relevant mechanical parameter for fine soils under undrained conditions. It represents the minimum value of fine soil resistance under static loading. Many lab and in-field testing methods have been conceived to measure s_u, such as the shear Vane test (VST), the undrained unconsolidated triaxial test (UU-TX), and the Piezocone Test (CPTu), among others. The lacustrine silt and clay range between 0.25 and 1.00 I_c values and accordingly show s_u less than 50 kPa and a number of blow counts less than 8. One example of lacustrine clay soils are the deposits of Mexico City. Mexico City lacustrine deposits are clay rich in allophane. This implies a large plasticity index, a low resistance, and a very low consistency.

In order to classify inorganic and organic fine soils, the Casagrande's plasticity chart is commonly used (Fig. 1). The soils with LL lower than 30% shows low plasticity; those with LL ranging from 30% to 50% show a medium plasticity, and when LL is higher than 50%, the soils show high plasticity. Furthermore, high compressibility is suggested when LL is greater than 50%, whereas low compressibility is commonly recognized when LL is lower than 50%. Accordingly, from

Lacustrine Deposits, Fig. 1 Casagrande plasticity chart. (1) cohesionless soil; (2) CL-ML: inorganic clay, low plasticity; (3) OL-ML: inorganic silt of low compressibility; (4) CI: inorganic clay, medium plasticity; (5) MI-OI: inorganic silt and organic clay, medium plasticity; (6) CH: inorganic clay, high plasticity; (7) MH-OH: inorganic silt and organic clay, high compressibility. A-line: separates the more clay like materials from silty materials, and the organics from the inorganics. U-line indicates the upper bound for general soils



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Lacustrine Deposits, Table 7 Darcy coefficient of permeability k (Data source: http://geotechdata.info/parameter/permeability.html)

Soil Description	k min (m/s)	k max (m/s)
Clayey gravel, clayey sandy gravel	5.00E-09	5.00E-06
Poorly graded sand, gravelly sand, with little or no fines	2.55E-05	5.35E-04
Silty sand	1.00E-08	5.00E-06
Clayey sand	5.50E-09	5.50E-06
Inorganic silt, silty or clayey fine sand, with slight plasticity	5.00E-09	1.00E-06
Inorganic clay, silty clay, sandy clay of low plasticity	5.00E-10	5.00E-08
Organic silt and organic silty clay of low plasticity	5.00E-09	1.00E-07
Inorganic silt of high plasticity	1.00E-10	5.00E-08
Inorganic clay of high plasticity	1.00E-10	1.00E-07
Compacted silt	7.00E-10	7.00E-08
Compacted clay	–	1.00E-09
Organic clay of high plasticity	5.00E-10	1.00E-07

Fig. 1 it is evident that the higher the liquid limit LL, the higher the compressibility and the lower the soil resistance. The organic clay shows very high plasticity and consequently high compressibility and the worst mechanical resistance among fine soils although the plasticity indexes are largely variable.

Furthermore, it is well known that clay and silt, due to their very low permeability, act as aquicludes or aquitards. Permeability in soils is defined as the property which permits the seepage of water (or other fluids) through its connected voids. It was found experimentally by Darcy that for laminar flow of water through soil, the rate of flow v is proportional to the hydraulic gradient i by the permeability coefficient k:

$$v = k \bullet i \tag{3}$$

According to Darcy's seepage simplified model, Table 7 shows the minimum and maximum values of Darcy coefficient of permeability k measured for fine soils. Gravel is highly permeable ($k \geq 4{\cdot}10^{-3}$ m/s), whereas clay is practically impermeable. Thus, lacustrine soils show a low permeability that reduces as the clay content increases.

Darcy permeability coefficient in fine deposits can be measured both in the laboratory with permeameter devices and in field testing, using the Lefranc test. Finally, in order to calculate the Darcy permeability coefficient of clay, a relation with the void ratio and the plasticity index has been proposed by Nishida and Nakagawa (1969) as follows:

$$e = (0.01PI + 0.05) \bullet (10 + \log_{10}(k)) \tag{4}$$

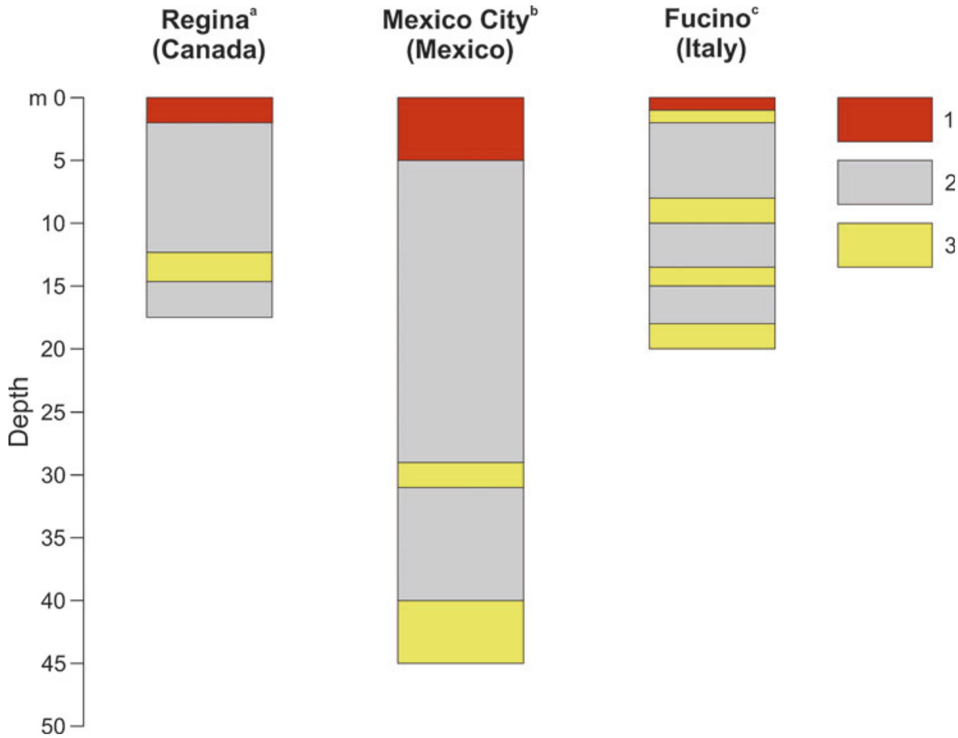
where PI is the plasticity index, k is the permeability index for clay (cm/s), and e is the void ratio of clay. Some updates for all types of inorganic silt and clay is provided by Chapuis (2012).

Examples of Lacustrine Litho-Technical Successions from Worldwide Sites

Lacustrine deposits are located in many different climatic areas and are often associated with many large urban areas. Due to their high compressibility and weak mechanical properties, they show many geotechnical problems: (1) large subsidence under constant load, such as in Taipei city (Taiwan, China), or Mexico city (Mexico); (2) large swelling-shrinkage phenomena that impacts those cities situated on glacial lacustrine deposits as in the case of the city of Regina (Canada) and the city of Edmond (Oklahoma, USA); (3) in seismic areas, the liquefaction hazard must be taken into account in soft fine deposits as well as the amplification phenomena, as in the Fucino Basin (AQ, Italy), Campotosto Lake (AQ, Italy); or the Peligna valley basin (AQ, Italy). Some physical and mechanical characterizations of the abovementioned lacustrine deposits are provided by Rodríguez-Rebolledo et al. (2015), Fredlund (1975), Calabresi and Manfredini (1976), Desiderio et al. (2012), Boncio et al. (2017), and Vessia and Russo (2017). From these case studies, some lithotechnical successions are summarized in Fig. 2.

The first case is from glacial Lake Regina lacustrine silt and clay, whose thickness ranges from 4 to 15 m (Fredlund 1975). They are made up of about 10 m of montmorillonite clay (about 80%) and overlie 5 m of silt (about 60%) that in turn sit upon till deposits comprising by clay, silt, and sand. The LL of Regina clay varies between 60% and 80%, the I_P varies between 30% and 60%: it is an inorganic clay with high plasticity. The activity coefficient is 0.7 with a very high swell potential. As the water content increases from 25% to 40%, the undrained shear strength decreases from 206 to 35 kPa. Conversely, the silty soils are characterized by the LL that varies between 30% and 40% and I_P varying between 15% and 20%. The Casagrande plasticity chart suggests the silt is inorganic silty with low compressibility.

The second case is the Lake zone of Mexico City. Here the lacustrine soils are covered by a dry crust of 5 m overlying three clay strata that are 5–40 m thick. The upper 25 m contains two clay units that overlie a harder layer of 2 or 3 m comprising sandy silt or clay, located at 30 m depth. Under this hard layer there is another lower clay stratum about 8 m thick. Here, the lacustrine sediments comprise highly plastic soft clay interbedded with layers of silt, sand, and sandy gravel of alluvial origin. According to field studies by Rodríguez-Rebolledo et al. (2015), this clay has a very high water content (with the void ratio varying between 7 and 10), the unit weight is 11 kN/m^3, a low cone resistance in the CPTu and practically a nil blow count in an SPT test. Undrained shear strength s_u increases with depth, with a mean value of 20 kPa in the upper part of the clay and 80 kPa at 30 m depth.



Lacustrine Deposits, Fig. 2 Typical successions of lacustrine deposits stratigraphy at three sites: (a) Regina city settled on the glacial Lake Regina (Canada) that are expansive clay; (b) Mexico City (Mexico) located upon very weak clay; (c) Fucino basin (Italy) made up of hundreds of meters of clay and silt mixtures. In legend: 1. Top soil and/or anthropogenic fill; 2. Lacustrine deposits; 3. Sandy and gravelly deposits

The final example, the Fucino basin, is characterized by hundreds of meters of lacustrine deposits, although near the borders of the basin (where several cities exist, i.e., Avezzano city that was destroyed by the 1915 Fucino Earthquake), gravel and sand are interbedded with fine silt and clay soils. A recent study by Boncio et al. (2017) measured the physical and mechanical properties of the upper 20 m for these lacustrine soils: the LL ranges from 27 to 47 and the I_p varies from 2 to 11, meaning that these soils vary from inorganic silt to organic clay with low plasticity and compressibility. The undrained shear strength is about 25 kPa and the shear velocity values increase with depth from 100 m/s to 250 m/s up to 14 m.

Summary and Conclusions

Lacustrine deposits are characterized by weak soils, with a very low permeability, sometimes expansive and very compressible especially when they are rich in organic matter and plastic clay minerals. A range of values of the most relevant engineering geological parameters were reviewed and further elaborated using three case studies. Examples considered show typical lithological successions and common physical and mechanical properties of lacustrine deposits, although in the case of expansive clay, they must be mechanically characterized through the unsaturated soil theory and its variables.

Cross-References

▶ Angle of Internal Friction
▶ Aquitard
▶ Clay
▶ Cohesive Soils
▶ Darcy's Law
▶ Engineering Properties
▶ Expansive Soils
▶ Glacier Environments
▶ Liquefaction
▶ Soil Laboratory Tests

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Land Use

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Definition

Land use encompasses the many ways in which humans around the world utilize the landscape and those resources present in the surface or near-surface environment. This can range from intense utilization associated with urbanization such as Paris to the preservation of natural conditions as found in the Wrangell-Saint Elias Wilderness in Alaska (Marker 2013; FAO 2015).

Introduction

Land use documents historical and current exploitation of the land by people and varies depending on characteristics such as natural features, land cover, and climate (NOAA 2015). The origins of many present-day urban centers can be traced to some landscape feature that made it a desirable place for people to build a settlement. Over time, a growing population expands the original settlement to towns, cities, megacities, and metropolitan areas as a consequence of landscape, climate, natural resources, cultural development, and historic events. Often, modern cities can trace their origins to landscape features such as coastal locations providing safe harbors (e.g., Cape Town, South Africa), favorable locations along navigable rivers (e.g., Bangkok, Thailand), nearby mountain passes allowing for trade routes (e.g., Kashgar, China), and proximity to fertile land for agriculture (e.g., Chicago, USA).

Managed land use is also recognized as a means for limiting losses from natural disasters and ensuring sustainability of communities (Mileti 1999). The planning process for an area with an expanding population can use the land use planning process along with construction codes, design requirements, development regulations, and public education to avoid the consequences that arise from ignoring geological hazards that exist within the local environment.

Influence of Engineering Geology on Land Use

A primary way that engineering geology facilitates land use in an expanding population center is through mapping and interpretation of the soil and rock that is present (Gonzalez de Vallejo and Ferrer 2011). This mapping includes determining the subsurface extent of soil and rock, the presence of shallow groundwater, and the presence of potential problems such as subsidence, liquefaction, or movement on faults or existing landslide slip planes. Engineering geologic information also plays a role in identifying suitable aggregate and other Earth materials necessary to satisfy construction requirements. A function of engineering geology that is especially important to land use decision-making is the identification of where special conditions identified in building codes or planning documents are present (De Graff 2012). This identification ensures that design specifications in these codes and documents to protect public safety and ensure full function of constructed works are implemented.

Engineering geology should play an ongoing role in land use decisions spurred by expansion of any modern population center. The primary purposes would be to safeguard public safety and ensure that works are constructed in an effective manner and function as designed. A simple example would be a section of roadway. Engineering geologic information can identify where the preliminary alignment might cross soil or rock that necessitates importation of additional construction materials or specialized equipment for excavation or encounter shallow groundwater that require drainage control during construction. Such information allows realignment or changes to the proposed project design to facilitate road construction and avoid costly delays. In the expanding urban area of the Los Angeles basin in the USA, the value of increasing requirements for engineering geologic

Land Use, Fig. 1 A view of historic buildings in Florence, Italy, along the Arno River including the Ponte Vecchio. View is downriver as seen from Piazzale Michelangelo (Photo: J.V. De Graff)

information in the land use planning process is demonstrated by decreased losses from storm-induced landslides (Fleming et al. 1979). Utilizing this type of information to improve design for new or existing structures is not limited to only expanding population centers in industrialized countries. Similar uses of engineering geologic information have been effectively implemented for communities in more rural areas (Anderson and Holcombe 2013).

Present-day urban areas often include structures and facilities which remain in use from much earlier times. The ability to preserve, upgrade, or replace these older works also should include engineering geologic information. Subsurface soil or rock conditions which were adequate for supporting an older two-story building may not be fully adequate for simple replacement by a modern multistory structure. In some instances, the historic buildings are located where modern land planning would not allow new buildings to be erected or would require extensive works to mitigate potential hazards (Fig. 1). This was made painfully evident from the 1966 flood damage inflicted along the Arno River to historic structures in Florence, Italy.

We expect that an expanding population center will require more or bigger buildings for housing, work, and recreation. Similarly, streets, electrical and telecommunications lines, sewer systems, water distribution, and other transport infrastructure will grow within the urban area. As a consequence, this expansion can influence actions some distance from the urban area. This arises because a greater population can require importation of water, energy, and other resources. This necessitates dams, canals, pipelines, and major roads extending out from the urban area. Just as engineering geologic information is important to land use decision-making within the urban area, this information is equally necessary to constructing and developing the means for satisfying the growing demands of an expanding population center.

Granada, Spain – An Example of Engineering Geology in Land Use

Granada, Spain, and its metropolitan area (Chacón et al. 2012) illustrates this path to urbanization. A seventh-century BC walled settlement on a hill has grown to a modern city and surrounding metropolitan area covering 860 km^2. Albeit many cultural and historical factors influenced Granada's growth, human utilization of the natural environment including its location in a river valley with arable land and in proximity to exploitable salt and gold played a significant role.

The growth of Granada has necessitated development of transportation systems and construction of buildings (Chacón et al. 2012). Building this modern infrastructure requires using engineering geology to ensure proper foundation designs for the variety of bedrock and soil conditions present. It is also important for maintaining the structures still in use from historic building activity. Engineering geology plays a role in locating and exploiting Earth materials for construction and for reclamation where these materials are extracted. Water supply for both potable and agricultural use draws upon engineering geologic expertise for siting dams, constructing canals, and building water treatment facilities. For the sizeable population present, it is important to understand the existing engineering geologic conditions in order to properly remediate past waste disposal areas and to develop current solid waste disposal facilities.

Land use planning in Granada began in 1951. Both recent planning updates and implementation of various national codes and standards stimulated the need for engineering geologic information (Chacón et al. 2012). These regulations serve as the basis for required site investigations necessary for developing engineering geologic information. Especially valuable engineering geologic information is an understanding of the flood, seismic, subsidence, and landslide hazards present in the landscape of the metropolitan area. Understanding how these geological hazards affect different parts of this urbanized area is crucial to mitigating future hazardous events and to ensuring continuing function of the infrastructure upon which the inhabitants depend on each day (Chacón et al. 2012; Marker 2013).

Cross-References

- ▶ Bedrock
- ▶ Engineering Geological Maps
- ▶ Engineering Geology
- ▶ Geohazards
- ▶ Landslide
- ▶ Site Investigation
- ▶ Soil Properties

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Landfill

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Definition

The disposal of society's wastes by tipping them into existing or specially dug voids in the ground (or, sometimes, by tipping onto the land surface, known as land-raising). Landfill does not include the management of high level radioactive wastes which requires special measures. Rather, it applies to construction, commercial, industrial, and household solid and some liquid wastes.

Synonyms

Dump site; Garbage dump; Waste disposal site; Waste tip

Introduction

Disposal of wastes by tipping has been usual for thousands of years (early sites are now archaeologically valuable). But landfill has caused increasing problems as large populations in growing urban areas have produced ever greater volumes of wastes. Larger more numerous landfills prevent other productive uses of land and can cause potential short and long-term hazards.

This has led to an approach to management of wastes known as the waste hierarchy to reduce amounts of landfilled wastes. It consists of reuse, recycling, recovery of value (e.g., energy from waste), and landfill, in decreasing order of desirability. By placing a strong emphasis on the first three, the amount of residual material that goes to landfill is dramatically decreased. However, landfill is always the cheaper option. In some less developed countries, many people make a living by picking over poorly designed and hazardous waste tips and landfills for saleable materials in unhealthy conditions.

Until the 1970s, landfills were generally poorly designed and managed causing long-term environmental problems. Improved regulations for locating sites and design, operation, monitoring, treatment, and after-care have been introduced in many countries. It was once common for all types of wastes to be accepted at a site. Now, there is separation between sites that are allowed to accept mixed wastes and those which can accept hazardous wastes (e.g., contaminated soils, residues from recovery operations, asbestos and low level radioactive materials). Hazardous waste sites require very careful design, management, and control and, once completed, can still present a potential long-term problem but the basic principles are similar to those for less hazardous landfills.

The main approach is containment of the deposited wastes (beneath, laterally, and above) to isolate potential pollutants, even though that slows the rate of decomposition. Good site location, design, operation, closure, and aftercare are required to minimize potential hazards (e.g., gases, leachates); nuisances (such as odors and vermin); visual intrusion in the landscape; and subsidence due to settlement of wastes as decomposition takes place.

Compositions of Wastes

Landfill sites may receive domestic, construction, commercial, and industrial wastes ranging from chemically inert to chemically active. Proportions of wastes from various sources change as time passes. For instance, in some countries in the early twentieth century, much waste was ash from domestic coal fires with less organic waste. But, now, domestic and retail (municipal wastes) dominate. Municipal wastes consist mainly of organic decomposable materials (e.g., food, paper, cardboard, or wood) together with potentially combustible plastics and metals depending on how effective recycling and recovery of value have been.

Construction wastes such as soil, concrete, and brick rubble are mainly inert although some may be contaminated by hazardous materials such as asbestos. Because of the variability of materials received at most landfills, fills are usually heterogeneous with composition and permeability varying vertically and horizontally over short distances. Also, liquids are often discharged into specially constructed ponds within sites. Compositions often vary between sites in a single country (Staley and Barlaz 2009). One can expect future changes in proportions and need to adapt to these changes.

Over the past few decades there has been increasing separation of "general" landfill sites from fewer sites located, designed, and managed to receive hazardous wastes. This reduces the hazard level of most landfills but concentrates the most hazardous components in fewer sites which require very careful management and monitoring now and in the longer term.

For instance, some sites may receive stable non-reactive hazardous wastes such as monolithic solidified wastes (wastes in large blocky forms that have been mixed with cement or PFA) or granular solid wastes produced by a variety of treatment plants (such as filter cakes and treated fly ash). A general classification of these waste types and the processes through which they have been treated is not possible. Each must be individually assessed and a risk assessment conducted (c.f. Scottish Environment Protection Agency 2004).

But hazardous wastes have potentially dangerous properties to human health or the environment. These can be liquids, solids, or gases from manufacturing processes or discarded used materials or unused commercial products, such as cleaning fluids (solvents) or pesticides. In the US regulatory terms, a hazardous waste is any that is included in one of four Resource Conservation and Recovery Act or Atomic Energy Act hazardous wastes lists or which exhibit a characteristic of hazardous waste – ignitability, corrosivity, reactivity, or toxicity. Hazardous wastes require careful handling, deposition, containment, and treatment in special landfill (or alternative) facilities http://www.dtsc.ca.gov/hazardouswaste/upload/hwmp_defininghw111.pdf

Regulation

Three main aspects of regulation of landfills are:

- Planning – to select the most suitable sites
- Environmental permitting (licensing) – to ensure minimum adverse environmental impacts during operation and after closure. A general principle is to use the best available techniques not entailing excessive cost and
- Health and safety – to protect workers at, visitors to, and trespassers on, sites

Regulations vary from country to country but National and International Standards exist to ensure a consistent approach to landfill design. Quality assurance and quality control are important elements of any landfill design to ensure that the design and construction of the facility are satisfactory (e.g., Environment Protection Agency 2000).

It is essential that all regulations are: carefully observed by the operator and properly enforced by the responsible authority; that prompt remedial action is taken if necessary; and that the operator is financially liable for any infringements. Close liaison between planning and environmental permitting authorities and operators is important.

The Operational Site

Site Selection

Searches of areas for potentially suitable sites are undertaken when developing planning policies for waste management. Once options are identified, closer investigation is required (walk-over survey, sampling, and testing) to narrow the choices. When a site is selected for more detailed investigation, the engineering geologist makes a major contribution to environmental assessment and site investigations. But identification of potentially suitable sites for landfills is an essential part of spatial and land use planning that should also take account of other options for use of land.

The engineering geology input is, initially, through strategic mapping to identify promising areas (Proske et al. 2005). Some principal aspects are:

- Stratigraphy – identifying potential sites with impermeable or poorly permeable strata beneath and lateral to any intended landfill
- Structural geology – seeking areas with few faults (particularly avoiding active faults), fissures, open bedding planes, underground cavities, or structures such as decomposed dykes that allow passage of fluids or gases
- Hydrogeology – assessing the potential for any emissions to cause pollution to ground or surface-water or the extent that an encapsulated large landfill might modify groundwater flow patterns and
- Access to cover material – considering whether materials excavated from a landfill site or part of the delivered wastes (soils, inert fill) can be used as daily or final cover or whether material should be quarried nearby for that purpose

Modern site identification is normally undertaken using GIS (Şener et al. 2006).

Site Design

Good site design reduces or prevents adverse effects on the environment and human health. Methods and standards must be based on current best practices. Design is an interactive process between conceptual design proposals, the findings of environmental assessment and environmental monitoring, and risk assessment.

The design concept depends on ground conditions, geology, hydrogeology, potential environmental impacts, and the surroundings of the landfill. Factors that are taken into account include:

- The nature and quantities of wastes – requirements at a landfill accepting inert waste are different to those that accept either nonhazardous biodegradable waste or facilities accepting hazardous waste
- Reduction of ingress of precipitation, surface or ground water to reduce the amount of water entering the fill
- Protection of surface and ground water and soils by means of a liner system consisting of natural or artificial mineral layers combined with synthetic materials that together meet prescribed permeability and thickness requirements
- An efficient leachate collection system to minimize accumulation at the base of the landfill and a pipe network to convey leachate to a storage or treatment facility
- A gas collection facility to prevent accumulation and migration of landfill gas linked to an appropriate venting, flaring, or power recovery system
- Measures to reduce nuisances such as bunds to reduce noise and visual intrusion or fences to capture wind-blown debris (although most nuisances are abated through good day-to-day management practices)
- The sub-grade and basal liner must be sufficiently stable to prevent excessive settlement or slippages, taking account of the hydraulic uplift pressure of groundwater on the lining system. The method of emplacement must ensure stability of the waste mass against sliding and rotational failure. Also, the capping system should be stable against sliding
- Consideration of the visual appearance of the landform and site infrastructure during and after operation in relation to surrounding landforms taking account of any change in topography of the landfill as decomposing materials settle
- Monitoring requirements during and after operation
- Estimated total costs of construction, operation, monitoring, closure, and aftercare (including sufficient money to cover any long-term liabilities) and
- The intended after-use of the site (Environment Protection Agency 2000)

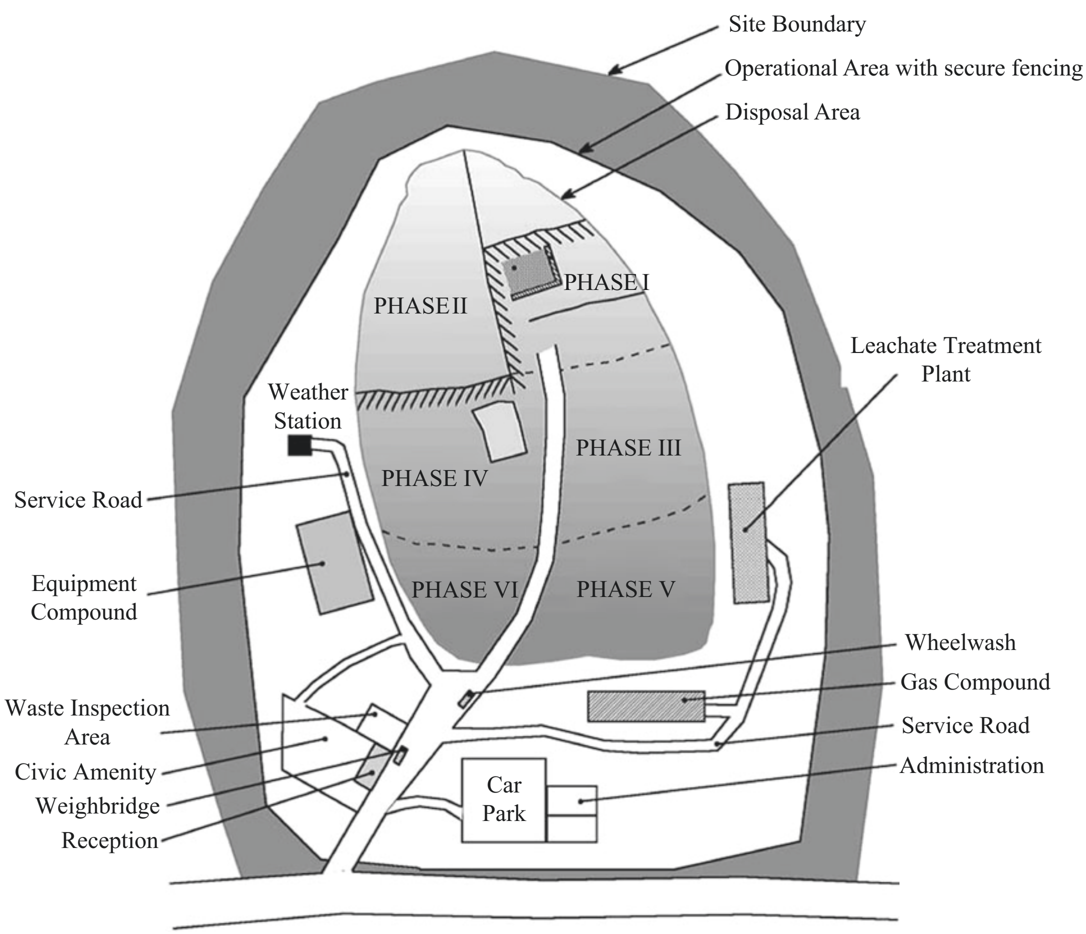
The expertise of the engineering geologist is required for only some of these stages, but it is important that the whole process is understood so that collaboration and advice can be provided to multidisciplinary teams at the right times.

Site Construction

Some landfills are constructed in existing voids such as former quarries. Others are emplaced in natural depressions in valleys or even on valley sides (although the latter can present slope stability challenges). Few are purposefully made excavations because of the expense of digging large cavities. If terrain is fairly even and there are no suitable voids available then land raising (building up the waste into a dome in the landscape) can be undertaken by constructing isolating barriers as the deposits rise (Environment Agency n.d.-a; Douglas et al. 2010).

The first step is to ensure that the base and sides are isolated from the geological and hydrogeological environment if the base and sides of the void are in hydrogeological continuity with the geological environment (Fig. 1). Measures generally consist of a layer of compacted clay in combination with a high density polyethylene membrane (Fig. 2). Lateral protection may also be given by very low permeability cut-off-walls often incorporating montmorillonite slurry (Environment Agency n.d.b). However, liners have a finite life but, currently, the long-term effectiveness of these measures is uncertain (Environment Agency n.d.-b).

Landfill, Fig. 1 Operational landfill with polyethylene liner (black) at left and on slope at the back of the landfill (Photo by the author)



Landfill, Fig. 2 Diagrammatic layout of a landfill (By permission of the Environment Protection Agency, Wexford, Eire)

Site Operation and Management

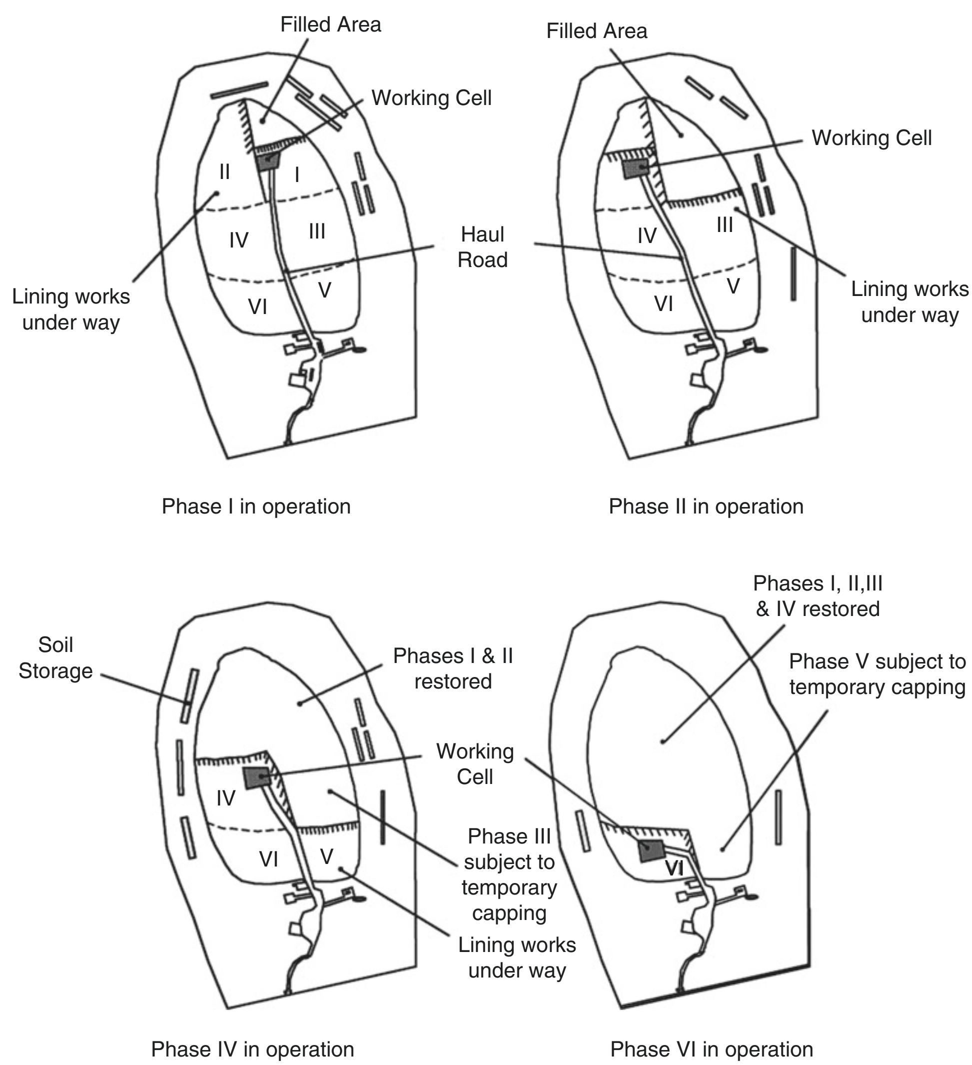
As wastes are brought to the site, they are inspected to make sure that they comply with the site acceptance criteria which define what materials can be deposited. They are then taken to the active tipping area. Tipping is confined at any one time to a specific area within the site (Fig. 2). The loads are deposited and are then spread by bulldozers and are then compacted to maximize the capacity of the landfill. At the end of the working day the tipped area is covered with soil, inert wastes or wood chips, sprayed foam products, or chemically "fixed" bio-solids. Temporary blankets can be used at night then removed next day. These measures help to deter vermin (rats, flies, birds, etc.) and to reduce wind-blown debris and odors. The space that is occupied daily by the compacted waste and the cover material is called a daily cell. The landfill is built up gradually laterally cell by cell. The waste is compressed by running a compactor across it until it acquires the intended density. This increases the amount of waste that can be deposited in the site (Environment Protection Agency 2000).

The cellular approach allows progressive restoration of the site during operation thus keeping the active area, at any one time, to a minimum (Fig. 3).

Closure and Aftercare

Landfills, on completion, are capped with impermeable materials before soil is spread and, commonly, are then vegetated. The choice of plants is important because those with deep tap roots can disturb the cap. Shallow rooting plants are more appropriate. The site is a dome in the landscape, out of scale with the local surroundings, on completion but becomes less visually obtrusive as time passes if proper allowance is made for the compaction of material during decomposition. But uneven compaction can cause problems through ponding of water or disruption of the capping layer. Also, sites are, if necessary, provided with either a recovery system for, or a

Landfill, Fig. 3 Phasing of landfilling (By permission of the Environment Protection Agency, Wexford, Eire)



means of flaring off, emissions of methane as well as measures for draining and treating leachates. Regulators require operators to have long-term responsibility for monitoring the site and undertaking remedial works if necessary; for instance, draining off and treating any leachate leaking from the site or installing cut-off walls (Heyer et al. 2003).

After-Uses

Because of the ongoing production of gases and leachates landfill sites are not generally suitable for built development because of ongoing problems. Subsequent uses generally include recreational facilities such golf courses and football pitches, parks and meadows, fields for grazing livestock, or nature conservation areas (Misgav et al. 2001).

Potential Hazards and Problems

The main hazards are leachates and landfill gases, but there are also other problems and nuisances.

Leachates

Precipitation falls on, and seeps down into, landfills during operation. Liquids produced during the bacterial decomposition of organic wastes combine with this to form solutions, often weakly acidic, which dissolve soluble compounds forming leachates. These may react with other materials in the landfill as they percolate downward to accumulate at the base of the fill or become "perched" above poorly permeable layers higher in the fill. Physical (filtration, sorption, advection, and dispersion), chemical (oxidation-reduction, precipitation-dissolution, adsorption-desorption, hydrolysis, and ion exchange), and biological (microbial degradation) processes in the landfill modify the leachate. The extent of these reactions depends on the materials underlying the landfill, the behavior of the groundwater system, and the chemistry of the leachate (Lee and Jones 1991).

Leachate composition reflects the quantities and types of wastes and precipitation at the site during emplacement and after closure. Whereas leachates contain non-hazardous components similar to those in natural groundwater (e.g., dissolved metals and salts) the concentrations are generally several times higher. In addition, leachates contain deleterious and hazardous substances such as volatile organic compounds (VOCs) and heavy metals. Wood-waste leachates typically are high in iron, manganese, tannins, and lignins. Leachate from municipal waste landfills typically has high levels of total dissolved solids and chemical oxygen demand and is slightly or moderately acidic. Leachate from ash landfills is likely to have a high pH and to contain more salts and metals than other leachates. Fluids such as oils also percolate downward. Chemical reactions often reduce potential impacts to groundwater but some, including microbial degradation, can increase toxicity by producing more hazardous by-products (e.g., vinyl chloride from the degradation of trichloroethene).

If there is hydrogeological continuity between a landfill and the surrounding environment, leachate will migrate through the unsaturated zone until it reaches the saturated zone and then follows the hydraulic gradient of the groundwater system giving rise to a pollution plume that can travel for many kilometers or may resurge from the ground to affect surface water. Pollution damages water quality and gives rise to hazards to health and to aquatic ecosystems. Therefore, it is important to prevent hydrogeological continuity by confining the leachate to the landfill and dealing with it on-site. Leachate draining to the base is pumped to a treatment plant. Conventional treatments involve using:

- Activated sludge (formed by injecting air into a medium containing suitable bacteria and protozoa) to oxidize carbonaceous organic and nitrogenous matter and to remove nitrogen and phosphorus, but this is rendered less effective by poly-aromatic hydrocarbons, adsorbant organic halogens, polychlorinated biphenyls, humic substances, and surfactants. Ultra-filtration is often used to separate sludge arising from biological processes
- An aerated packed tower to remove ammoniacal nitrogen (air stripping) or
- Chemical reagents to promote coagulation, flocculation, and settling of solids

These require expensive investments in equipment, energy, and additives, but the resulting fluid is not usually of good enough quality to be released into the environment. It is commonly taken by tanker or pipeline to a sewage processing facility for further treatment.

Alternative methods are:

- Use of a semipermeable membrane to remove ions, molecules, and particles from water (reverse osmosis) but conductivity, organic content, and encrusting caused by some elements and compounds ($CaSO_4$, Si, and Ba) result in low recovery and blockage of the membrane. The technique is also limited if the salt content of leachate is high, leading to high pressures across the membrane, and therefore requiring large amounts of electricity to drive the process. The concentrate from reverse osmosis must have further treatment before careful disposal or
- Removal of persistent biological compounds by adsorption on activated carbon filters, followed by incineration, and oxidation of contaminants or conversion into biodegradable forms using ozone

However, the products of treatment processes often have to be disposed of elsewhere transferring pollution rather than solving the environmental problem (Environment Agency 2007; Wizniowski et al. 2006).

Landfill Gas

Air is incorporated in the landfill during tipping. After burial, other gases are produced by:

- Microbial action
- Chemical reactions between materials in the wastes and
- Evaporation of volatile organic compounds (VOCs) from solvents, oils paints, etc.

Due to the physical, chemical, and biological variability of the landfill gas concentrations can vary over short distances.

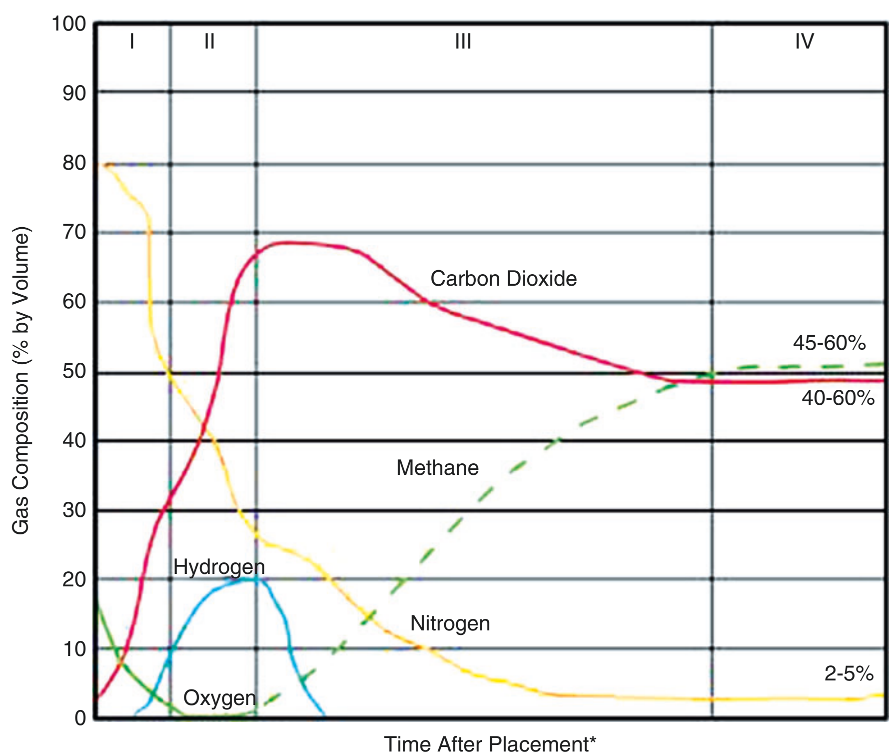
In the early stages of decomposition, oxygen is depleted by aerobic bacteria after which the main processes in landfills containing putrescible materials involve decomposition by anaerobic bacteria to produce biogas (methane and carbon dioxide together with traces of other gases). Formation of methane and CO_2 commences about 6 months after burying the waste, reaches a maximum after about 20 years and then slowly declines. The resulting mixture is about 40–60% methane with carbon dioxide making up most of the rest. In addition, small amounts of other gases are produced that have unpleasant odors (e.g., hydrogen sulfide and mercaptans) (Fig. 4). One ton of degradable waste produces about 400–500 cubic meters of landfill gas.

Differential pressure governs the behavior of the gases. Because methane is less dense than air it tends to migrate toward the top of the landfill, whereas carbon dioxide, denser than air, tends to sink within it. Low atmospheric pressure can cause more gas to be emitted from the top of an uncapped landfill. Similarly, a pressure gradient between the landfill and the surrounding geo-environment will lead to migration if the gas is not isolated in the landfill (Nastev et al. 2001).

Methane is flammable and potentially explosive but, for a fire or explosion to occur, methane and oxygen coexist in certain proportions and must be subjected to a spark or flame. The minimum concentration of methane necessary to support its combustion in air is defined as the lower explosive limit (LEL) and below that concentration is too diffuse to burn. The maximum concentration that will burn in air is defined as the upper explosive limit (UEL) at which point the mixture is too concentrated to burn. The flammable range between the LEL and UEL for methane is about 5–15% at "normal" surface temperatures and pressures (but varies in other conditions). Methane is controlled by flaring into the atmosphere or is used as an energy resource. Carbon dioxide is asphyxiating and toxic. Because it is heavier than air it can migrate downward and accumulate in cavities to present a hazard off-site if the landfill is not properly encapsulated (Environment Agency 2004).

Combustion

Recent landfills contain materials such as plastic and textiles and some VOCs which are potentially combustible. Landfill fires can be minor surface outbreaks or major events deep in the landfill that can burn for many years. Surface fires can be



Landfill, Fig. 4 Gas evolution during the life of a landfill (Source: Wikipedia: Creative Commons Attribution – ShareAlike License) caused by deposition of hot wastes, lightning strike, or arson. Deep-seated fires can begin due to spontaneous combustion if either exothermic biological decomposition or chemical oxidation increases the temperature. If heat cannot disperse faster than it is produced, the temperature rises until combustion begins. This tends to occur early in the life of the landfill when oxygen is still present or at the time of maximum settlement of the landfill disturbing the cap allowing oxygen to enter. Fires can also be started by short circuiting of batteries, organic materials soaked in highly combustible fluids, or burial of hot objects. Fires can perforate the cap of the landfill or damage liners allowing the escape of gases, including dioxins, and leachates. Uncontrolled atmospheric emissions are potentially hazardous to health and the environment. Techniques to extinguish fires include excavation and dousing with water, injection of nitrogen or carbon dioxide, injection of foams, and ground freezing. But treatment is difficult and expensive therefore prevention is preferable (Copping et al. 2007).

Settlement

Landfilled materials are compacted during site operation to extend the capacity of the fill but, in addition, biodegradation of organic matter in municipal solid waste landfills increases the void ratio and weakens the structural strength of the refuse within a landfill leading to a substantial loss of volume and settlement after site closure. Initial settlement is due to the loading of the upper parts of the landfill onto the lower levels followed by consolidation of the fill, whereas secondary settlement is due to mechanical creep and loss of volume during decomposition. The rate and amount of settlement depend on refuse composition and operational practices. It is variable but can sometimes be as much as 40% leading to subsidence of the landfill surface of several meters. The processes are complicated and lengthy, so accurate prediction of settlement is difficult. Estimating long-term landfill settlement is necessary when planning for closure and reuse of a site. The fill is domed to allow for expected subsidence so that the final profile is more compatible with the surrounding landscape. Also, due to unevenness in the lateral and vertical composition of the fill, differential settlement may damage the landfill cap allowing gases to escape (EI Fadel et al. 1999; Park et al. 2007).

Monitoring

Monitoring is required to ensure that the site has been adequately designed and operated to prevent adverse effects and to initiate early remedial action if problems emerge. Monitoring is undertaken on both the operational site and the adjacent surroundings. Results are compared with baseline information during the initial environmental assessment to establish whether changes are taking place. Monitoring continues, usually for decades, after site closure because of continuing development of gases and leachates.

Surface Water

Surface water is monitored periodically to check the quantity and quality of surface waters at representative points, both upstream and downstream, around the landfill (streams, rivers, canals, ditches, lakes, reservoirs and lagoons, wetlands, estuaries, and coastal waters as appropriate). Discharges may be intentional (e.g., discharge of treated leachate) or unintentional (e.g., leachate escape, contaminated surface water run-off, or accidental spillages). Monitoring is commonly undertaken quarterly to assess seasonal changes.

Surface water flow can be rapid, spreading pollution in minutes or hours, but can be large in volume dilution pollutants. However, seasonal changes can cause large variations in dilution so risk assessment is normally based on the lowest flows in surface watercourses. Samples of water and, sometimes, bottom sediments are taken for physical and chemical analysis. Analysis includes pH, temperature, dissolved oxygen, and electrical conductivity. Biological assessments are normally based on macro-invertebrate community analysis because increased pollution causes a decrease in diversity and an increase in tolerant organisms. Where necessary the fisheries status is monitored. Observations are also made of litter, sewage fungus, surface scum, oil, weeds, algae, and odor as well as river or tidal behavior (Environment Protection Agency 2003).

Groundwater

Monitoring locations may be existing springs, wells, and water supply boreholes or new boreholes. At least three monitoring boreholes are required at appropriate locations and depths to monitor groundwater quality and quantity up and down-gradient from a site to determine the direction of flow and effectiveness of the environmental control systems. Water and piezometric levels are determined and representative samples are taken for analysis. The number of boreholes depends on the area of the landfill, heterogeneity and permeability of aquifer (s), flow velocities, groundwater abstraction, expected compositions of leachate, the containment system, and proximity to any nearby contaminated sites (Nielsen 1991).

Results of analyses are compared with defined threshold values which, if exceeded, require remedial actions in conformity with a predefined contingency plan. In sites receiving nonhazardous biodegradable wastes these are set for, at least, ammonia, total organic carbon, and chloride. Levels may also be set for pH, VOCs and semi-VOCs, phenols, heavy metals, and fluoride (Environment Protection Agency 2003).

Leachates

Leachate monitoring checks the process of decomposition of waste, that management systems are operating properly and whether approaches to monitoring of surface and

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groundwater should be modified. Measurement of the volume and composition of leachate is undertaken cell by cell and at each place that leachate is discharged from the site. The monitoring points are placed in relation to flow paths so as to provide representative samples. Treatment plants and storage lagoons are also monitored. The frequency of monitoring reflects the quantity and types of waste deposited, operational practices, the sizes of operational cells, and the effectiveness of leachate drainage and collection systems. Representative samples from each monitoring location are analyzed. Toxicity limits may be set in waste licenses if treated leachate is discharged to surface water. Techniques for the sampling of leachate wells are similar to that used for groundwater boreholes but safety precautions are stringent; samples have to be isolated quickly to prevent changes to composition. Small diameter wells are purged of stagnant water before samples are taken. In highly compacted or dry waste, recovery to an adequate sample may not be possible. Leachates are also sampled from chambers, sumps, or combined collection systems by grab sampling. Samples are sent for chemical, metal, and microbiological analysis (Environment Protection Agency 2003).

Landfill Gas

Landfill gas escapes through any defects in the cap or the gas collection system. If the cap is intact, less gas escapes through any permeable material that covers deposited waste. Some parts of the capping layer (such as side slopes) are often more permeable than the main part of the cap. The total emissions through the cap reflect: poor quality capping, minor fissures/discontinuities in the cap, leaking gas wells, faulty pipes, or poorly sealed leachate chambers. Surface monitoring can be undertaken to give preliminary results on migration of landfill gas and VOCs and the integrity of the capping layer. This can be done by walking over the surface of the landfill carrying a flame ionizing detector or pumping samples into bags. Samples can then be analyzed using the detector or sent to a laboratory.

A flux box is an effective technique for measuring normal surface emission through a landfill cap (Bogner et al. 1997). It is often recommended in preference to other direct methods, such as depth/concentration profiling, because they involve assumptions and parameters that can rarely be measured adequately in the field. The box is an enclosed chamber placed on the ground in which changes of methane concentration above the small enclosed area are measured at intervals to quantify the rate of emission beneath the box. A flux tent is a similar device that covers a larger sample area, but this requires careful use to ensure the large enclosed volume is mixed adequately and encloses a consistent volume during sampling. By taking measurements at a number of representative places using a flux box, it is possible to calculate an emission rate for individual zones and hence emissions for the entire capped area of the landfill site. The methodology is simple, quantitative, and repeatable at a particular location, and many individual locations can be measured in one day.

The surface flux is calculated directly from the rate of change in methane concentration within the flux chamber. The method gives the actual emission rate from the particular surface over which the flux box is placed. It does not require information on variables such as soil physical parameters and prevailing meteorological conditions. The flux measurements are easy to calculate, but judgment is needed to ensure that the sampling points are sufficiently representative (Reinhart et al. 1992; Environment Agency Wales 2010).

Monitoring devices can be hand held single reading at the chosen point, or wired or wireless continuously reading and mounted within the borehole or elsewhere. Near-surface monitoring is affected over short time periods by weather. As atmospheric pressure rises, the rate of gas escape from the landfill is reduced and may cease with oxygen incursion into the upper layers. Differential diffusion and gas solubility (varying strongly with temperature and pH) complicates this behavior. Structures such as monitoring boreholes can also caused localized variability at greater depths giving an impression that bioactivity and gas composition are changing more rapidly than is actually the case. Isolated point measurements are unreliable due to this variability (Environment Protection Agency 2003).

Subsurface monitoring requires gas probes, strategically placed at intervals of not more than a few hundred meters in a circle around the landfill. An alternative is to monitor the composition, temperature, pressure, and rate of flow of the gas collected by an extraction system either at each well or at a flare or power plant.

Landfill gas often contains significant amounts of corrosive gases such as H_2S and SO_2 which shorten the life of most monitoring equipment. Physical settlement as waste decomposes makes borehole monitoring systems vulnerable to breakage as the weight of the material shifts and fractures equipment.

Determination of Sources of VOCs

Techniques developed for evaluating whether landfill gas or leachate is the source of VOCs in groundwater samples are that:

- Leachate frequently has elevated levels of tritium compared to groundwater and whereas leachate would increase tritium levels landfill gas does not.
- Landfill gas components can react with minerals and enrich the leachate in chloride so high chloride levels can be indicative of leachate contamination.

- Highly soluble VOC's are indicative of leachate inputs because they are too soluble in water to migrate in landfill gas.
- Highly soluble semivolatile VOCs, such as phenols, are also linked to leachates.
- High levels of dissolved CO_2 are indicative of landfill gas effects because not all of the CO_2 in landfill gas reacts immediately with aquifer minerals but such reactions are complete in leachate because of soils used as daily cover.

To assess whether VOCs are partitioning into groundwater in a monitoring well or headspace gas the dissolved VOC concentrations can be compared. If the Henry's Law constant multiplied by the water concentration is significantly less than the measured gas concentration, the data are consistent with VOCs partitioning from landfill gas into the groundwater (Sabel and Clark 1984).

Combustion

Combustion in landfills can lead to emissions of smoke through a damaged cap but deeper seated fires may, at least initially, be cryptic. However, a subsurface oxidation event or landfill combustion are indicated by the presence in the landfill gas of compounds that are more stable at high temperatures (above 500°C) including propene (formed from propane at temperatures above several hundred degrees C) and dihydrogen (H_2) which indicates with thermal inactivation of CO_2-reducing microbes which normally produce methane (Copping et al. 2007).

Settlement

Changes in the site profile during settlement need to be monitored by normal surveying methods, such as laser ranging, to determine whether compaction is proceeding as predicted of, if it is not, whether remedial action is needed (Olivier et al. 2005).

Recovery of Resources

If enough methane is generated during decomposition, it can be either removed by flaring off or can be collected for power generation either for local use or feeding into a wider supply grid (Fig. 5). The efficiency of gas collection at landfills directly affects the amount of energy that can be recovered – closed landfills (those no longer accepting waste) collect gas more efficiently than open landfills (those that are still accepting waste). Comparison of collection efficiencies at closed and open landfills has indicated about a 17% difference between the two. Landfill gas can also be used to evaporate leachates. Provisions for use should be designed into municipal landfills at the design stage (Themelis and Ulloa 2007).

Before large scale recycling became common, many valuable materials were tipped into landfills. Landfill mining is sometimes undertaken to recover, by excavation and sieving, recyclable fractions, but residual combustible materials can also be used to generate power. Caution is needed for safety, to prevent pollution and minimize local nuisances such as odors, wind-blown debris, and vermin (Krook et al. 2012).

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Landfill, Fig. 5 Pipe-work laid between methane abstraction wells after capping of a landfill and before topsoil spreading to convey methane to a plant for energy production (Photo by the author)

Summary

Landfill has been the main means of dealing with wastes for many years but, recently, in many countries, increased recycling is reducing the amounts of materials deposited. Even so, this remains a widespread means of waste management and is the only practical way of dealing with residues that cannot be used productively, including hazardous wastes. Modern landfills are encapsulated below, laterally, and above to isolate products of decomposition (gases and leachates) that are evolved over 30 years or more. The long-term performance of landfill liners is not yet known, so it is possible that even the best designed containment systems may eventually deteriorate allowing future discharges of leachates or gases.

Regulators require operators to be responsible for monitoring and, if necessary, remedial actions for as long as emissions are produced. An alternative approach could be to encourage ingress of water into the landfill to accelerate chemical and biological reactions and reduce the period. That would produce more leachate but with relatively lower concentrations of toxins. Decisions must be taken within the context of risk management.

The main inputs of the engineering geologist to landfill operations are:

- Identification of areas which may contain suitable sites for consideration in planning of land use
- Contributing to environmental impact assessments of proposed sites
- Site investigations to establish the suitability of sites in terms of geology, geological structure, engineering properties of soils and bedrock, and hydrogeological behavior and
- Monitoring of gas and leachate emissions and of the compaction behavior of the site during operations and after closure

But it is important for the engineering geologist to also understand the nature of the whole operation and of the regulatory systems to make sure that appropriate and timely advice is provided.

Cross-References

▶ Boreholes
▶ Engineering Geological Maps
▶ Environmental Assessment
▶ Gases
▶ Groundwater
▶ Liners
▶ Stabilization
▶ Surveying
▶ Waste Management

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Landforms

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Definition

Landforms are the different features of the land surface with distinctive morphological characteristics that comprise landscapes.

Study of Landforms

The scientific study of landforms and the earth surface processes that create them is the focus of geomorphology (Shroder 2013). Landforms are formed by geomorphological processes that involve the movement of mass (rock, sediment, water). The changes in the shape of the Earth's surface may be related to endogenous processes like tectonic activity and volcanism, exogenous processes including weathering, erosion, transportation, deposition, and locally biogenic activity (Goudie 2004; Migoń 2010; Tooth and Viles 2016). Endogenous processes, largely controlled by plate tectonics, create topography through differential vertical movements (uplift, subsidence) and the construction of volcanic edifices, whereas exogenous processes, mainly driven by the solar energy and governed by gravity, tend to erode upland regions and accumulate sediments in lowlands and subsiding areas. Landforms can form by the accumulation of sediments or volcanic products (depositional/constructional landforms), may be carved on pre-existing material (erosional/degradational landforms), or result from the deformation of the land surface (deformational landforms). The origin of some landforms, either active or inactive, can be explained by contemporaneous processes acting in the region, whereas relict or inherited landforms formed in the past under environmental conditions or tectonic regimes different than the present-day ones. The genetic interpretation of some landforms may be complicated by morphological convergence or equifinality, whereby different processes result in the development of very similar features; e.g., fault scarp related to surface ruptures versus fault-line scarp related to differential erosion controlled by an inactive fault.

Landforms are commonly classified into different genetic groups according to the geological factors that control their formation, the climatic environment in which they typically develop, and the type of geomorphic system or process to which they are associated (Gutiérrez and Gutiérrez 2016).

The field of **structural geomorphology** covers those landforms controlled by geological factors (tectonism, geological structure, lithology) (Fig. 1). *Tectonic landforms* are created by surface deformation related to active tectonic structures, mainly related to regional stress fields driven by the relative motion of the lithospheric plates (Fig. 1a) (McCalpin 2009; Burbank and Anderson 2012). Non-tectonic gravitational processes such as deep-seated slope deformations/movements, evaporite dissolution-induced subsidence, salt flow (halokinensis and diapirism), or aquifer consolidation may produce landforms that mimic those related to tectonic strain. *Structural landforms* are primarily erosional features controlled by the underlying geological structure (e.g., attitude of strata) and the distribution of rocks with different resistance to erosion (Fig. 1b). *Volcanic landforms* include those generated by constructional processes related to the emission of magma, as well as erosional and collapse processes characteristic of volcanic environments, like calderas or giant flank collapses in volcanoes (Fig. 1c). Some geomorphic features produced by hydrothermal activity can be also included within this category. *Karst landforms* develop on soluble

Landforms, Fig. 1 Examples of landforms controlled by geological factors. (**a**) Normal fault scarp and associated fissure developed on basalts in the Þingvellir, Iceland (© Francisco Gutiérrez); (**b**) Attitude of strata controlling mountain shape and geomorphological processes at Mt. Pelmo (3168 m a.s.l.), Dolomites, Italy (© Mauro Soldati); (**c**) Teide volcano with historical lava flows in Tenerife, Canary Islands, Spain (© Francisco Gutiérrez); (**d**) Sinkhole in the Dead Sea related to salt dissolution, Jordan. Sinkhole in this region are induced by the human-induced dramatic drop in the lake level, current at a rate of around 1 m/yr. (© Francisco Gutiérrez); (**e**) Spitzkoppe in the Namib Desert, one of the highest granite inselbergs on Earth (© Francisco Gutiérrez)

rocks, either carbonates or evaporites, and are mainly produced by dissolution or precipitation processes (Fig. 1d) (Ford and Williams 2007). *Granite landforms* refer to some geomorphic features that typically form in granitic terrains **(**Fig. 1e**)** (Migoń 2006).

Landforms developed under specific climatic environments fall within the scope of **climatic geomorphology** (Gutiérrez 2005) (Fig. 2). *Glacial landforms* are formed by the erosional and depositional action of glaciers in cold regions, essentially high latitudes (polar regions) and

Landforms, Fig. 2 Examples of climate-related landforms. (**a**) Glacial landforms and moraine-barrier lake at Cerro Torre, Southern Patagonian Ice Field, Parque Nacional Los Glaciares, Argentina (© Mauro Soldati); (**b**) Patterned ground typical of periglacial regions where freezing and thawing of soil alternate. Sorted polygons and circles are evident at Ny Ålesund, Svalbard (© Mauro Soldati); (**c**) Great Sand Dunes National Park, Rocky Mountains, Colorado. This erg is located in the northern sector of the Rio Grande Rift, at the foot of the Sangre the Cristo Mountains, an uplifted fault block (© Francisco Gutiérrez); (**d**) Weathering and erosional landforms in red sandstone beds in the tropical environment of Danxiashan, Guangdong Province, China (© Mauro Soldati)

mountain environments where snowfall is greater than snowmelt (Fig. 2a) (Benn and Evans 1998). *Periglacial landforms* are associated with cold non-glacial environments characterized by intense freezing and thawing activity and frozen ground (permafrost) (Fig. 2b) (French 2007). *Desert landforms* occur in regions characterized by very limited precipitation and sparse or non-existent vegetation cover (Fig. 2c) (Parsons and Abrahams 2009). These are the areas where wind activity and aeolian landforms are more significant (Lancaster 1995). *Tropical landforms* predominate in warm and humid tropical environments characterized by intense weathering that allows the development of thick saprolite mantles and lateritic soils (Fig. 2d) (Thomas 1994).

There are also landforms associated with geomorphic systems found in most climatic environments: these are part of **non-zonal geomorphology** (Fig. 3). *Weathering landforms* are related to the breakdown and decomposition of materials at or near the surface by physical, chemical, and biological processes (Fig. 3a). *Gravity-induced landforms* mainly consist of slope movements that involve the gravitational downward and outward displacement of slope forming materials (Fig. 3b) (Cruden and Varnes 1996). *Fluvial landforms* are features generated by running water, mainly perennial rivers and ephemeral streams) (Fig. 3c) (Charlton 2009). *Coastal landforms* develop where the land and the sea interact and waves, tides, and nearshore currents play a prevalent morphogenetic role (Fig. 3d) (Bird 2008). *Anthropogenic landforms* of many different types (e.g., terraced slopes, quarries, dumps, artificial levees) are presently observable in all morphoclimatic regions and deserve much attention due to the environmental impacts and potential engineering-geological implications that they may pose (Fig. 3e) (Szabó et al. 2010).

Geomorphological studies address multiple aspects with practical implications: (1) processes and factors involved in the origin and development of landforms; (2) quantitative characterization of landforms (morphometry); (4) landform chronology, long and short-term evolution, and current state

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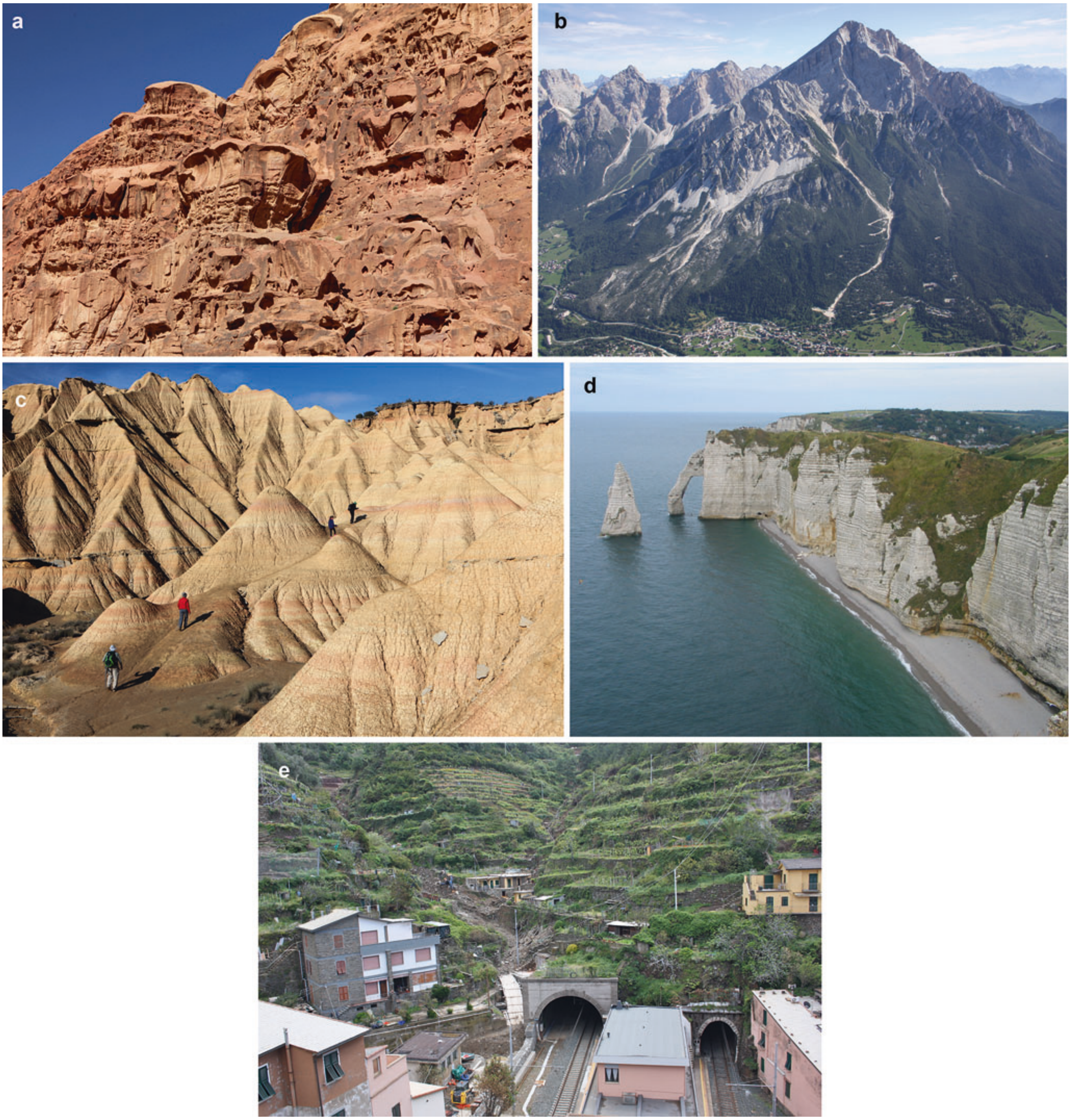

Landforms, Fig. 3 Examples of non-zonal landforms. (**a**) Carvernous weathering (tafoni) in sandstone cliffs, Wadi Rum, Jordan (© Francisco Gutiérrez); (**b**) Gravity-induced landforms at Mt. Antelao (3263 m), Dolomites, Italy. On the left, the historic landslide of 1814 which buried two villages and caused 314 victims; on the right, a debris flow track ending into a retaining basin meant to reduce risk at the underlying village of Cancia, where even recently inhabitants lost their lives in occasion of intense rainfall events (© Mauro Soldati); (**c**) Badland landscape characterized by rapid running-water erosion developed on argillaceous sediments, Bardenas Reales in the Ebro Cenozoic Basin, NE Spain (© Francisco Gutiérrez); (**d**) Coastal cliff and sandy beach modeled by intense wave and tide action at Étratat, Normandy, France; a well-developed sea notch and rock-fall deposits are observable (© Mauro Soldati); (**e**) Anthropogenic terraced slopes at Vernazza, Cinque Terre, Northern Italy, affected by soil slips and debris flows occurred in occasion of torrential rainfall in October 2011 (© Mauro Soldati) of activity; (5) monitoring and modeling of surface and near-surface processes and related landform changes; (6) analysis and prediction of the magnitude and spatial-temporal distribution of processes; and (7) expected impacts of the ongoing global environmental change and human activities on landscapes and earth surface processes, particularly geohazards.

Importance of Landforms in Engineering Geology

Geomorphological investigations provide highly useful information for the successful planning, design, and development of engineering projects, assisting in the identification and anticipation of potential problems and in the exploration of resources (Fookes and Lee 2005). Geomorphological studies in combination with geological background are essential for understanding a site within its broad spatial and temporal context. Although engineering works are commonly carried out in spatially restricted areas, the identification and analysis of potential problems require a broader view and conduct geomorphological investigations in much larger areas. *Geomorphological maps* covering large areas can be highly useful for identifying the optimum site for a project (Smith et al. 2011). For instance, an active seismogenic fault running close to a site proposed for a critical facility (e.g., nuclear power plant) may have no geomorphic expression in the site vicinity, but may be clearly detectable through geomorphic anomalies at considerable distance. Similarly, the estimation of the probability of occurrence of some hazardous processes and the construction of magnitude and frequency relationships require data gathered over large areas and covering long time spans (e.g., landslides, sinkholes). Proper understanding of the geomorphology of a project area (geomorphological model), which generally requires the production of detailed geomorphic maps, provides the basis for designing an adequate and cost-effective site investigation. Geomorphological insight helps in the determination of the most adequate investigation methods (e.g., geophysical surveys, boreholes, trial pits, trenches), the extent to which they should be applied, and their best spatial distribution.

Landforms inform us about potentially *hazardous processes*. For instance, geomorphological mapping may reveal that a valley constriction selected for dam construction may be related to an active deep-seated landslide consisting of highly permeable and mechanically weak deposits (Fig. 4). Similarly, geomorphic maps may show active sinkholes along the trace proposed for a linear infrastructure and serve as the basis for redefining its route with significant economic savings (Fig. 5a). The characterization of landforms, including detailed mapping, historical analysis, dating, monitoring, and modeling provide key information on geomorphic processes that may have detrimental impacts on existing or projected engineering structures (e.g., construction of a critical facility close to a shifting river channel). Geomorphic features may also provide enlightening clues on the presence of difficult ground conditions at a site, including soft surficial deposits (e.g., paleolake); collapsible soils (e.g., subsidence features); swelling and dispersive clays (e.g., subsurface pipes, collapse

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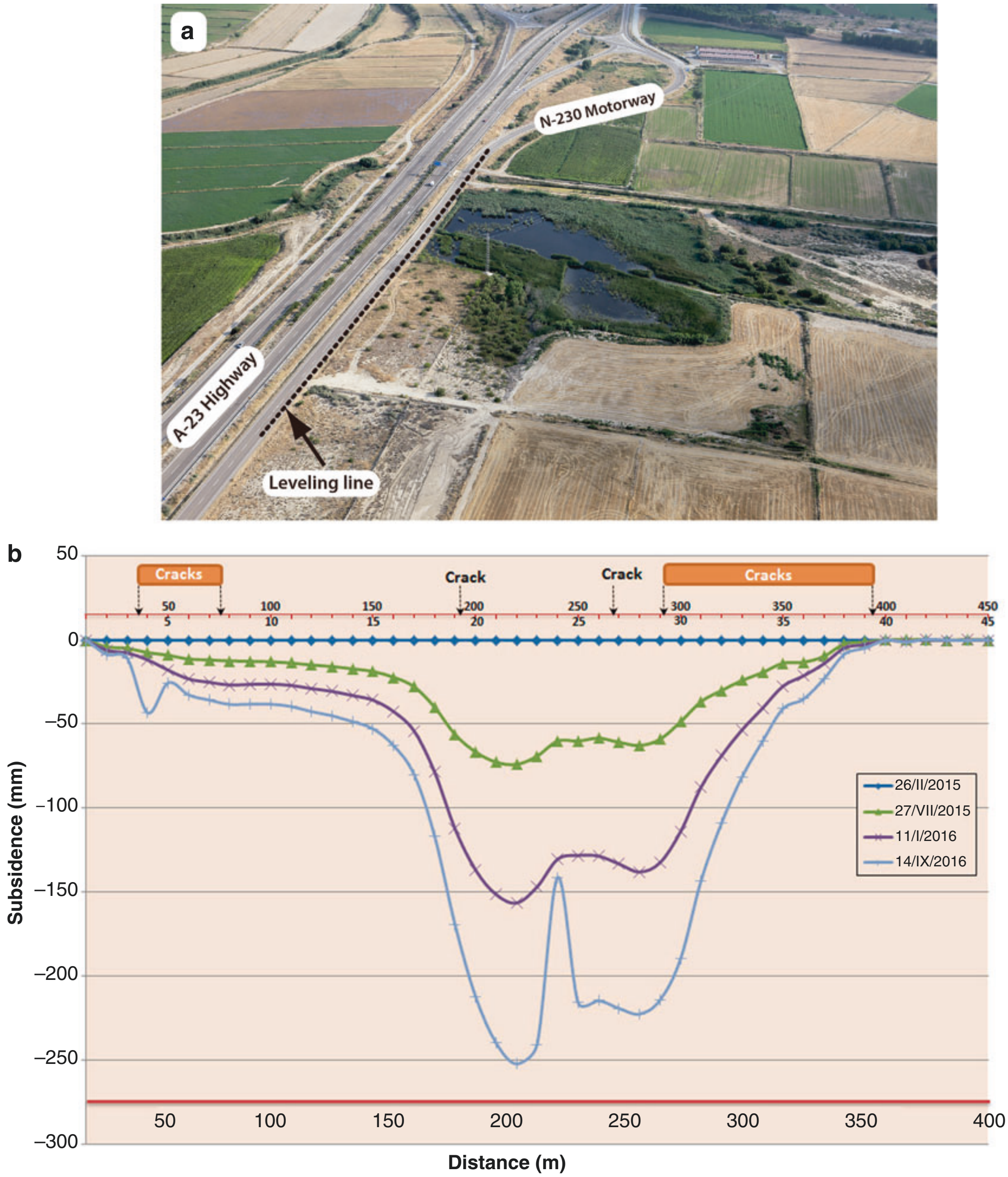
Landforms, Fig. 4 The left abutment of the Escarra Dam in the Spanish Pyrenees is founded on a fresh-looking deep-seated landslide developed on limestones underlain by slates. Note the prominent head scarp of the landslide acting as the source for large rock falls, and the irregular topography of the displaced mass, with a series of transverse ridges, troughs, and uphill-facing scarps (© Francisco Gutiérrez)



Landforms, Fig. 5 (**a**) Oblique aerial photograph of an active sinkhole ca. 600 m across in the vicinity of Zuera, NE Spain, traversed by two major roads (A-23 highway, N-230 motorway) that require continuous repair works. The production of a geomorphic map would have allowed the recognition and avoidance of this hazardous zone. (**b**) Cumulative subsidence profiles constructed with high-precision levelling data acquired along the N-230 road (see location above). Monitoring data reveal continuous subsidence at an average rate as high as 16 cm/yr. related to rapid halite dissolution (© Francisco Gutiérrez) (© Mauro Soldati)

hollows, gilgai); permafrost (e.g., depressions related to thawing of ground ice); slopes prone to landsliding (e.g., hummocky topography); weathered, weak, and fractured bedrock (e.g., differential erosion features); irregular rockhead (e.g., pinnacles and grikes); or the presence of natural or artificial cavities (e.g., sinkholes). Resources needed for the development of projects may be also identified on the basis of geomorphic knowledge (e.g., aggregates).

Engineering projects may cause significant on-site and off-site environmental impacts, both negative and positive. For instance, dams may substantially reduce flooding hazards, but may also cause adverse alterations in the water and sediment flux and induce large catastrophic landslides (e.g., the Vajont landslide in Italy in 1963, Fig. 6; Ghirotti 2012). Geomorphological investigations, typically covering larger time spans than the historical and instrumental records, contribute to

Landforms, Fig. 6 The Vajont landslide, NE Italy, occurred in October 1963 resulting in almost 2000 victims mostly due to the water overflow from the artificial reservoir located at the base of Mt. Toc (1921 m). The M-shaped landslide scarp is clearly visible (© Mauro Soldati)

predict the magnitude and spatial-temporal probability of future impacts and to design preventive and corrective measures to ameliorate their detrimental effects.

Geomorphological studies may play a decisive role in determining the nature and causes of damage associated with engineering projects and subject to litigations (forensic geomorphology). Recently, the agency in New Zealand responsible for national hazard insurance refused a multimillion dollar pay-out to a community affected by a catastrophic flow event (May, 2005, Matata), because they only cover damage related to mass movements, but not flood damage. Expert geomorphological judgment resolved that the damage was in fact caused by a debris flow, thereby reversing the decision. Geomorphological studies are also critical for resolving frequent conflicts about territorial boundaries established by geomorphic features. Over one-third of the total length of international boundaries worldwide follows rivers and streams that are inherently dynamic landforms. Commonly, legal practice makes a distinction between accretion (gradual change) and avulsion (abrupt alteration of the channel course). Accretion generally entails boundary shift and transference of land. In the case of avulsion, the boundary remains in the abandoned channel with the consequent loss of access to water and land in some portions of the territory. Some countries, in order to prevent these geomorphic changes with controversial political implications, undertake major engineering works (e.g., channelization) aimed at fixing the boundaries which eventually cause severe environmental damage (e.g., border between Mexico and USA along the Rio Grande) (Donaldson 2011). Recently, the International Association of Geomorphologists has provided expert opinion to the International Court of Justice in relation with a territorial dispute concerning the definition of the boundary between Costa Rica and Nicaragua in a stretch of the Caribbean Coast affected by rapid geomorphic changes in historical times (Fouache and Gutiérrez 2017).

Landform Origin, Development, and Degradation

The identification, mapping, characterization, dating, monitoring, and modeling of landforms provide a great deal of information on contemporaneous and past Earth surface processes of relevance for engineering projects. Those processes may be involved in the creation of landforms (e.g., collapse sinkhole), the rejuvenation or reshaping of pre-existing ones (e.g., collapse sinkhole reactivation), or their degradation and obliteration (e.g., erosion of sinkhole margin and infill of the depression). *Geomorphological processes* can operate over a wide range of spatial and temporal scales and with different frequencies and magnitudes. Some processes act continuously and produce gradual geomorphic changes. These slow but persistent processes are frequently perceived as of limited relevance because they are not attractive for the media and very rarely cause damage to people. However, they can cause significant economic losses (e.g., soil erosion, swelling soils). For instance, the American Society of Civil Engineers estimates that one fourth of all homes in the USA suffer from damage related to expansive soils and these cause billions of dollars in damage every year. Other processes like volcanic eruptions or earthquakes typically operate discontinuously. Long periods of inactivity are punctuated by short-lasting or even instantaneous high magnitude and low frequency events that involve the movement of large amounts of mass at the Earth surface and abrupt changes in the landscape. The 1964 Great Alaska Earthquake (M_w 9.2) was accompanied by

Landforms, Fig. 7 Groundwater overexploitation and the consequent water table decline may cause loss of buoyant support in cavity roofs and the development of human-induced sinkholes. Cover collapse sinkhole in An Nu'ayriyah area, Saudi Arabia, developed in a pivot irrigation field (© Francisco Gutiérrez)

meter-sized vertical displacements (subsidence and uplift) over an area of several hundred square kilometers, resulting in instantaneous geomorphic changes along the coast (e.g., emergence of the sea floor). Some geomorphic processes, characterized by a mixed gradual-catastrophic behavior, act regularly at low rates, and eventually experience high magnitude and low frequency events (e.g., river discharge and floods, sea waves and tsunamis). These major catastrophic events with long return periods are commonly responsible for the large disasters that cause high numbers of fatalities and extensive damage in engineering structures. Often, these events tend to be described as unexpected and unpredictable. However, geomorphological knowledge, covering a much larger time span than the limited human record, often reveals that such catastrophic events have precedents and are both expectable and predictable, at least in regard to their spatial distribution (Alcántara-Ayala and Goudie 2010).

Geomorphological processes and systems are controlled by multiple internal (intrinsic) and external (extrinsic) factors. The main controlling factors should be identified and their roles assessed at least qualitatively. This information is essential for preventing engineering problems and designing adequate corrective measures. Glade and Crozier (2005) propose four categories of factors applicable to landslide investigation: predisposing, preparatory, triggering, and sustaining. Predisposing factors are static, intrinsic variables that favor slope instability (e.g., bedding planes dipping out of the slope). Preparatory factors are dynamic factors that may shift a slope from a stable to a marginally stable state, without initiating the movement (e.g., weathering, slope oversteepening by glacial erosion). Triggering factors are external stimuli that effectively initiate the movement, shifting the slope from marginally stable to unstable (e.g., seismic loading). Sustaining factors dictate the behavior of actively unstable slopes (e.g., fluvial undermining at the toe of a landslide). Some potentially damaging processes, mainly exogenous, are frequently triggered or accelerated by human activities (human-induced hazards). Humans are presently one of the main geomorphic agents on our planet and are rapidly enhancing the frequency and intensity of a number of hazardous processes (e.g., floods, landslides, sinkholes, permafrost thawing). The recognition and assessment of the human contribution to these hazards is an essential step for the design and application of strategies aimed at reducing the associated damage. For instance, sinkhole hazard is increasing dramatically in some arid regions of the Middle-East underlain by soluble rocks. Here, the rise in the sinkhole frequency is commonly related to rapid declines in the water table caused by the overexploitation of aquifers with extremely low recharge rates (Fig. 7).

Some geomorphic processes and changes occur when a critical condition is reached (geomorphic threshold). Such conditions may be determined by internal and external factors (intrinsic and extrinsic thresholds). For example, the head of an alluvial fan may experience incision (fan-head trenching) due to the oversteepening of its longitudinal profile, which may be related to an external driver like tectonic deformation, or an internal adjustment like proximal aggradation. Rainfall thresholds (magnitude, intensity, antecedence) that determine the development of some landslide types such as shallow soil slides can be identified in some areas by analyzing precipitation records and landslide inventories with chronological data (Borgatti and Soldati 2013). However, such thresholds may vary through time and may rapidly lose their validity. The rainfall threshold for the occurrence of rock falls may differ drastically before and after a major earthquake. When the earthquake occurs, there may be many rock cliffs at marginal stability conditions that are instantaneously modified by the seismic event. Subsequently, a much higher threshold needs to be reached (higher earthquake intensity) for the occurrence of similar rock falls. This reveals that some geomorphic systems have memory, and that their adjustments depend on the antecedent conditions and events. All these concepts illustrate the importance of identifying thresholds and reconstructing the past history of geomorphic systems for anticipating and managing engineering problems. Geomorphic changes may be strongly influenced by the relative spatial distribution of landforms and degree of connection among them. Paired landslides are common in narrow valleys. A first landslide may cause the deflection of the river channel, which undermines the opposite slope, and eventually induces an additional landslide. Regarding connectivity, the amount of sediment that reaches a reservoir from the surrounding slopes depends to a great extent on the connectivity between the slopes and the drainage network that flow into the artificial lake.

Frequently, there is a temporal lag between a geomorphic change and the causative stimulus, called reaction time. Moreover, the geomorphic adjustment may be achieved during a prolonged period, called relaxation time (e.g., isostatic rebound related to deglaciation). The time over which the resulting geomorphic features exist is generally designated as the landform lifetime or persistence time. The response of the geomorphic systems to the impulses of change and the persistence of the resulting geomorphic features depend on the sensitivity versus resilience of landforms and landscapes (Brunsden 2001). Alluvial meandering channels are highly sensitive features that experience continuous and rapid alterations, whereas geomorphic adjustments in bedrock channels are generally extremely slow and are characterized by long relaxation and persistence times. Some geomorphological processes with significant applied implications induce changes, which in turn contribute to enhance the process in a self-accelerating loop (positive feedback). For instance, water leakage from reservoirs through dissolutional conduits contributes to enlarge the size of the underground pathways, stimulating higher leakage and karstification. Mosul Dam on the Tigris River, Iraq, was built on a Miocene gypsum-bearing formation. The dam, located 50 km upstream of Mosul city with 3 million inhabitants, is one of the most strategic infrastructures of the country and is currently considered as one of the most dangerous in the world. Since the initial filling, a grouting program was applied along the dam foundation to arrest leakage, dam settlement, and sinkhole formation. The construction of the unfinished Badush Dam was started in 1988 upstream of Mosul city to act as a backstop structure in the event of a dam burst. The interruption of the grouting program after the capture of the infrastructure by the Islamic State created an alarming high risk and uncertainty situation. Basic geomorphic studies at the site, including mapping of karst features (e.g., sinkholes), would have allowed the anticipation of the problems.

Use of Landforms for Identifying, Assessing, and Solving Engineering Problems

Engineering Geology is defined by the IAEG as "the science devoted to the investigation, study and solution of the engineering and environmental problems which may arise as the result of the interaction between geology and the works and activities of man, as well as to the prediction and of the development of measures for prevention or remediation of geological hazards." Therefore, the main focus of Engineering Geology is the wide range of problems related to the interaction between engineering projects and the geological environment. Since engineering works and human activity are essentially developed on the surface of the Earth, landforms, and the associated deposits, as products that record earth surface processes, are of special usefulness for engineering geologists. In fact, one of the key roles of engineering geologists is the definition of a geological model for the areas under investigation, including geomorphology as one of the main aspects of the model. Landforms help in answering a number of crucial questions during the selection of a site for an engineering project, as well as during the design and development of the project:

1. Which are the potential problems that may be encountered?

Geomorphological mapping is a very powerful tool for identifying potential hazardous processes and difficult ground conditions (Smith et al. 2011). These can be recognized through multiple types of landforms such as landslides (Figs. 3b, 4, 6), sinkholes (Figs. 1d, 5, 7), dunes (Figs. 2c, 8), flood-prone areas (Fig. 5), and fault scarps (Gutiérrez and Gutiérrez 2016). Today, there is a great amount of high-quality remote-sensed data (aerial photographs, LIDAR data, and satellite images) from most regions of the world. However, this valuable information should never be considered as a substitute for field work. Detailed field surveying of the area under investigation by professionals capable of producing a comprehensive geomorphological map and model will most probably help to increase the success of the engineering project. We only see what we know. The geomorphological background can be used as the basis for selecting a site for a project, relocating a previously proposed site (hazard avoidance), planning more detailed investigations, and preliminarily selecting potential corrective measures. The old city of Valdez in Alaska, located at low elevation along the coast and on the outwash plain of Valdez Glacier, was frequently damaged by flooding.

Landforms, Fig. 8 Railway affected by sand encroachment between Swakopmund and Walvis Bay in the Namib Desert, Namibia (© Francisco Gutiérrez)

After its devastation due to tsunami flooding related to the 1964 Great Alaska Earthquake, the city was relocated to a safer site selected by geomorphic mapping; a perched alluvial fan protected from sea waves by a series of bedrock ridges and islands.

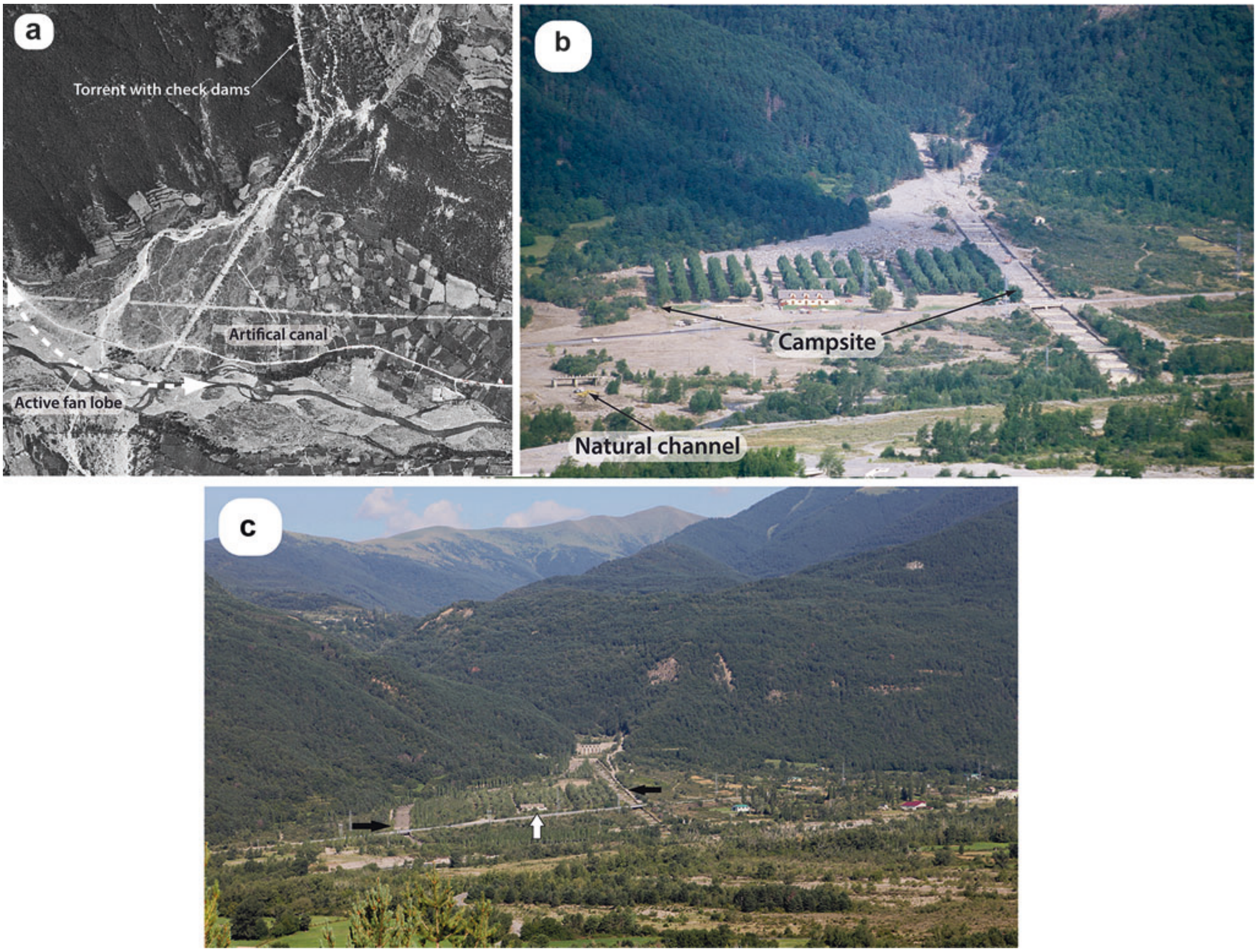
2. Can the identified hazardous processes be considered as active?

A key practical issue in landform and process characterization is to determine whether a specific geomorphic feature and process can be considered as active or inactive (state of activity). The term active may have different connotations depending on various factors, such as the landform type, the frequency of the process, the time scale considered, or the investigation method and its aims. A geomorphological process may be qualified as active if it is (1) operating at the present time (e.g., subsidence related to aquifer consolidation, dune migration); (2) the associated landforms have a fresh appearance (e.g., vegetation-free landslide scar); (3) has occurred in historical times and is expected to act in the near future (e.g., floods, gully erosion, avalanches, volcanic activity); or (3) there is geological evidence of activity in the Holocene or Quaternary record (e.g., seismogenic fault). Historical data may be sufficient for demonstrating the recency and expected activity of some processes (e.g. floods in an alluvial fan) (Fig. 9). In the case of low-frequency processes, especially those with larger recurrences than the temporal length of the historical record (e.g., earthquakes, tsunamis, some large landslides), the application of geochronological methods and cartographic relationships may be indispensable. Cartographic and geometric relationships between landforms and deposits (e.g. cross-cutting, inset, superposition) allow establishing a relative chronology, which can be highly useful for the application of geochronological methods. Numerical ages may provide an age for an event (e.g., landform produced by a paleoflood) or bracket its timing by dating older and younger units (e.g., age of an earthquake from faulted and non-faulted landforms). Geochronological methods are experiencing very rapid improvements, increasing their temporal range, and reducing the uncertainties and amount of required material.

An important concept that links with the state of activity of some processes characterized by episodic behavior is the elapsed time since the most recent event (MRE). For instance, Cruden and Varnes (1996) propose that inactive landslides are those that last moved more than one annual cycle of seasons ago. They subdivide inactive landslides into dormant and abandoned, depending on whether the causes of movement remain apparent or not, respectively (e.g., fluvial undercutting). For some hazardous processes, the identification and dating of the MRE require the application of specific techniques. The MRE on seismogenic faults is commonly inferred through the application of the trenching technique in combination with retrodeformation analyses and dating methods (McCalpin 2009), which have been also applied satisfactorily in sinkhole and landslide investigations (e.g., Gutiérrez et al. 2015). However, in some cases, the determination of the state of activity of some processes is difficult to resolve with confidence. By regulatory definition, faults may be considered as active if they have ruptured in the Holocene; the elapsed time of the most recent event is lower than 11.5 kyr. However, a trenching and dating investigation may lead to indeterminate results if the youngest faulted deposits are older than 11.5 ka or if the fault is buried by undeformed deposits younger than 11.5 ka (McCalpin 2009).

3. Which are the patterns and rates of activity?

Hazardous processes may have different temporal patterns that should be identified for the proper assessment of their activity. Some processes are characterized by continuous and rather constant activity. For instance, sagging sinkholes related to salt dissolution may be affected by rapid and steady



Landforms, Fig. 9 (**a**) Aerial photograph from 1957 of the Arás alluvial fan, Gállego River valley, Spanish Pyrenees, with an artificial canal with limited discharge capacity (ca. 150 m^3/s) along its medial line. The fan was fed by a steep torrent with around 30 check dams constructed with relatively weak masonry walls. The image clearly shows the active lobe of the fan to the left of the artificial canal. (**b**) Image of the Arás fan taken a few days after the 7 August 1996 flash flood, which killed 87 people in the camp site built on the active lobe of the fan (house and plot with planted trees). The flood destroyed most of the check dams releasing a large amount of debris that buried the artificial canal at the fan apex. The hyper-concentrated sheet flood expanded over the active fan portion situated at a lower elevation than the artificial canal, depositing a lobe of metre-sized boulders. (**c**) Image of the Arás fan taken in 2013 showing the costly correction measures applied after the 1996 flood. A large check dam was built at the fan apex, as well as a large diversion channel following the path of a pre-existing natural channel carved in the active fan lobe (left black arrow). The diversion canal functions when the water stage in the central canal (right black arrow) overflows a diversion weir. The white arrow indicates the location of the abandoned campsite (© Francisco Gutiérrez)

subsidence (Fig. 5). Other processes show an episodic regime, with long periods of quiescence punctuated by short events of activity. This behavior is common in tectonic and gravitational faults, volcanoes, some landslides, collapse sinkholes, or floods in ephemeral alluvial systems. A combination of both styles of activity is also possible, with periods of gradual activity and episodes of acceleration or catastrophic behavior (e.g., discharge in perennial rivers). Earth surface processes typically involve changes in the shape of landforms, which can be monitored through the application of numerous methods commonly used in geomorphological investigations. Our ability to detect and measure precisely changes in the land surface at different spatial and temporal scales has greatly increased in recent times thanks to the development of new geodetic techniques, such as LIDAR (Laser Imaging Detection and Ranging), DGPS (Differential Global Positioning System), and DInSAR (Interferometric Synthetic Aperture Radar). Processes that operate continuously or with high frequencies can be measured directly through the application of instrumental methods. Generally, the activity of processes with episodic behavior and high recurrence needs to be assessed indirectly, using the geomorphic and stratigraphic record. Morphometric parameters can be used to indirectly estimate various variables related to the magnitude and activity of past processes (e.g., peak discharge of a paleoflood, run out, and speed of mass movements).

4. Is it possible to identify the most hazardous and safe zones and define a spatial boundary for the zones potentially affected by the hazardous processes?

Once a hazardous process has been identified in an area, a safe alternative is to adopt a preventive strategy by avoiding the hazardous zones. This can be carried out through the development of susceptibility and hazard maps, which indicate the probability of an area being affected by the hazardous process. Susceptibility models indicate relative probability, whereas hazard models provide estimates of spatial-temporal probability. These maps should allow identifying not only the most hazardous zones but also the safest area, which can be selected for siting an engineered structure or as refuge zone for evacuation schemes. The production of these maps through various approaches (e.g. statistical, heuristic) is largely based on cartographic inventories that provide information on the spatial distribution of the process in the past, frequently recorded by landforms (e.g., landslides, sinkholes). The underlying assumption of these models is that the hazardous processes will have a similar spatial distribution pattern in the future as in the past. The past is the key for the future (reverse Uniformitarianism). However, this assumption may not be valid, especially in areas affected by engineering projects and human activities that may significantly alter the dynamics of processes.

At a more detailed scale, a very important issue for preventive planning is to define the precise boundaries of the areas affected by hazardous processes or difficult ground conditions (e.g., edge of a landslide deposit), which is commonly carried out on the basis of landform mapping and analysis. An additional step is to establish a setback distance from the mapped hazardous feature (e.g., sinkhole, active fault, landslide, retreating cliff, shifting channels, erosional beaches) that should be avoided for siting structures. These setback lines are frequently based on geomorphic criteria and landform analysis. For instance minimum setback distances on the upthrown and downthrown sides of dip-slip faults, characterized by asymmetric surface rupture style, may be defined from the toe and crest of fault scarps (e.g., McCalpin 2009). Setback distances should account for various aspects such us the uncertainty of the previous mapping (e.g., concealed active fault), the probable spatial migration of the landform and associated processes (e.g., expanding landslide), long-term and short-term trends inferred from geomorphic and historical analyses, or the time horizon and changes related to future scenarios. For instance, setback lines in beach-dune coastal sections affected by erosion and flooding should be established considering the expected retreat rates related to the current human-induced sea-level rise.

5. Which is the probability of occurrence?

In case an engineering project can be affected by a hazardous process, a key issue is to assess the temporal probability of the process occurring in the area under consideration or surpassing a magnitude (probability of exceedance). The annual frequency or probability is usually expressed by its inverse; the return period or recurrence interval. Ideally, we should produce magnitude and frequency relationships for the hazardous processes incorporating epistemic and aleatoric uncertainties (probabilistic hazard analysis). These relationships are used to establish critical design parameters for engineering projects, commonly determined by return periods established by regulations (e.g., peak ground acceleration, peak discharge, wave runup). Their upper bounds may also indicate the maximum expected magnitude for a hazardous process in the area. For instance, the maximum diameter of a collapse sinkhole at the time of formation is constrained by the maximum cavity span that can withstand a rock formation and its mechanical strength. The maximum expected earthquake is determined by the size of the active faults in the region. Hazard curves are frequently described by exponential functions and typically show linear patterns when plotted in semilogarithmic graphs (e.g., earthquakes, floods, landslides, sinkholes). Typically, small events occur regularly, whereas large-magnitude events have very low frequencies.

The empirical magnitude and frequency relationships are constructed with data on past events obtained from the instrumental, historical, and geological record. Generally, the temporal length of the instrumental and historical records is insufficient to reliably assess the probability of low frequency and large magnitude events, which are the most important from the hazard and engineering perspective. Geomorphology and Quaternary geology investigations provide the means to expand the human record analyzing geological archives of past hazardous processes (e.g., paleohydrology, paleoseismology). In the studies that use the prehistorical geological record, geochronology plays an instrumental role, since numerical ages are indispensable for incorporating data on paleoevents in hazard assessments. The length of the record can be also expanded virtually applying the ergodic assumption, by substituting space by time. For instance, landslides mapped in an area of 100 km^2 and formed over a decade could be considered equivalent to the landslides that may occur in an investigation zone 10 km^2 over a century.

Generally, it is implicitly assumed that magnitude and frequency relationships generated from past events can be extrapolated to the future. However, hazard curves may change substantially, especially in the case of some nonendogenous processes whose dynamics can be drastically altered by human activity and engineering projects. For instance, dams may reduce dramatically the frequency of floods due to their laminating effect. In contrast, major dam projects may significantly increase landslide hazard in the slopes of the reservoir and downstream of the dam, where rivers with lower sediment load tend to downcut and undermine the slopes. Moreover, some hazardous processes do not have a stochastic temporal distribution. Conversely, they may show temporal clusters, like floods controlled by climate variability or their probability may depend on the time elapsed since the MRE, like processes with "memory" such as earthquakes (conditional probability).

6. Which are the expected environmental impacts?

Geomorphological analysis allows one to identify and appraise landscapes and landforms with special value (e.g., scientific, aesthetic, educational) that may be adversely affected or destroyed by engineering projects resulting in environmental impacts (Panizza 1996). It also provides the basis for designing protection strategies and lower-impact alternatives. Geomorphological investigations should be also conducted to predict how engineering works will impact on earth surface processes, which may also involve human-induced hazards (Goudie 2006). Reduction of water and sediment flux in fluvial systems by the construction of dams or flow diversion can induce a cascade of adverse effects, including impacts and hazards. Rapid incision downstream of dams may accelerate erosion processes in the trunk and tributary channels and induce a decline in the water table with adverse effects on agriculture. Lower sediment supply in deltas and coastal environments will make it difficult to arrest retreat in the coastline with large economic implications. The construction of groynes on a coast may affect littoral drift inducing accelerated beach expansion and destruction with potential environmental, economic, and legal implications.

7. Which measures could be implemented to effectively reduce hazards and impacts?

The design and application of measures aimed at mitigating hazards and impacts should be based on a comprehensive knowledge of the area, as well as a sound characterization of the processes and landforms involved in the potential engineering and environmental problems. A proper diagnosis of the problem is essential for the selection of effective correction measures. For instance, in the case of slope instability problems, the type of landslide, which can be inferred from its geomorphological features, is essential for selecting adequate measures. Other relevant aspects that should be addressed in the analysis include controlling factors, spatial and temporal patterns, and rates of activity. For instance, the main problem that affects the 444 km long Haramian high-speed railway that connects Makka and Al Madinah in Saudi Arabia is the potential hazardous processes related to aeolian activity. These processes may occur along a total of ca. 200 km of the infrastructure and include damage by windblown sand on the platform and the trains, loss of visibility related to dust storms, and sand encroachment by shifting dunes. Geomorphic mapping allows the identification of the problematic sections of the railway, including those that traverse active dune fields (Fig. 10). It also provides practical information on the activity of the process, including the prevailing wind direction or the rate of dune migration through the comparison of remote-sensed data from different dates. This information will be highly useful for the design of corrective measures that will require expensive maintenance works, such as trenches, embankments, and walls on the upwind side of the railway to block the movement of sand and dunes.

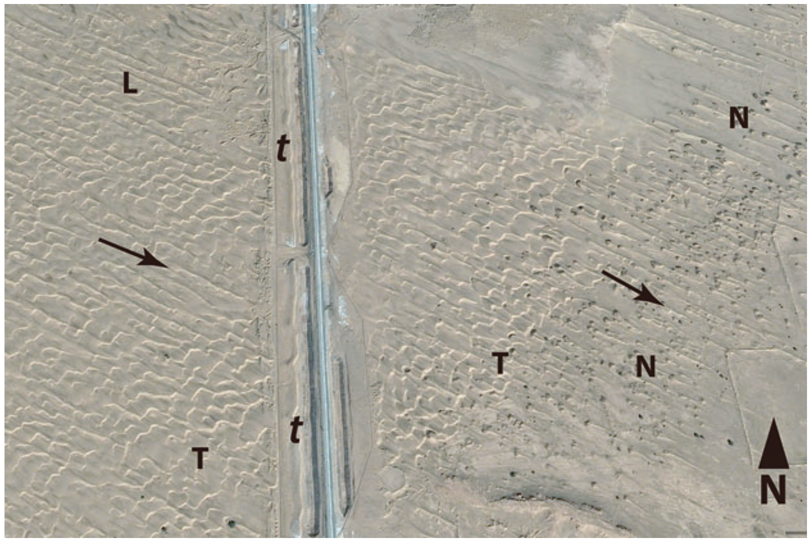
Summary

Landforms are distinctive features of the land surface shaped by erosion, accumulation, or deformational processes that involve the movement of mass (rock, sediment, water). Landforms are normally classified according to their genesis within three main fields of geomorphological investigation: *structural geomorphology*, covering landforms controlled by geological factors; *climatic geomorphology*, including landforms developed under specific climatic conditions); and *non-zonal geomorphology*, comprising landforms shaped by geomophological processes that occur in most climatic zones.

The study of landforms is crucial for (paleo)environmental reconstructions, prediction of the spatial distribution, magnitude and frequency of geomorphological processes (e.g., hazard assessments), and the analysis of local and global environmental impacts.

A proper understanding of landform genesis and evolution is of paramount importance for the successful planning, design, and implementation of engineering projects. Geomorphological mapping can be essential for identifying the suitable site for a project and understanding the processes occurring in the area, including those that may adversely affect the development or feasibility of the project itself. Landforms can be dated, monitored, and modeled providing significant clues for the interpretation of past, present, and future earth surface processes, both subaerial and submarine.

Since most engineering works are developed at the Earth's surface, the study of landforms – produced by processes occurring at different temporal and spatial scales – is of special usefulness for engineering geologists. In this context,



Landforms, Fig. 10 Satellite image captured on January 2016 showing a stretch of the Haramain high-speed railway that connects Makka and Al Madinah in the coastal plain of the Red Sea, Saudi Arabia, around 60 km north of Jeddah. The infrastructure traverses an active dune field. The direction of the prevailing wind can be inferred from the orientation of longitudinal dunes (L), transverse dunes (T) and large nebkha dunes (N) developed on the lee side of vegetation. Trenches (t) have been excavated on the upwind side of the railway to trap the sand and prevent sand encroachment. This dune field could have been avoided by tracing the railway route ca. 3 km to the west (© Francisco Gutiérrez)

the analysis and assessment of the state of activity of landforms (active, dormant, inactive), especially in highly dynamic environments, provide important clues for hazard and risk assessments and related mitigation measures.

Cross-References

- ▶ Aeolian Processes
- ▶ Coastal Environments
- ▶ Desert Environments
- ▶ Designing Site Investigations
- ▶ Engineering Geological Maps
- ▶ Engineering Geomorphology
- ▶ Environments
- ▶ Erosion
- ▶ Floods
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- ▶ Permafrost
- ▶ Risk Mapping
- ▶ Sea Level
- ▶ Sinkholes
- ▶ Surface Rupture
- ▶ Vegetation Cover
- ▶ Volcanic Environments

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Landslide

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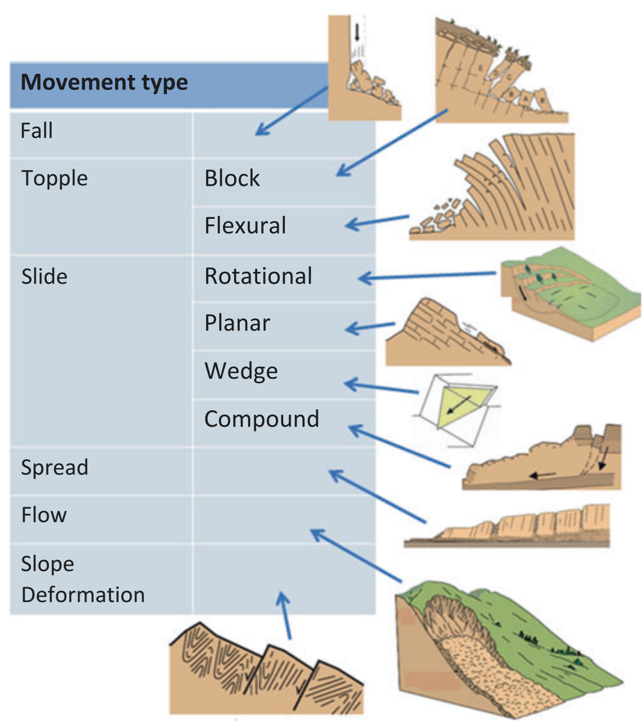
Definition

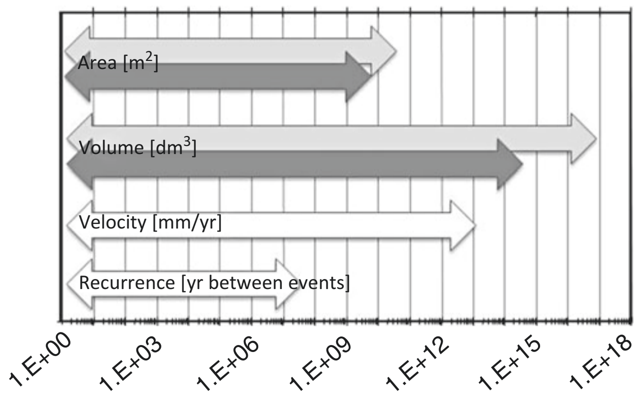
A landslide is the gravitational downslope movement of solids on natural or artificial slopes. The solids are geotechnical materials that can contain water, ice, and air; however the solids are volumetrically dominant over the transport medium (water, ice, and air).

Landslides have been classified based on the movement types (falls, topples, slides, spreads, flows, and complex) and the material involved (*bedrock*, debris, soils) (Varnes 1978). This classification is broadly accepted although *engineering properties* of materials (rock, clay, mud, silt, sand, gravel, boulders, debris, peat, and ice) rather than the division into debris and earth are today used, and the term complex is not further recommended, whereas the term slope deformation is introduced as a movement type (Fig. 1) (Hungr et al. 2014). Therefore, ice is today reconsidered as a possible material. In contrast snow is considered only in Scandinavia as a material in landslide science and snow *avalanche* science is in general a scientific discipline apart. Although intensively discussed, there is still no agreement on a hierarchical organization of the landslide classification, and there is no general agreement on umbrella terms (e.g., massive rock slope failures, rockslides).

Landslides have a wide spectrum of behavior depending on the material content and movement type but also on the geologic, geotechnical, and geomorphic *environments* of their occurrence (e.g., on land, subaquatic, in permafrost environment). For example, landslides have a wide range of magnitudes in terms of area, volume velocity, and recurrence interval (Fig. 2). However all types of landslides can be destructive when impacting settlements or *infrastructure*, and many can cause loss of life including those that impact



Landslide, Fig. 1 Classification of landslide movement types (Simplified after Hungr et al. (2014))



Landslide, Fig. 2 Diagram showing the wide spread of magnitudes of landslide phenomena illustrated here in terms of area, volume, speed, and recurrence time. *Light gray arrow* shows subaquatic landslides, *dark gray* shows landslides on land, and *white arrow* represents both environments together

on individuals traveling far from any settlements. The largest events causing loss of life have been reported in relation to massive failures from nonvolcanic rocks or volcanoes and their secondary effects, displacement waves, and landslide damming with related dam breaching. Individual landslides and their secondary effects have caused up to several tens to more than a hundred thousand deaths (Evans et al. 2006).

In order to prevent or mitigate landslide disasters, several strategies have been applied, including stabilization, control of the landslide path, avoiding development in landslide prone areas, and early warning practices. Prerequisite to all that work is an appropriate understanding of landslide hazards and their consequences in terms of the spatial extent, temporal occurrence, magnitude (size), and intensity (velocity). Several mapping products have been developed which span from inventory, to susceptibility, and then to *hazard* and *risk* maps (Fell et al. 2008). Modern remote sensing data such as Synthetic Aperture Radar (SAR) and airborne Light Detection and Ranging (*Lidar*) and high-resolution optical data obtained from planes, satellites, and most recently drones allow the mapping of small *deformations* that often precede failure. In combination with geodetic tools that allow today to detect displacements of only few millimeters, even over large areas, geologists are today better equipped to classify the *hazard* and *risk* of landslides than only a few decades ago (Hermanns et al. 2013). However, as long as people settle in mountain environments or even in subdued landscapes formed by weak rocks or sediments such as quick clay or close to coastal bluffs, the landslide risk can be effectively reduced by landslide mapping in combination with *land use* planning and mitigation measures, but a zero risk will never be achieved. This is because landslides are often spontaneously triggered by *earthquakes* or meteorological events that cannot be predicted or only predicted a short time ahead, respectively.

Cross-References

▶ Avalanche
▶ Bedrock
▶ Deformation
▶ Earthquake
▶ Engineering Properties
▶ Hazard
▶ Infrastructure
▶ Land Use
▶ Lidar
▶ Mass Movement
▶ Monitoring
▶ Mountain Environments
▶ Quick Clay
▶ Remote Sensing
▶ Risk Mapping
▶ Stabilization
▶ Water

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Laplace Equation

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Definition

The Laplace equation is a second-order partial differential equation of a mathematical function attributed to Pierre Simon Laplace based on a paper written in 1784 (Ball, 1908). In Cartesian coordinates x, y, and z, the second derivative of the function $f(x,y,z)$ is expressed with the use of the Laplacian operator $\nabla^2 f$ of the Laplace equation in the form

$$\nabla^2 f = \frac{\partial^2 f}{\partial x^2} + \frac{\partial^2 f}{\partial y^2} + \frac{\partial^2 f}{\partial z^2} = 0$$

where $f(x,y,z)$ is a harmonic function for which the three independent variables x, y, and z are solutions (Jeffrey, 2002). The Laplace equation is important in potential fields including gravitational attraction, electrical current, and groundwater head. In steady-state groundwater flow following Darcy's law in an isotropic aquifer with no additions or losses of water, the Laplace equation is $\nabla^2 h = 0$. Darcy's law relates unit discharge, q in each direction per unit of aquifer width, to the product of hydraulic conductivity (K) and the change in head (h) in each direction ($q_x = -K(\partial h/\partial x)$, $q_y = -K(\partial h/\partial y)$, $q_z = -K(\partial h/\partial z)$). Steady-state groundwater flow has no transient flow (Domenico and Schwartz, 1998). Isotropic aquifer conditions require that hydraulic conductivity is equal in all three Cartesian directions ($K_x = K_y = K_z = K$).

Additions or losses of water (sources or sinks) in groundwater flow could be injections or withdrawals from a well completed in the aquifer. If steady-state groundwater flow is subject to an increase or decrease (W) across an aquifer reach, then the Laplace equation ($\nabla^2 h = 0$) becomes the Poisson equation (Domenico and Schwartz, 1998), which is $\nabla^2 h = W/K$.

The Laplace equation can be solved numerically using the finite difference method after the problem is discretized in the x-y plane. For the groundwater flow problem, the quantities at each node in the finite difference mesh would be head and flow. Head would be a scalar ($h_{i,j}$), whereas flow would be a vector separated into the x and y components ($q_{x(i,j)}$, $q_{y(i,j)}$). The second derivative is obtained by calculating the first derivative of the value and then repeating the procedure to the result.

An attempt to use the Laplace equation with age as a surrogate for potential was proposed by Hirano (1993) using bedding planes in sedimentary rock units as iso-potential surfaces with cross-cutting geologic structures being represented as boundary conditions.

Cross-References

- ▶ Aquifer
- ▶ Darcy's Law
- ▶ Groundwater
- ▶ Poisson's Ratio

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Laser Scanning

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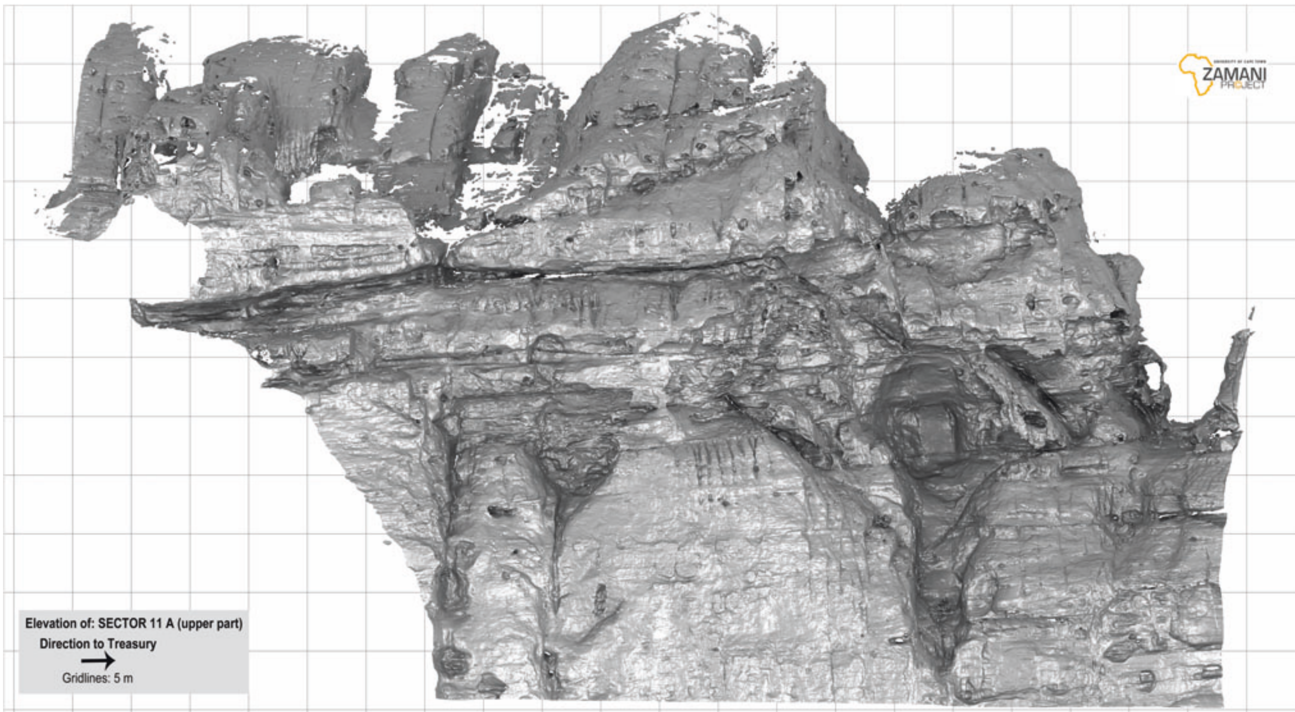
Definition

Traditionally mapping of surfaces for geological applications relied on conventional survey techniques and analogue and analytical photogrammetry. Technological developments made it possible to move from the measurement of individual points and the derivation of contours to the collection of mass surface data. The introduction of digital photogrammetry enabled the automatic simultaneous extraction of thousands of surface points or point clouds and the subsequent generation of meshed surfaces and spatial models.

The advent of Lidar (Light Detection and Ranging), also referred to as laser scanning, provided a new approach to the creation of point clouds. A laser scanner determines the polar coordinates of a surface point by measuring the distance from the scanner to the surface to be mapped, together with the horizontal and vertical angles of the vector from the instrument to the point. These polar coordinates are then transformed to Cartesian xyz coordinates (Rüther et al. 2009). This is basically the function of an electronic theodolite or Total Station, the difference being that a scanner can carry out up to a million of such measurements per second while moving stepwise from point to point, whereas Total Stations measure individual points. In this process, distances are measured by the time-of-flight method, phase shift measurement, frequency difference method, or triangulation.

In the time-of-flight method, a short laser pulse is sent from the scanner to the object surface and (retro)reflected to the scanner and the distance is derived from the travel time of the pulse. The phase shift and frequency difference methods employ continuous waves. In the former method, the phase shift between the outgoing and the returned signal is measured. As this shift increases proportionally to the scanner-object distance, the required distance can be determined. Ambiguities will occur if only one wavelength is used and these are resolved by using more than one frequency. In this method, the laser-carrier wave is amplitude-modulated to create a superimposed wave for the phase shift measurement. In the frequency



Laser Scanning, Fig. 1 Ortho image of a rock wall (Siq, Petra) showing detail of the rock surface derived by laser scanning

difference method, the frequency of the carrier is modulated and frequency differences between outgoing and incoming wave serve to determine the distance (Fig. 1).

In the triangulation method, an emitter and a receiver are mounted at the end of a base, and a light (laser) beam is sent to the object and reflected to the receiver. Unlike in the other methods, where the retroreflective signal is measured, here the object surface serves as a mirror and the angle of incidence of the reflected signal is measured. This angle is proportional to the distance.

Scanners can operate stationary or mobile. As stationary Terrestrial Laser Scanners (TLS), they are typically mounted on a tripod (Rüther et al. 2012), while Mobile Mapping Systems (MMS) make use of planes (Aerial Laser Scanning), drones or ground-based vehicles. In MMSs, the position of the scanner, and thus the co-ordinate reference system, changes continuously. High-precision navigation and positioning units have therefore to be connected to the scanner in order to obtain a point cloud in which all points are referenced to the same co-ordinate system. These units comprise of high precision GNNS (Global Navigation Satellite Systems) and Inertial Measurement Units (IMU). IUMs contain sets of gyroscopes and accelerometers making it possible to record linear movements, speed and rotations, thus providing an uninterrupted flow of precise information on the positions and orientation of the scanner. Combinations of scanner, IMU and GNSS, have become so small and lightweight that they can be carried by small drones.

Terrestrial, aerial, and mobile laser scanning have been employed in geological applications for the past decade (Buckley et al. 2008; Nguy et al. 2011; Bellian et al. 2005).

Cross-References

- ► Aerial Photography
- ► Insar
- ► Lidar

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Lateral Pressure

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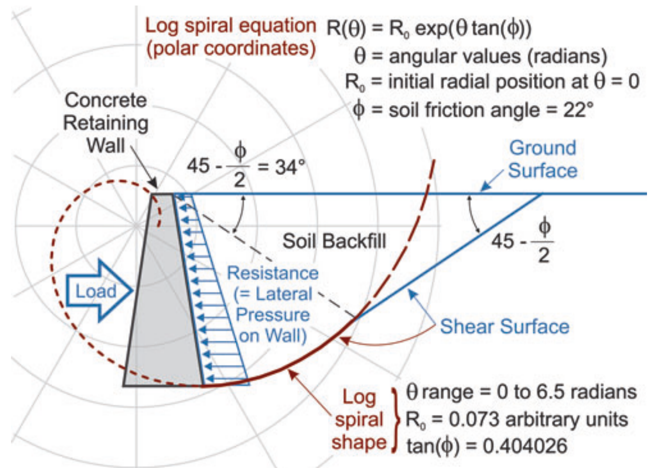
Definition

Lateral pressure in engineering geology refers to the horizontal effect of geostatic stress or overburden pressure.

In geotechnical engineering, lateral Earth pressure refers to stress on a retaining wall imposed by the soil behind it pushing on the wall (active earth pressure), to stress on a retaining structure or wall imposed by loads pushing the wall into the soil behind it (passive earth pressure), or to an ideal condition in which the wall movement is essentially zero (earth pressure at rest). The ratio of the horizontal component of effective stress (σ_x') to the vertical component of effective stress (σ_z') is the lateral earth pressure coefficient (K), which is similar to Poisson's ratio for elastic materials. Groundwater saturation of the soil behind a wall results is hydrostatic pressure, which produces a horizontal pressure equal to the unit weight of water times the height of water per unit length of wall, in addition to the lateral pressure imposed by the soil. The vertical effective stress per unit length of retaining wall is calculated by multiplying the unit weight of soil by the height of soil, using moist unit weight of soil above the groundwater level and the saturated unit weight of soil below the groundwater level. The calculation must take into account soil layers of different unit weight. The horizontal effective stress per unit length of retaining wall is calculated by multiplying the vertical effective stress by the appropriate active (K_A), passive (K_P), or at rest (K_o) Earth pressure coefficient.

Three theories of lateral Earth pressure have been developed: Rankine, Coulomb, and log-spiral (Anonymous 2002). The Rankine theory assumes that the retaining wall is smooth and vertical, which neglects shear stress developed along the soil-wall interface. The Coulomb theory accounts for retaining walls that are rough and inclined. Both Rankine and Coulomb theories assume that a planar failure surface develops in the soil. The log-spiral theory is based on the Coulomb theory and assumes that the failure surface geometry extending upward from the base of the retaining wall is described by the equation of a log-spiral (Fig. 1). All theories allow for backfill behind the retaining wall to be inclined upward away from the wall. For a simple case of level, noncohesive soil backfill behind a vertical wall, the Rankine earth pressure coefficients are



Lateral Pressure, Fig. 1 Cross section of a gravity retaining wall in response to a horizontal load that is resisted by the lateral earth pressure of the soil on the wall. Geometry of shear surface is log-spiral shape and Rankine discussed in Coduto et al. (2016)

$$K_A = \tan^2\left(45^\circ - \frac{\phi}{2}\right) \tag{1}$$

$$K_P = \tan^2\left(45^\circ + \frac{\phi}{2}\right) \tag{2}$$

$$K_o = 1 - \sin\phi \tag{3}$$

The Coulomb earth pressure coefficients are

$$K_A = \frac{\cos^2(\phi)}{\cos\delta\left(1 + \sqrt{\frac{\sin(\phi+\delta)\sin\phi}{\cos\delta}}\right)^2} \tag{4}$$

$$K_P = \frac{\cos^2(\phi)}{\cos\delta\left(1 - \sqrt{\frac{\sin^2\phi}{\cos\delta}}\right)^2} \tag{5}$$

where ϕ is the angle of internal friction of the soil and δ is the angle of inclination of the resultant force of Earth pressure acting on the wall because of wall roughness and soil friction. The Earth pressure at rest for Coulomb theory is the same as for the Rankine theory. Earth pressure coefficients for cohesive soil backfill include more terms because of the shear strength of the soil (Anonymous 2002).

Cross-References

- ▶ Angle of Internal Friction
- ▶ Cohesive Soils
- ▶ Effective Stress
- ▶ Geostatic Stress
- ▶ Noncohesive Soils
- ▶ Poisson's Ratio
- ▶ Retaining Structures

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Levees

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Synonyms

Dikes; Embankments

Definition

Natural levees routinely develop along alluvial streams in flood plains, when a river overtops its natural banks during an overbank flood or the flood stage (Fig. 1, section 2). Deposition is favored in the flow transition area because of a dramatic decrease in the depth of flow divided by the wetted perimeter (hydraulic radius). The increased friction hastened by shallow flow over the normally vegetated flood plain decreases flow velocity, allowing entrained sediment to settle and deposit along the banks. Natural levees are often difficult to discern because they may be locally breached and re-worked by subsequent overbank events. Consequently, natural levees are usually discontinuous features (Leopold et al. 1964).

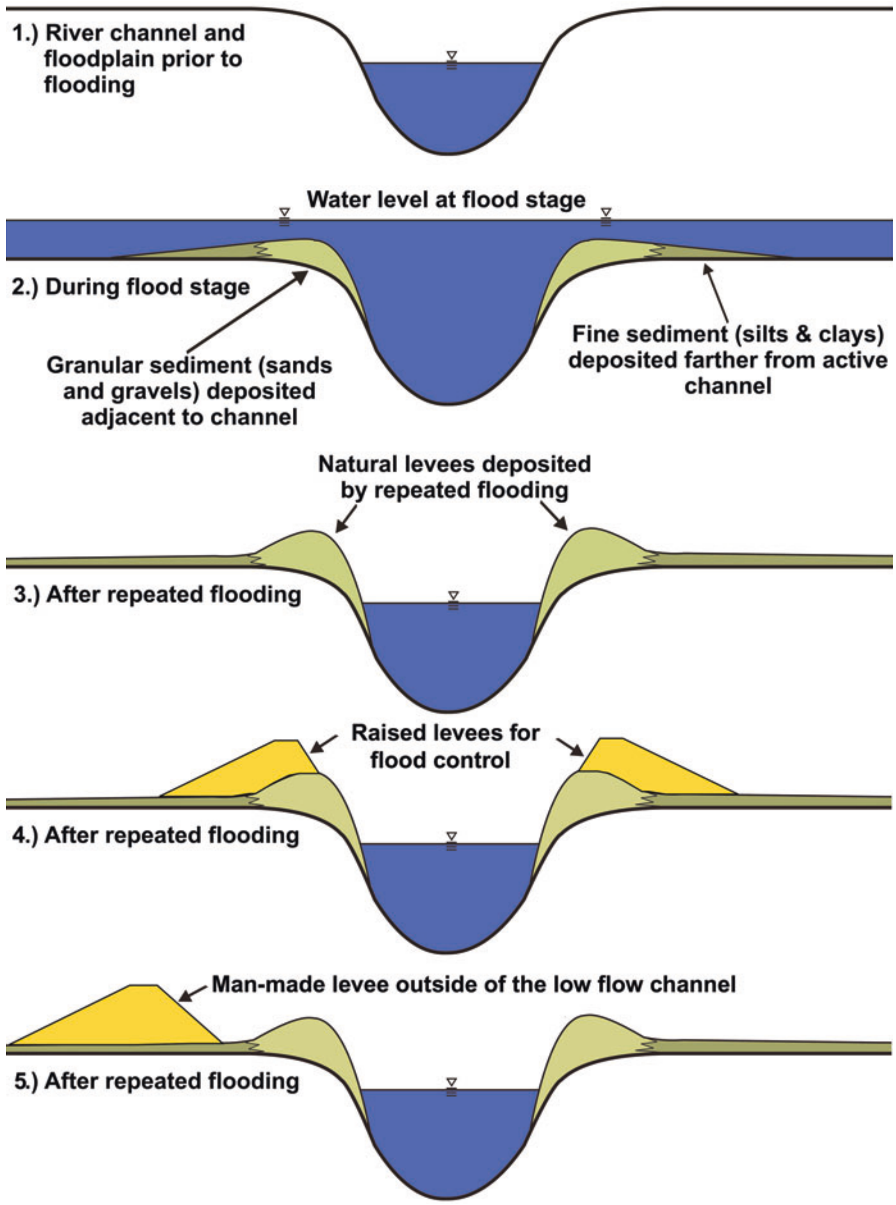
Once formed, natural levees tend to become heightened through repeated sequences of overtopping (Fig. 1, section 3). Levee formation is often hastened by breaches that quickly erode low cohesion materials (principally silt and sand), excavating crevasses, which allow high velocity flows, which upon reaching the landward slope of the levee, split into distributaries and deposit silt-sand mixtures eroded from the natural levee, thereby elevating its landward flanks.

Man-made levees are dikes or embankments built parallel to the banks of a channel, lake, or body of water that are intended to protect reclaimed areas from flood inundation or to confine the flow within a natural channel. In some instances, such artificial structures have been constructed on top of natural levees (Fig. 1, section 4). The use of "heightened levees" was very popular in the USA from the post-Civil War era up until the 1970s, commonly referred to as the "levees only policy" for flood control by the federal government (Leopold and Maddock Jr 1954; Hoyt and Langbein 1955).

Since the 1973 Mississippi River and 1993 lower Missouri River floods (IAFMRC 1994), there has been a conscious effort to locate man-made levees on firmer foundations (Fig. 1, section 5). These are often referred to as "*setback levees*" because they are set back from the low flow channel and are often situated on less pervious foundations than exist along the active channel, and are, thereby, less prone to underseepage problems. Setback levees also allow for a less constricted channel and increased flood storage.

In the wake of Hurricanes Katrina and Rita in 2005, there has emerged a movement to replenish natural deltas that are being starved of sediment because of upstream reservoirs that catch much of the fine-grained sediment. Evolving concepts envision "training levees" closer to the low flow channels, and "porous levees" or "gated levees" that can safely allow water to pass through or across existing flood control levees onto the natural flood plains (Galloway et al. 2013). Many of these concepts are currently under evaluation in the Mississippi Delta (Day et al. 2014; Kemp et al. 2014).

Levees, Fig. 1 Schematic illustrations of natural and artificial levees



Cross-References

▶ Current Action
▶ Floods
▶ Fluvial Environments

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Lidar

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Synonyms

Laser scanning

Definition

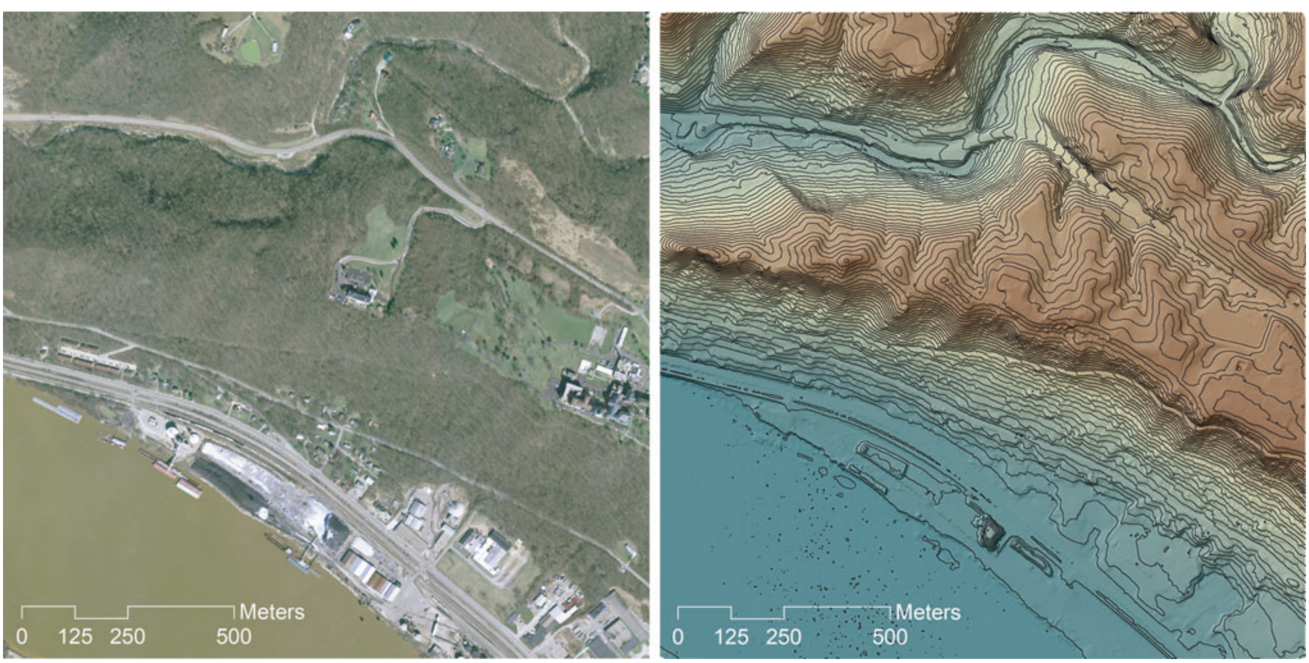
Lidar, sometimes written as LiDAR, is a distance measuring technology based on the reflection of pulses from a laser scanner, which is used to create highly detailed digital representations of topographic surfaces. The name originated during the 1960s as a combination of the words light and radar. Two kinds of lidar are commonly used in engineering geology applications: airborne and terrestrial (Fig. 1).

Characteristics

In airborne lidar applications, a laser scanner is coupled with a differential global positioning system (GPS) and an inertial reference unit (IRU) in a fixed-wing aircraft or helicopter to scan Earth's surface. The scanner measures distance and orientation relative to the aircraft, and the IRU and GPS are used to determine the orientation and location of the aircraft. Airborne lidar is sometimes referred to as airborne laser scanning (ALS) or airborne laser swath mapping (ALSM). In terrestrial lidar applications, a scanner is affixed to a tripod and returns are recorded from a fixed point. Terrestrial lidar is useful for scanning nearly vertical surfaces such as surface mine walls.

Lidar pulse rates range from tens to hundreds of thousands of pulses per second and scanners can record multiple returns from each pulse, with return densities as high as tens or hundreds per square meter. Pulses reflected from vegetation or structures can be filtered to produce bare-earth surface models, shaded relief renderings, and contour maps of value to engineering geologists; this is one of the major benefits of lidar in heavily forested areas. Bathymetric lidar is used to map near-shore bathymetry in areas of where water is relatively shallow and clear. Full-waveform lidar units provide more information about each return than do conventional lidar units, which record only the peak intensity of each return, but have not yet come into widespread use in engineering geology.

The returns recorded by the laser scanner are referred to as a point cloud, in which returns can be unclassified or classified into categories such as ground, low vegetation, high vegetation, and other categories. Point clouds include the horizontal and vertical coordinates of the points from which pulses were reflected as well as additional information such as



Lidar, Fig. 1 Comparison of leaf-off color orthophoto (0.5 m pixel size) with a bare earth airborne lidar digital elevation model rendering (2 m pixel size) of a landslide prone area along the Ohio River near Cincinnati, USA, illustrating the utility of airborne lidar digital elevation models in forested areas. Data source: Ohio Statewide Imagery Program the off-nadir angles of the pulses, classification of the return (e.g., ground, low vegetation, high vegetation, unclassified), intensity of the returns, and in some cases red-green-blue (RGB) color codes.

Irregularly spaced returns in a point cloud can be used to create surfaces by triangulation (triangulated irregular network or TIN) or interpolation onto a regular grid (digital elevation model or DEM; sometimes called a digital terrain model or DTM).

Once a TIN or DEM surface is created, attributes such as slope angle, aspect, curvature, and various forms of roughness can be calculated to accentuate landforms and support geomorphological analysis. In cases where rock faces are scanned, the slope angle and aspect correspond to the dip angle and dip direction of discontinuities such as joints and faults. Lidar surface models are useful for high-resolution engineering geologic mapping, rock slope characterization, and modeling applications such as watershed scale slope stability analyses (e.g., Haneberg et al. 2009; Lan et al. 2010; Jaboyedoff et al. 2012; Abellán et al. 2014).

Cross-References

► Laser Scanning
► Remote Sensing

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Limestone

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Synonyms

Coquina; Oolitic limestone; Shelly limestone; Travertine; Tufa

Definition

Sedimentary rocks consisting mainly of calcium carbonate (calcite and, to some extent, aragonite), but often containing some ferroan calcite, magnesian calcite, or dolomite.

Context

Limestone may constitute 10% of the volume of all sedimentary rocks. Most is formed in marine environments but others form in fluvial or lacustrine environments. They were absent from the early geological record but, due to atmospheric change following the evolution of photosynthetic organisms, became more common at various times particularly during the Phanerozoic (Chilingar et al. 1967).

Calcite is dissolved or precipitated from water depending on the water temperature, acidity/alkalinity (pH), and presence of carbon dioxide and dissolved ion concentrations. Calcite and aragonite are soluble in even mildly acidic waters. Aragonite dissolves more readily unless protected by organic tissues (Morse and Berner 1972). Calcite can also dissolve at pressure points and move in solution to lower pressure zones to precipitate as crystalline calcite. Because of these properties, limestone tends to cement with redeposited calcite in complicated stages. Some may become almost completely recrystallized. Therefore, limestone ranges from loosely cemented to hard and completely cemented rock. Some cementation can take place soon after deposition. During diagenesis, if sufficient magnesium is present, limestone may be partly or wholly dolomitized (Porter 2007; Pray and Murray 1965).

Other calcite in limestone directly precipitates from oversaturated solutions in hypersaline marine or lake environments. But much limestone, partly or wholly, contains or consists of the remains of calcium carbonate from protective, support or skeletal structures and secretions of a wide variety of organisms (e.g., some algae and annelids and many mollusks, brachiopods, and corals). Most limestone consists largely of such debris. Some that are the result of large organic associations, such as coral reefs called **bioherms,** have high topography, but others that are thinner and laterally extensive are termed **biostromes**.

Classifications

There are two main, complementary, classifications of limestone. Both have advantages and disadvantages.

Folk's classification (Folk 1959) describes the composition of grains and interstitial material in terms of types of grains (allochems), fine-grained matrix (micrite), and crystalline cement (sparite). Grains include shell debris (bioclasts), ooids (small ovoidal bodies which accrete in layers), peloids

L

Limestone, Table 1 Folk Classification (Adapted from Folk 1959 as modified by Kendall and Flood 2005)

Matrix	Over 66% micrite (lime mud)				Micrite and sparite roughly equal	Over 66% sparite (crystalline calcite)		
% of allochems	0–1%	1–10%	10–50%	Over 50%				
Description	Matrix supported			Grain supported				
	Micrite	Scattered allochems	Sparse allochems	Packed allochems	Poorly washed sparite	Unsorted allochems	Sorted allochems	Rounded allochems
Types of allochems								
Bioclasts		Ooids (Ooliths)		Peloids (pellets)		Intraclasts		
Fossils and fragments of shells		Small ovoid accreted grains		Pellets of carbonate some of which are of fecal origin		Eroded fragments of limestone from within the depositional area (as opposed to extraclasts from outside the depositional area)		

(grains which are pellet like and some of which are fecal), fragments of locally eroded limestone (intraclasts), and erosional limestone debris from elsewhere (extraclasts). The first part of each name refers to types of grains and the second to the matrix (e.g., oospartite, intramicrite) but, if several types of allochems occur together in significant proportions, hybrid terms such as intraoomicrite or biooosparite are used. This terminology is sometimes applied to some limestone in the field but usually requires examination of thin-sections or peels (Table 1).

Dunham's classification (Dunham 1962) focused on depositional textures of four main groups based on: relative proportions of coarse clastic particles; whether or not the grains were originally in mutual contact; or whether the rock is characterized by frame-building biogenic structures, such as corals or algal mats. It partly reflects the original porosity of the rock. It was designed to be readily applicable to hand specimens in the field or cuttings from drilling operations (Table 2).

Some other terms that are used for specific types of limestone that fall outside these classifications are:

- Coquina – a limestone consisting mainly of carbonate shell debris (largely from mollusk or brachiopods) which is, in the Folk classification, a biomicrite or biosparite depending on the nature of the surrounding matrix.
- Travertine and tufa – layered deposits precipitated from springs (especially hot springs), with travertine less porous than tufa (Ford and Pedley 1996).

Impurities

Most limestone contains impurities including: siliceous skeletal material (sponge spicules, diatoms, radiolaria) or clay, silt, and sand introduced by winds and rivers. Some contain silica in the form of chert (chalcedony, flint, jasper); barite (or other carbonates); or oxides, hydroxides, and sulfides of iron and other elements. Impurities give a wide range of colors especially when the rock is weathered. Limestone can

Limestone, Table 2 Dunham's Classification (Adapted and simplified from Dunham 1992 as modified by Kendall and Flood 2005)

Constituents not bound together when deposited				Bound together at the time of deposition
Contains calcium carbonate particles of clay and silt size (mud) = micrite			No mud	Intergrown skeletal structures such as coral, bryozoan and algal bioherms and biostromes (but usually with cavities containing deposited sediments)
Mud matrix supported		Grain supported		
Less than 10% grains	More than 10% grains	Grains in mud matrix	Grains in crystalline calcite matrix	
Mudstone	Wackestone	Packstone	Grainstone	Boundstone

grade laterally into sandy, silty, or clayey carbonate rocks that are transitional to sandstone, siltstone, and mudstone. Impurities may limit suitability for some practical uses.

Marine Environments

Most marine limestone consists mainly of **skeletal material** of marine organisms. Some organisms, especially certain corals, stromatoporoids, algae, and annelids, slowly construct reefs (bioherms) mainly in tropical waters. Reef building corals get part of their nutrition from symbiotic algae that need sunlight for photosynthesis of nutrients. Sunlight does not penetrate to great depths in the marine environment that limits the depth of reef formation.

Shallow marine waters are widely saturated to supersaturated in calcium carbonate so calcite or aragonite debris accumulate without dissolving. Deep waters are undersaturated with calcium carbonate because solubility increases with increasing pressure, salinity, and decreasing temperature. Therefore, at greater depths, dissolution occurs. At a depth known as the lysocline, the rate of dissolution increases dramatically. Below that level is the carbonate compensation depth, beneath which the rate of supply of calcite equals the rate of dissolution, so calcite is not deposited. The depth of the lysocline currently averages about 4,500 m below sea level but varies between oceans and is at greater depths in Equatorial than Polar Regions (a range of about 4000–6000 m) because of the varying chemical composition and temperature of the water. The depth of the lysocline has varied through geological time (Morse and Berner 1972).

Because of fluctuations of conditions in the intertidal zone, carbonate sediments can begin to lithify very quickly, sometimes over a few decades, giving rise to "beach rock" (Hanor 1978). Limestone is susceptible to bioerosion by many types of marine organisms.

Non-Marine Environments

In terrestrial environments, rainwater is usually slightly acidic and can readily dissolve limestone to form solution hollows, caves, and sinkholes (karst landscapes) which include major engineering hazards. The resulting supersaturated waters lead to redeposition in caves (e.g., stalactites, stalagmites) or in rivers, often around waterfalls or cold or hot springs, to form compact, banded, travertine, or porous tufa deposits. Accumulations of freshwater mollusk shells can also form limestone (Chilingar et al. 1967).

Uses

Limestone has numerous uses, depending on strength and purity (British Geological Survey 2006), including:

- **Building materials,** e.g., building stone, construction aggregates, lime, cement, and mortar
- **Fillers** and as white pigment in paper, plastics, toothpaste, paint, and pharmaceutical products
- Soil conditioners to neutralize acidic soils
- Food or water additives to provide nutritional calcium for people and livestock
- Removal of sulfur dioxide from flue gases to reduce atmospheric pollution
- Reduction of corrosion of pipes by increasing the alkalinity of water after purification
- As an additive in manufacture of some types of glass
- For sculpture and ornamental work

Cross-References

► Aggregate
► Building Stone
► Karst
► Lacustrine Deposits
► Marine Environments
► Sedimentary Rocks

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L

Liners

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Synonyms

Compacted clay soil liner; Geomembrane; Geosynthetic clay liner

Definition

Liners protect against contamination of groundwater, surrounding soil, and air by providing a barrier against diffusive and hydraulic transport of water and/or leachate solutes and solutions and diffusive transport of undesirable gasses. A liner system can consist of a compacted clay soil, geomembrane, geosynthetic clay liner, and/or a combination of these (Qian et al. 2002). Liners are widely used in landfills and waste impoundments, to cap new waste disposal units, to close old waste disposal sites, impound lagoons, potable water containment, and to line tunnels.

Compacted Clay Liners

Compacted clay soil liners are constructed in lifts that typically have a maximum thickness of 150 mm after compaction (Daniel 1993). Compaction densifies the soil and reduces hydraulic conductivity by reducing porosity. Compaction also remolds the soil into a homogeneous mass of dispersed parallel layers free of large voids. Dispersed particles arranged in parallel layers lower hydraulic conductivity by creating tortuous paths for the flow of water. Typical design objectives for compacted clay soil liners include: low hydraulic conductivity to minimize leakage ($k \leq 10^{-7}$ cm/sec), minimal shrinkage potential to minimize cracking, and adequate shear strength to maintain liner stability. The US EPA recommends that soil liners be compacted to a water content and dry unit weight that leads to low hydraulic conductivity and adequate engineering performance with respect to other considerations such as shear strength. As a result, clay soil liners often are compacted slightly wet of optimum water content, which maintains acceptable strength and reduces hydraulic conductivity. Soils with high plasticity are undesirable because they form hard clots when dry and are too sticky when wet. The minimum soil composition specifications for reaching hydraulic conductivity $\leq 10^{-7}$ cm/sec are shown in Table 1.

Geomembranes

Geomembranes are thin sheets of prefabricated flexible thermoplastic or thermoset polymeric materials. They are widely used as landfill liners and covers due to their inherent impermeability that provides a functional liquid and/or vapor barrier (Qian et al. 2002). Geomembranes are placed directly on the subgrade or another geosynthetic liner and seamed together according to procedures specified by the manufacturer. Common materials used to manufacture geomembranes are high density polyethylene (HDPE), linear low-density polyethylene (LLDPE), flexible polypropylene (fPP), and polyvinyl chloride (PVC), and chlorosulfonated polyethylene (CSPE). Recommended minimum thickness for LLDPE, fPP, PVC, and CSPE geomembranes is 0.75 mm. The recommended thickness for a geomembrane constructed of HDPE is 1.5 mm to allow for extrusion seaming (Qian et al. 2002). HDPE is the most effective barrier against water and methane transmission, whereas PVC is the least effective. However, the permeability of geomembranes is 10^3 to 10^6 times lower than compacted clay liners. Therefore, geomembranes constructed of thermoplastic or thermoset polymeric materials are essentially impermeable.

Liners, Table 1 Minimum soil composition specifications to obtain hydraulic conductivity $\leq 10^{-7}$ cm/sec (MIT OpenCourseWare – Compacted Soil Liners 2017)

Fines (<75 μm)	20–30%
Gravel (≥4.76 mm)	≤30%
Maximum particle size	25–50 mm
Plasticity index	7–10%

Placement of a geomembrane on the subgrade (compacted clay soil liner) presents a number of challenges. Moving the membrane to where it needs to be, laying it down, and seaming it together is difficult because compacted clay liners are wet. Membrane liners can wrinkle or wave from exposure to direct sunlight or high ambient temperature. This reduces the contact between materials which is essential for low leakage rates. Terminating the geomembrane in an anchor trench at the top of the slope must be done carefully to prevent liquids from wicking to adjacent soil. The controlling design feature for geomembranes is how well the membrane accommodates unbalanced shear stresses when loaded on slopes. Typical slopes using textured (reinforced) geomembranes are 3(H):1(V).

Geosynthetic Clay Liners

Geosynthetic clay liners (GCLs) are relatively thin hydraulic barriers typically constructed of 5 kg/m^2 bentonite clay bonded onto or fixed between two sheets of geomembrane or geotextile. A geomembrane is impervious to liquid as long as it maintains its integrity. Geotextiles are either woven or nonwoven material less impervious to liquid than a geomembrane but more resistant to penetration damage. GCLs are manufactured in continuous sheets and are installed by unrolling and overlapping the edges and ends of the panels. The overlaps self-seal when the bentonite hydrates (Qian et al. 2002). Manufacturers determine the required seam overlap at adjoining sheets to guard against potential opening of the barrier system. Laboratory tests demonstrate that bentonite will maintain an effective barrier system by self-sealing holes up to 75 mm in diameter (US EPA 2001).

Geomembrane GCLs are fixed to the clay via adhesive. Geotextile GCLs affix to the clay using adhesives, stitchbonding, needlepunching, or a combination of these three methods. Stitchbonding adds considerable shear strength and needle punching yields an even stronger more rigid GCL. Needle punching requires nonwoven geotextile on at least one side. This provides enhanced interface frictional resistance to the adjoining clay layer. Nonwoven geotextiles are usually more expensive than woven textiles. However, needle punched GCLs are particularly well suited for lining slopes because of their enhanced strength (Qian et al. 2002; US EPA 2001). GCLs can be susceptible to damage during transport and installation. Hydration should be avoided because unconfined swelling of the bentonite clay can reduce the integrity of the barrier by pulling the geotextile layers apart.

GCLs have low hydraulic conductivity, install quickly and easily, self-repair rips and holes, and are cost effective in regions where clay is not readily available. GCLs maximize landfill space and minimize excavation work for a given landfill volume because they are much thinner than compacted clay liners and caps.

Cross-References

- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Compaction
- ▶ Conductivity
- ▶ Contamination
- ▶ Effective Stress
- ▶ Geotextiles
- ▶ Groundwater
- ▶ Hydrocompaction
- ▶ Hydrology
- ▶ Infiltration
- ▶ Landfill
- ▶ Pollution
- ▶ Soil Properties
- ▶ Water

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Liquefaction

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Definition

Liquefaction is a physical process in which the strength or effective stress of the soil is reduced due to shaking of the ground. Typically, the ground shaking necessary to initiate liquefaction is caused by earthquake loading.

Liquefaction occurs in saturated sandy soils, where the space between the soil grains, known as pores, is filled with water. The pressure exerted by the water on the soil grains is called the pore pressure, and it relates inversely to the effective stress of the soil. Initially, the pore pressure is relatively low. However, rapid loading, such as earthquake shaking, can cause the pore pressure to increase, which, in turn, reduces the strength of the soil. This reduction in strength can cause the soil to behave as a viscous liquid rather than as a solid; this phenomenon is known as liquefaction.

L

Introduction

Throughout history, earthquake-induced liquefaction has caused extensive ground, structural, and lifeline damage around the world. Liquefaction-induced damage captured the attention of the geological/geotechnical engineering community after the dramatic and infamous liquefaction failures that resulted from the 1964 earthquakes in Japan and Alaska (Seed 1979). Since 1964, liquefaction-related failures have been commonly observed in large earthquakes. The damage includes sinking or tilting of buildings, subsidence or lateral displacement of ground, sand blows, and slope failures. Lateral displacement of ground is the most persistent type of liquefaction-induced ground failure.

As Seed (1987) points out, in relation to liquefaction, the engineer is posed with two questions: (1) given a likely seismic event, is the soil prone to liquefy? and (2) if liquefaction occurs, what consequences can be expected in terms of ground movements or lateral displacements? The first question is generally addressed by estimating the liquefaction potential that is developed by empirical correlations based on *in situ* index tests, such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), Shear Wave Velocity (Vs), and Becker Penetration Test (BPT). The most widely used empirical liquefaction correlation is the "simplified procedure" by Seed and Idriss (1971).

Recent research on the development of empirical liquefaction correlations represents an update to the datasets combined with probabilistic and advanced computing techniques (Oommen et al. 2010). The second question is addressed by estimating the amount of displacement due to lateral spreading, which is a function of the physical and mechanical characteristics of the soil layers at the site (Youd et al. 2002).

Cross-References

- ▶ Artificial Ground
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Cone Penetrometer
- ▶ Earthquake
- ▶ Ground Shaking
- ▶ Hazard
- ▶ Liquid Limit
- ▶ Plastic Limit
- ▶ Pore Pressure
- ▶ Sand
- ▶ Shear Strength
- ▶ Soil Properties

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Liquid Limit

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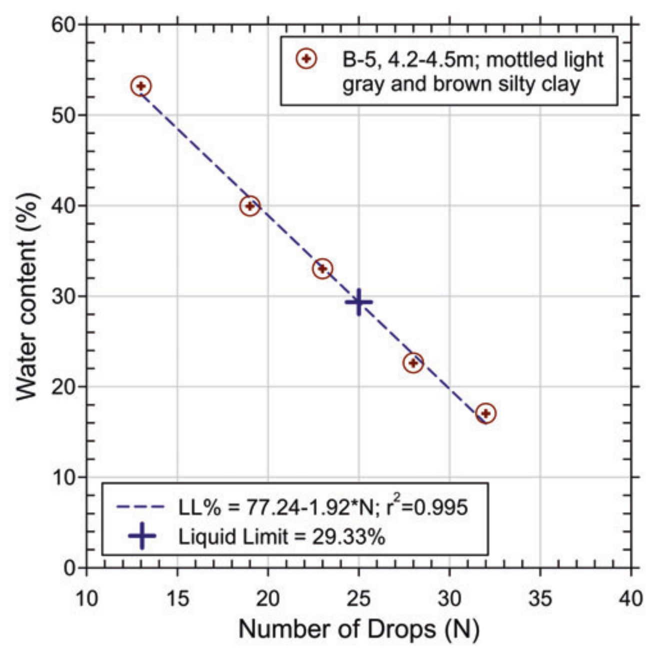
Definition

The water content at which a soil-water paste changes from a plastic to a liquid-like consistency as indicated by the behavior of a standard groove carved into a pat of soil molded into a round-bottomed brass cup which is part of a standard liquid limit test device.

The liquid limit is one of the measured parameters of the Atterberg limits test, which is used for differentiating consistency states of finer particles in soil material. If coarser particles are present (coarse sand, gravel, cobbles), the finer particles act as matrix and may govern the behavior of the soil mass. Consistency states of soil depend on water content; with increasing water, the consistency states are solid, semi-solid, plastic, and liquid.

The standard liquid limit test device was designed by Arthur Casagrande in the 1930s based on the procedure developed by Albert Atterberg; therefore, the liquid limit test is sometimes called the Casagrande test. The soil pat is smoothed across the brass cup to have a maximum thickness of 10 mm; a standard grooving tool is used to make a groove completely through the soil pat that is 2 mm wide at the bottom, 11 mm wide at the top, and 8 mm deep (ASTM 2010). The brass cup is hinged on one edge so that a cam shaft with a hand-operated crank can be used to raise the cup and allow it to drop abruptly 10 mm onto a hard rubber base at a rate of 120 drops per minute. The number of drops required to cause the soil to flow from both sides to close the groove over a distance of 13.5 mm is recorded; a sample of soil from the section that closed the groove is collected for determination of the water content. The test is repeated at least five times



Liquid Limit, Fig. 1 Liquid limit determination from a plot of the number of drops of the standard cup required to close a standard groove in a soil pat against the water content of the soil. The liquid limit is the water content of the soil that would correspond to the standard groove closing in 25 drops of the cup with the soil pat having different water contents such that the number of drops required to close the groove ranges from about 15 to about 35. The water content and the number of drops are used to calculate the water content that would close the standard groove the standard distance of 13.5 mm in exactly 25 drops (Fig. 1).

The second measured parameter of Atterberg limits test is the plastic limit. The Atterberg limits test also includes the plasticity index, which is calculated as the difference between the liquid limit and the plastic limit. All Atterberg limits are determined on samples of soil that pass the #40 sieve (ASTM 2009), which has 0.42-mm openings (medium sand size and smaller, including silt and clay sizes, that may be part of the soil material).

Cross-References

- ▶ Atterberg Limits
- ▶ Casagrande Test
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Cohesive Soils
- ▶ Plastic Limit
- ▶ Soil Laboratory Tests
- ▶ Soil Properties

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Loess

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Definition

An unstratified aeolian deposit composed largely of silt-size grains that are loosely cemented by calcium carbonate.

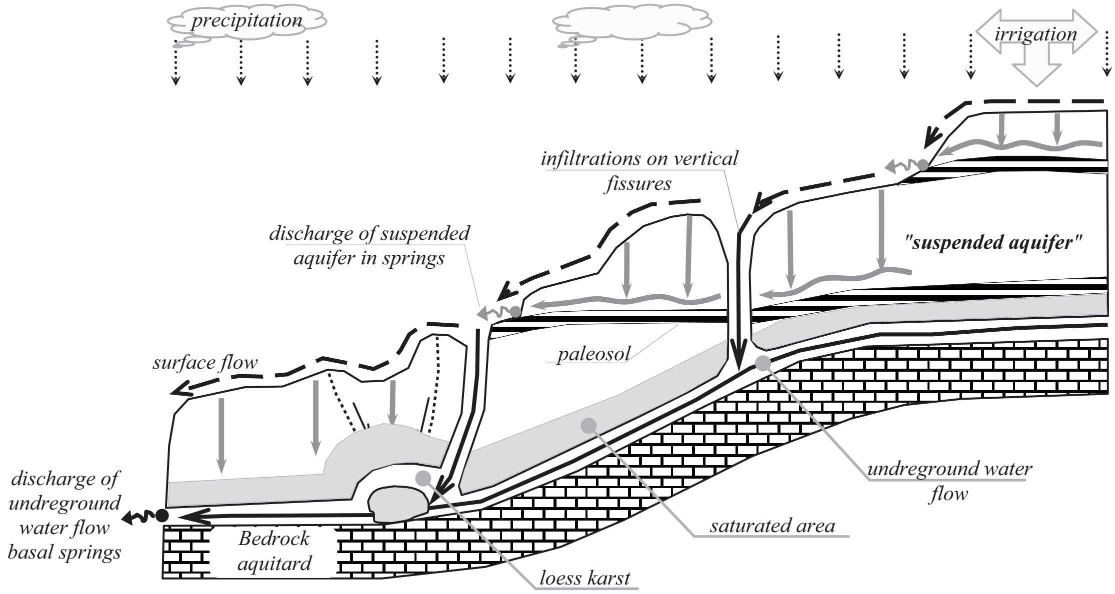
Loess formations cover more than 10% of the land surface of the world in both the Northern Hemisphere between 25° and 55°N (China, Siberia, Paris and Danube Basins) and Southern Hemisphere between 30° and 40° S (Uruguay, Argentina and New Zealand). This peculiar type of sedimentary deposit, worldwide, has similar characteristics which derive from aeolian deposition and has variable thickness from decimetres to tens of metres. Natural slopes are almost vertical and in arid conditions may attain 300–500 m (Derbyshire 1983; Liu 1985). Often, loess sequences contain rust-coloured paleosol intercalations which contrast with yellowish loess deposits.

The mineralogical composition of loess depends on the source of the dust and may contain 40 to 60 minerals grouped in two classes (Howayek et al. 2011): passive ones (quartz, feldspar, mica and heavy minerals) and active minerals (carbonates, sulphates, readily soluble salts, soluble oxides and hydroxides and clay minerals). The most prevalent and active is calcium carbonate that can be present as cementing bonds of grains, as disseminations, depositions on fissures and loess-dolls (concretions of nodules).

Most frequently, loess deposits are made mainly of silt fraction (>60%). If clay or sand contents exceed 20%, deposits are named clayey or sandy loess. Grains of silt or sand are bonded by four types of forces (Osipov and Sokolov 1994): molecular, ionic-electrostatic, capillary, and chemical, which are either unstable or dependent on water saturation.

Loess deposits have, beside inter-grain or inter-aggregate pores, additional macropores, mostly extending vertically. This specific feature (named an open metastable structure) leads to unique physical properties such as low dry densities (1.155–1.4 g/cm^3), high anisotropic porosity (40–70%), and low compressive strength. The most characteristic mechanical feature derived from macropore structure and weak bonding systems is collapsibility, which is the property of being stable in unsaturated conditions but to exhibit appreciable volume changes and alteration of physical properties in the saturated state (reduction of cohesion by 2/3 parts), under static external loading and sometimes even under its proper supplementary weight. The measure of the collapsibility is the difference between settlements measured in dry and wet conditions during a double-oedometric test. For reference loading pressures of 200–300 KPa, this parameter (collapse index *Ie/Im*) may, in severe cases, achieve levels of 10–18%.

Rain or irrigation waters may easily infiltrate loess formations through vertical fissures and the descending water movement is frequently aided by vertical macropores. Temporary suspended aquifers may subsist in rainy seasons allowing groundwater to dissolve soluble minerals, to hydrodynamically detach insoluble particles and to create wells, pipes, ravines, sinkholes, and gully erosion (see Fig. 1 modified after Billard



Loess, Fig. 1 Schematic model of water routing in thin (ca. 10 m thick) loess at Lanzhou (After Billard et al. 1993)

et.al. 1993). In thick loess deposits, this phenomenon can produce systems of large subsurface pipes, tunnels and caves, named “loess pseudo-karst”.

In some cases when loess formations serve as foundation terrain or construction material, in the absence of special design measures, these deposits may be hazardous, producing failure of engineering structures or huge flow slides (e.g., Teton Dam, Idaho, USA 1976; numerous earthquake induced landslides in Gansu Province, China, 1920).

Cross-References

- ► Aeolian Processes
- ► Geohazards
- ► Landslide
- ► Soil Properties

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M

Manning Formula

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Definition

The Manning formula is an empirical relationship relating depth-averaged velocity of water flowing in open-channel conditions ($\bar{v}$) to hydraulic radius (R), slope of the energy grade line (S_e), and channel roughness (n), which is known as Manning's n value. Open-channel conditions are under the influence of gravity without pressure forcing water movement. In engineering geology applications, natural river and stream channels are of most interest; however, the Manning formula also applies to canals and pipes, provided that pressure flow does not occur. The Manning formula is

$$\bar{v} = \frac{1}{n} R^{\frac{2}{3}} S_e^{\frac{1}{2}} \tag{1}$$

where $\bar{v}$ is in m/s, R is in m (cross-section area, A, divided by wetted perimeter, P), and S_e is in m/m, resulting in n having the units of s/m$^{1/3}$ for $\bar{v}$ in US customary units (ft/s), a conversion factor of $1.486 = \sqrt[3]{3.281 \text{ ft/m}}$ is needed. The channel geometry typically can be measured or estimated; the energy grade line is more challenging to determine and commonly it is approximated by the slope of the channel. Manning's n value is complicated and guidance for estimating it uses components related to channel irregularities, variations in channel cross section, obstructions in the channel, vegetation in and adjacent to the channel, and channel meandering characteristics (Arcement and Schneider 2005).

The Darcy-Weisbach formula for turbulent flow and the equivalent Chézy equation use an absolute value of roughness (k_s) in m, which represents the size of irregularities on the channel bed and banks. Comparing these formulae with the Manning formula reveals that Manning's n value is not a constant for a particular stream channel, but varies with hydraulic radius (R) (Kruger 2006).

$$n = \frac{R^{\frac{1}{6}}}{18 \log\left(\frac{12\,R}{k_s}\right)} \tag{2}$$

Cross-References

- ▶ Chézy Formula
- ▶ Fluvial Environments

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Marine Environments

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Synonyms

Abyssal environments; Coastal; Continental shelf; Ocean floor; Oceanic island

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Definition

Marine environments are those in the World's Seas and Oceans below mean high tide mark.

Marine environments relate to the morphology of near shore, off shore, and deep water zones of the ocean. Current classifications focus mainly on marine topography, habitats, and fisheries rather than engineering characteristics (Fig. 1).

Submarine Geomorphology

Marine features include:

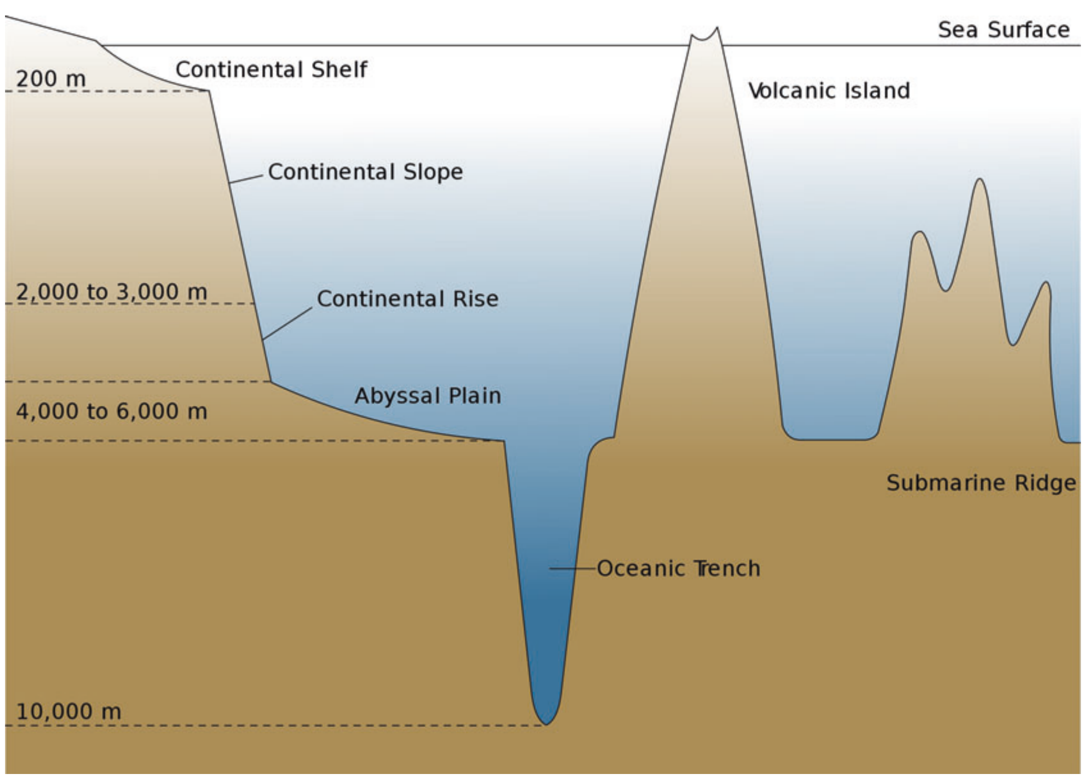
- Continental shelves, sometimes with reefs and river deltas; marginal canyons in continental slopes
- Seamounts (guyots), which sometimes support atolls
- Ocean rises and plateaux
- Mid-ocean ridge systems
- Subduction trenches with volcanoes and strong seismic activity
- Abyssal plains

Although there is some interest in exploiting deep ocean minerals, the main current focus for engineering geology is on near shore construction (coast protection and port works, jetties, piers, tidal energy facilities, and wind farms) or increasingly, in deeper waters of the continental shelves and oceanic plateaux, for hydrocarbon exploitation. As on land, substrates may consist of gravel, sand, silt, mud, or bedrock some of which are relict from past lower sea-levels, and some reflect more recent depositional processes. On shallow shelves the distribution and thickness of sediments often reflect past geomorphology such as submerged former river valleys and channels (Hedgepeth 1957).

Dynamic Processes

Some marine environments are strongly affected by dynamic changes due to tides, storms, and sediment instability. Hydrodynamic processes affecting submarine sediments are most vigorous at:

(a) Or close to, the shore where strong currents and storms cause shoreline and submarine erosion, shoreline and offshore sediment movement and deposition. Offshore currents also carry sediments across the shelf to submarine channels and canyons and into the deeper ocean.
(b) Continental margins where sediments are fed into deeper water along submarine channels or poorly consolidated sediments can be dislodged, for instance, by earthquakes, to form submarine slides and turbidity currents. Major sediment displacements can sever submarine cables.
(c) Margins of oceanic islands, atolls, and plateaux.



Marine Environments, Fig. 1 Ocean floor topography (Source: Wikimedia, Public Domain)

Key issues engineering geologists requiring careful study of substrates, processes, and geomorphology are:

- The nature, composition, and thickness of sea floor sediments
- Patterns of erosion, deposition, and potential instability
- Dynamic of the water column (currents and storms)
- Pollution of sediments and water
- Environmental impacts of proposed investigations and works (Erickson 1996; Seibold and Berger 1994)

Cross-References

- ▶ Beach Replenishment
- ▶ Coast Defenses
- ▶ Coastal Environments
- ▶ Environments
- ▶ Erosion
- ▶ Geohazards
- ▶ Hazard
- ▶ Landforms
- ▶ Mass Movement
- ▶ Nearshore Structures
- ▶ Sea Level
- ▶ Sediments
- ▶ Tsunamis

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Mass Movement

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Synonyms

Avalanche; Landslide; Mass wasting; Slope failure; Slope instability

Definition

Mass movement represents a broad spectrum of gravity-dominated down slope movements of snow, ice, water, soil, debris, and rock comprising submarine and terrestrial landslides, including soil creep, and snow avalanches.

Introduction

Mass movement is the displacement of material down slope under the force of gravity, and the material involved can include any combination of snow, ice, water, soil, debris, and rock. However, this relatively straightforward definition could be taken to include a multitude of processes that are not usually regarded as part of mass movement studies. The movement of water, containing minimal sediment load, down hill slopes and in rivers where the flow is primarily a function of fluid dynamics should be excluded from the definition of mass movement, as this is part of fluvial geomorphology and hydrology. However, the distinction between fluvial flows and very fluid sediment-laden forms of mass movement is somewhat arbitrary, and most mass movements do include water, either in its fluid or solid state, as part of the displaced mass. Volcanic pyroclastic and fluid lava flows are also normally excluded from the definition, being part of volcanology, although the down slope movement of the mixture of volcanic ash and water known as a "lahar" is usually included as these represent a particular form of mass movement that is not unique to volcanic debris. In some of the scientific literature, mass movement is taken to include gravity tectonics where uplift and massive gravitational sliding of large blocks of the Earth's surface occurs with movement at rates of <1 cm/year. However, it is more realistic to regard these processes as part of regional tectonics and therefore they are excluded from the definition used in this entry. The down slope movement of ice as part of a glacier could be construed as a mass movement process but again is normally excluded from the definition as it really concerned with glacial physics and normally studied by glaciologists.

Type of Mass Movement

Given the above qualifications, in engineering geology it is recommended that mass movement be divided into two main categories based on the nature of the material involved:

1. Landslides of soil, debris, and rock. Natural landslides and those in artificial materials (i.e., man-made) have been subject to a great deal of research in engineering geology. Landslides have a wide distribution, can be found in slopes that are <10°, and pose a significant risk to construction

development in many parts of the world. The term "landslide" includes terrestrial and submarine failures. Landslides range in scale from slow moving small-scale processes that fall under the category of soil creep to large-scale very rapid terrestrial rock avalanches and active submarine turbidity currents.

2. Snow avalanches. While snow avalanches do entrain soils and debris they are mainly comprised of snow and ice. Avalanches are a feature of mountainous regions that receive a significant snowfall and are most frequent where hillside slopes lie between 30 and 40°. The study of avalanches falls within the purview of "snow science" (International Snow Science Workshop 2014) although there will be some input from engineering geologists in the design of protection measures.

Landslides

There is a vast amount of literature on landslides with the most detailed studies represented by Turner and Schuster (1996) and Clague and Stead (2012). These two publications deal with all aspects of the identification, investigation, and treatment of all types of landslides. In addition, each year there are symposia and conferences on landslides presenting the latest research and case studies (e.g., Bromhead et al. 2000). There are detailed theoretical studies of the nature of landsliding processes (Hutchinson 2008) and websites that provide up-to-date information on landslides (BGS 2016; USGS 2016). There are also readily available popular books that have helped raise the public awareness of landslides and their associated hazards (Highland and Bobrowsky 2008). The scale of the risk from landslides to engineering works can be demonstrated by the Vaiont disaster in 1963 in Italy when a 250 million m^3 rockslide collapsed into a reservoir. This landslide sent a 100 m high flood wave over the crest of the newly constructed concrete arch dam, which survived. However, the flood destroyed five villages downstream and killed over 2000 people.

A broad subdivision can be made into terrestrial and submarine landslides, although the terminology to describe and classify both is essentially the same. The major difference is the potential size of the failure. Terrestrial landslides can range in size from a few cubic meters up to hundreds of cubic kilometers with movement speeds varying between <20 mm/year and >5 m/s. However, the largest landslides on Earth are submarine in nature and associated with either the collapse of mid-ocean hot spot volcanoes, such as in Hawaii and the Canary Islands, or the failure of sediment accumulations on the continental shelf, with the largest presently identified being the Agulhas Slide off South Africa that had a volume of around 20,000 km^3 and a run out distance (i.e., from crest to toe) of 140 km. When a submarine landslide travels down slope it will break up and, as it mixes with water, it becomes a turbidity current that can flow down slopes <1°. In terms of volume, turbidites, the deposits of turbidity currents are the most abundant type of deep-sea deposit (Piper et al. 2012). A significant secondary tsunami hazard is created by both submarine landslides and terrestrial landslides that enter a water body. Extraterrestrial landslides have also been identified on Mars, Mercury, and the Moon, although these are yet to have any engineering geological significance.

Hungr et al. (2014) suggested the mass movement phenomena of soil creep should also be considered a form of "landslide." Soil creep is the very slow (0.5–10 mm/year) down slope movement of the upper 1 m of surfical deposits on a slope (i.e., pedological soils and colluvium). The most prominent surface features created by soil creep are terracettes, which are long linear steps along the slope less than 0.5 m high.

Describing a Landslide

The general components of a landslide are shown in Fig. 1. Although this is a particular landside form known as multiple rotational, the terminology is applicable to all terrestrial and submarine landslide types. Using the standard terminology to describe a landslide facilitates communication between all parties involved in landslide investigations.

Classifying a Landslide

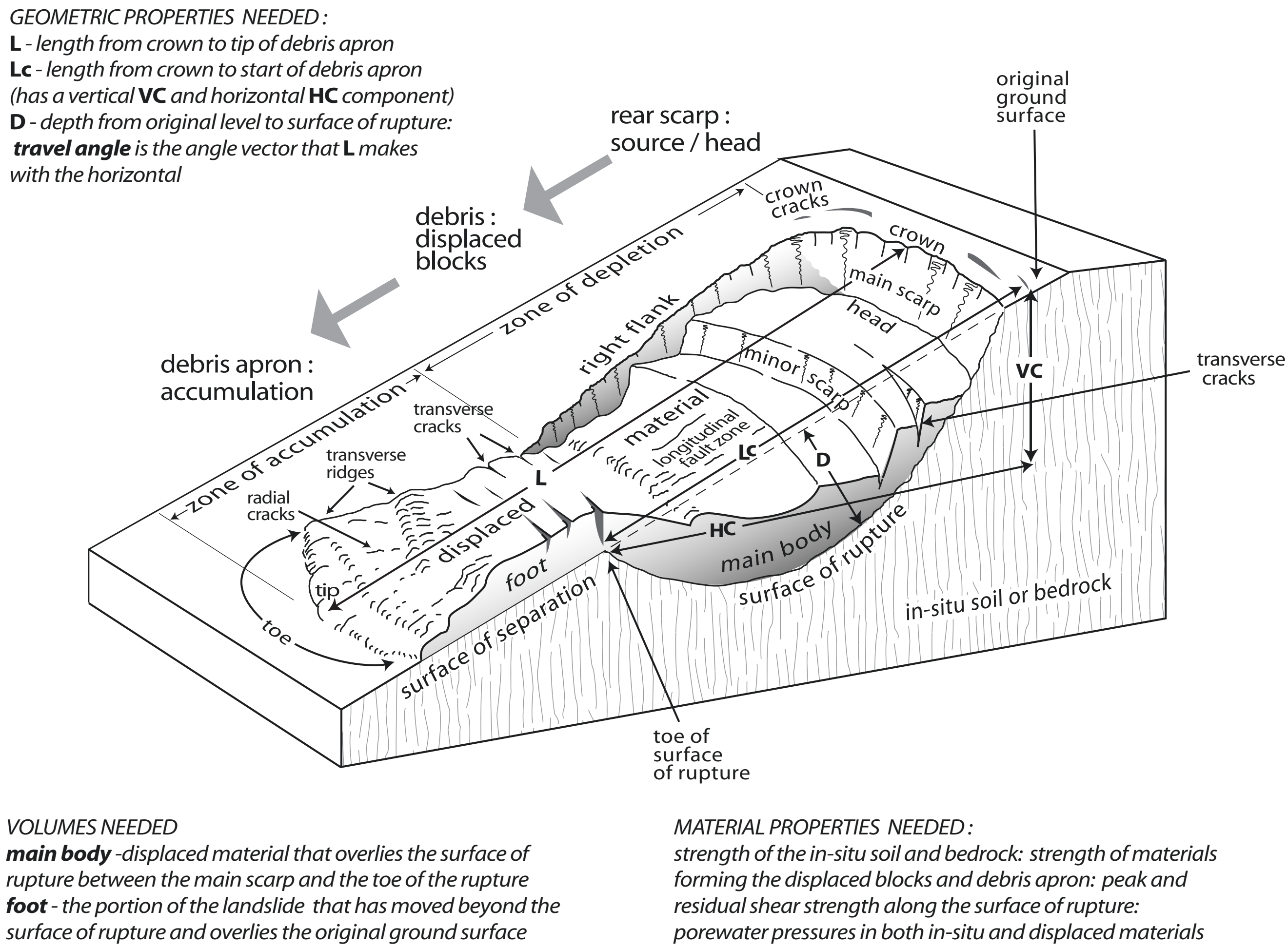
Landslides can be divided into a number of different types, and Table 1 presents the subdivision proposed by Hungr et al. (2014). Based on a similar landslide subdivision a useful *aide memoire* for the identification of the various forms of landslide was provided by Dikau et al. (1996). Correct classification of the landslide type is an essential requirement in any investigation to ensure that the nature of the potential risk is correctly evaluated and appropriate remedial measures are considered.

Landslide Cause

There are many factors that cause a landslide and these are listed in Table 2. These causes can be considered preparatory factors that either increase the stress on a slope or reduce the strength of the slope-forming materials. However, at some point a threshold is crossed when one of the preparatory factors actually triggers a failure. Understanding the cause of future or existing landslides is fundamental to the assessment of the spatial extent of the landslide hazard and an evaluation of the potential risk landslides may represent to any proposed construction development.

Engineering Geological Considerations

Depending on the terrain, landslides can represent a significant hazard to new or existing infrastructure development and should always be considered in any site investigation. The hazard from landslides can come from failures that already



Mass Movement, Fig. 1 The components of a landslide

exist in the natural landscape (relict landslides) or they can be first time failures that occur in either natural or artificial ground created by human activity including engineering and mining. In order to asses if there is any potential risk to a development from preexisting or first-times failures in the natural ground it will be necessary to undertake a landslide hazard and risk assessment, as described by Lee and Jones (2014). As part of these studies secondary hazards, such as tsunamis, need to be part of the overall risk assessment (Bornhold and Thomson 2012).

Should a potential landslide risk be identified there are three ways to deal with it: avoid the area of potential instability in future infrastructure developments through effective planning; stabilize any landslides that might affect new or existing infrastructure; or, if the level of risk is acceptable for example where there are very slow moving landslides, it can be possible to live with the potential for failures. For landslides that occur in artificial ground, notably earthworks or mine tailings, then it is necessary to stabilize them or preferably to ensure the design of the structure is correct in the first place to ensure no failure occurs.

Bromhead et al. (2012) divided the methods of stabilizing slopes into:

(a) Earthworks – excavation of the head of the slide; complete or partial replacement of the failed mass; placement of fill at the toe; overall slope angle reduction
(b) Drainage – surface water drainage; deep drainage; drainage to remove ephemeral water pressures; planting trees and vegetation.
(c) Passive engineering systems – shear keys or buttresses; piles; dowels; soil nails; walls; isolated piers. These become increasingly effective as slow ground movements generate reaction forces that ultimately lead to stabilization.
(d) Active engineering systems – ground anchors. In this case, the resistance load is put into the ground at the outset and it does not require landslide movement to generate a reaction.
(e) Geotechnical processes – these are methods that increase the shear resistance of the soil normally by altering the physics or chemistry of the soil within the landslide. For example, it might be possible to inject grout into the failed mass.

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Mass Movement, Table 1 Landslide classification (Hungr et al. 2014)

Falls and topples
1. *Rock/ice fall*: detachment, fall, rolling, and bouncing of rock or ice fragments. May occur singly or in clusters; little dynamic interaction between the most mobile moving fragments, which interact mainly with the substrate. Fragment deformation is unimportant although fragments can break during impacts. These types of failures are usually of limited volume.
2. *Boulder/debris/silt fall*: detachment, fall, rolling, and bouncing of soil fragments such as large clasts of soil deposits or blocks of cohesive (cemented or unsaturated) soil. The mechanism of propagation is similar to rock fall, although impacts may be strongly reduced by the weakness of the moving particles.
3. *Rock block topple*: forward rotation and overturning of rock columns or plates (one or many), separated by steeply dipping joints. The rock is relatively massive and rotation occurs on well-defined basal discontinuities. Movement may begin slowly, but the last stage of failure can be extremely rapid. This type of failure occurs at all scales.
4. *Rock flexural topple*: bending and forward rotation of a rock mass characterized by very closely spaced, steeply dipping joints or schistose partings, striking perpendicular to the fall line of the slope. The rock is relatively weak and fissile. There are no well-defined basal joints so that the rotation of the strata must be facilitated by bending. The movement is generally slow and tends to self-stabilize. However, secondary rotational sliding may develop in the hinge zone of the topple. These failures occur on a large scale.
5. *Gravel/sand/silt block topple*: block toppling of columns of cohesive (cemented) soil separated by vertical joints.
Slides in rock
6. *Rock rotational slide (rock slump)*: sliding of a mass of weak rock on a cylindrical or other rotational rupture surface, which is not structurally controlled. The morphology is characterized by a prominent main scarp, a characteristic back-tilted bench at the head, and limited internal deformation. Movement is usually slow to moderately slow.
7. *Rock planar slide (block slide)*: sliding of a mass of rock on a planar rupture surface. The surface may be stepped forward. There is little or no internal deformation. The slide head may be separated from the stable rock along a deep, vertical tension crack. Movement is usually extremely rapid.
8. *Rock wedge slide*: sliding of a mass of rock on a rupture surface formed of two planes with a down slope-oriented intersection. There is no internal deformation and movement is usually extremely rapid.
9. *Rock compound slide*: sliding of a mass of rock on rupture surface consisting of several joints or a surface of uneven curvature, so that motion is kinematically possible only if accompanied by significant internal distortion of the moving mass. Horst-and-graben features at the head and many secondary shear surfaces are typically associated with this type of failure, and movement range between slow and rapid.
10. *Rock irregular slide (rock collapse)*: sliding of a rock mass on an irregular rupture surface consisting of a number of randomly oriented joints, separated by segments of intact rock (rock bridges). These failures occur in strong rocks with a non-intact systematic structure. The failure mechanism is complex and often difficult to describe. The failure may include elements of toppling. These failures are often sudden and movement is extremely rapid.
Movements in soil
11. *Clay/silt rotational slide (soil slump)*: sliding of a mass of homogenous and usually cohesive soil on a rotational rupture surface. There is normally little internal deformation of the sliding mass. Forms a prominent main scarp and back-tilted landslide head. Movement is normally slow to rapid but may be extremely rapid in sensitive or collapsing soils.
12. *Clay/silt planar slide*: sliding of a block of cohesive soil on an inclined planar rupture surface formed by a weak layer that is often presheared. The head of the slide mass is separated from the stable soil along a deep tension crack and movement may be slow or rapid.
13. *Gravel/sand/debris slide*: sliding of a mass of granular material on a shallow, planar surface parallel with the ground. Usually the sliding mass is a veneer of colluvium, weathered soil, or pyroclastic deposits (a lahar) sliding over a stronger substrate. Many debris slides become flow-like after moving a short distance and transform into extremely rapid debris avalanches.
14. *Clay/silt compound slide*: sliding of a mass of soil on a rupture surface consisting of several planes or a surface of uneven curvature, so that motion is kinematically possible only if accompanied by significant internal distortion of the sliding mass. Horst-and-graben features at the head and many secondary shear surfaces are typically associated with this type of failure. The basal segment of the rupture surface often follows a weak horizon in the soil stratigraphy.
Spreading
15. *Rock slope spread*: near-horizontal stretching (elongation) of a mass of coherent blocks of rock as a result of intensive deformation of an underlying weak material or by multiple retrogressive sliding controlled by a weak basal surface. There is usually only limited total displacement in rock slope as it spreads and movement is normally slow.
16. *Sand/silt liquefaction spread*: extremely rapid lateral spreading of a series of soil blocks floating on a layer of saturated (loose) granular soil liquefied by earthquake shaking or spontaneous liquefaction.
17. *Sensitive clay spread*: extremely rapid lateral spreading of a series of coherent clay blocks floating on a layer of remolded sensitive clay.
Flow-like landslides
18. *Rock/ice avalanche*: Extremely rapid, massive, flow-like motion of fragmented rock that develops from a large rock slide or rock fall.
19. *Dry (or nonliquefied) sand/silt/gravel/debris flow*: slow or rapid flow-like movement of loose dry, moist or subaqueous, sorted or unsorted granular material that moves without the development of excess pore-water pressure.
20. *Sand/silt/debris flowslide*: very to extremely rapid flow of sorted or unsorted saturated granular material on moderate slopes involving the development of excess pore-water pressure or liquefaction of material originating from the landslide source. The material may range from loose sand to loose debris (including fill or mine waste), loess, and silt. The failure usually originates as a multiple retrogressive failure and can occur both subaerially or underwater.

(*continued*)

Mass Movement, Table 1 (continued)

21. *Sensitive clay flowslide*: very rapid to extremely rapid flow of liquefied sensitive clay that is a result of remolding during a multiple retrogressive slide failure at, or close to, the original depositional water content.
22. *Debris flow*: very to extremely rapid surging flow of saturated debris in a steep channel. Extensive entrainment of debris and water in the failure flow path.
23. *Mudflow*: very to extremely rapid surging flow of saturated plastic soil in a steep channel involving significantly greater water content relative to the source material. Extensive entrainment of debris and water in the failure flow path (Plasticity Index of material >5%).
24. *Debris flood*: very rapid flow of water, heavily discharged with debris, in a steep channel. The peak discharge of debris flood is comparable to a water flood.
25. *Debris avalanche*: very to extremely rapid shallow flow of partially or fully saturated debris on a steep slope without confinement to an established channel. This type of failure occurs at all scales.
26. *Earthflow*: rapid or slow intermittent flow-like movement of plastic, clayey soil, facilitated by a combination of sliding along multiple discrete shear surfaces and internal shear strains. Long period of relative dormancy alternate with more rapid surges.
27. *Peat flow*: rapid flow of liquefied peat caused by an undrained failure.
Slope deformation
28. *Mountain slope deformation*: large-scale gravitational deformation of steep, high mountain slopes. Failure identifiable by scarps, benches, cracks, trenches, bulges, but it lacks a fully defined rupture surface. Movement is extremely slow or unmeasurable.
29. *Rock slope deformation*: deep-seated slow to extremely slow deformation of valley or hill slopes. Failure form identifiable by sagging of slope crests and development of cracks or faults without a well-defined rupture surface.
30. *Soil slope deformation*: deep-seated, slow to extremely slow deformation of valley or hillslopes usually formed of cohesive soils. Often found in permafrost slopes with a high ice content.
31. *Soil creep*: extremely slow movement of surficial soil layers (typically <1 m deep) on a slope. Movement is a function of climate-driven cyclical volume changes resulting from wetting and drying of the soil and frost heave. No development of a rupture surface.
32. *Solifluction*: very slow shallow soil creep involving the active layer (i.e., seasonally frozen) in Alpine or polar permafrost. Failures often have a characteristic lobate form.

The above techniques can be employed on soil, debris, and rock slopes. However, all these techniques require a full understanding of the nature and properties of the landslides if the correct stabilization solution is to be designed and constructed (Simons et al. 2001).

The mass movement form "soil creep" is rarely of significance to major engineering structures, but it is a key component of soil erosion processes that can limit the long-term sustainable use of land for agriculture. Soil creep, therefore, has long history of investigation in soil conservation and pedological studies. Soil creep can be subject to intervention involving engineering geologists should excessive soil erosion become a concern. On slopes the typical engineering works that might be required to reduce the slope runoff that causes erosion include construction of: soil terraces; benches; small (<1 m high) bunds; and stone revetments. In gullies and small channels, there could be a requirement to build small loose-rock check dams to collect sediment and reduce peak discharge (Morgan 2004).

Snow Avalanches

Snow avalanches only occur in environments where sufficient snow can accumulate on a slope, thus they are limited to climates and altitudes where temperatures are low enough for significant snow to fall. The main differences between landslides and snow avalanche are: the predominance of snow in an avalanche movement; avalanches are usually much more seasonal in occurrence; the hazard and risk posed by avalanches vary enormously throughout the year depending on the local meteorological conditions; and the hazard can disappear for many months in regions where the snow melts from slopes in the summer.

Snow avalanches only occur on slopes between 20 and 60°, as slopes <20° are generally too shallow to generate movement and slopes >60° are too steep for sufficient depths of snow to accumulate. Snow avalanches occur when the shear stress on the snow exceeds its shear strength. The shear strength of the snowpack on a slope is a function of its density and temperature and unlike the soils involved in landslides, snow layers can undergo rapid and large changes in density and strength *in situ*. The density and shear strength of the lower layers of the snowpack will increase as a result of loading by additional snowfalls, whereas they can decrease as temperature rises. Increasing temperature above freezing will produce water that can lead to excess pore-water pressures developing within the snowpack, increases in the amount of liquid water present, and reduction of the effective shear strength.

The main risk posed by snow avalanches is to mountain villages, communication links between settlements in mountains, and increasingly the global skiing infrastructure. All countries that have significant development in mountain regions have their own snow avalanche specialist agencies that carry out research into the nature of avalanches, the threat they pose, and how to mitigate the risks (for Canada see Campbell et al. 2007; International Snow Science Workshop 2014).

Mass Movement, Table 2 The processes that cause landslides (Turner and Schuster 1996)

External process(es)	Causal effects	Description of typical changes	Examples of specific changes on slope
Weathering: physical, chemical, and biological	Changes in physical and chemical properties; horizonation; changes in regolith thickness	Changes in grading; cation exchange; cementation; formation of weak discontinuities or hard bands; increased depth of low strength materials	Changes in: density, strength, permeability; stress, pore, and cleft water pressure
Erosion of material from face or base of slope by fluvial, glacial, and/or coastal processes	Changes in slope geometry; unloading	Alterations to: relief; slope height, length, angle, and aspect	Changes in stress, permeability, and strength
Ground subsidence	Undermining	Mechanical eluviation of fines; solution; loss of cement; leaching seepage erosion; backsapping; piping	Loss of support; consolidation; changes in porewater pressure; loss of strength
Deposition of material to face or top of slope by fluvial, glacial, or mass movement processes	Loading; long term (drained) or short term (undrained)	Alterations to: relief; slope height, length, angle, and aspect	Changes in stress, permeability, strength, loading, and porewater pressure
Seismic activity and general shocks and vibrations	Rapid and repeated vertical and horizontal displacements	Disturbance to intergranular bonds; transient high porewater pressures; materials subject to transient and repeated periods of compression and tension	Changes in stress; loss of strength; high porewater pressures; potential for liquefaction
Air fall of loess or tephra	Mantling slopes with fines; adding fines to existing soils	New slope created with well-defined discontinuity boundary	Changes in stress; strength; water content and water pressure
Water regime change	Rising or falling groundwater; development of perched water tables; saturation of surface; flooding	Piping, floods, lake bursts; "wet" years; intense precipitation; snow and ice melt; rapid drawdown	Excess porewater pressures; changes in bulk density; reduction in effective shear strength
Complex follow-on or runout processes after initial failure	Liquefaction; remolding; fluidization; "acoustic grain flow"	Long runout landslides; low values for ratio of initial failure volume to total failure volume; low angles of reach; low breadth to length ratios	Changes in effective shear strength, water distribution, bulk density, and rheological characteristics
Human interference	Excavation at toe of slope Top loading of slopes Flooding (e.g., leaking services; reservoir construction)	Same as natural erosion Same as natural deposition Same as natural water regime change	Same as natural erosion Same as natural deposition Same as natural water regime change

Describing a Snow Avalanche

Some of the same terminology used for a landslide shown in Fig. 1 can be helpful in describing a snow avalanche. However, there are three distinct sections to an avalanche that can be recognized: the starting zone where the movement is initiated; the track or path that the avalanche follows; and the runout zone where the avalanche slows and stops.

Types of Snow Avalanche and Their Movement

There are two main types of snow avalanche:

(a) Loose snow avalanches – these occur in cohesionless snow and resemble dry sand flows (Type 19 in Table 1). These are relatively shallow failures and take the overall form of an inverted "V."
(b) Slab avalanches – these resemble soil planar slides (Type 12 in Table 1) and occur when a strongly cohesive layers of snow fails along a weak underlying layer. These are much larger failures with the failure surface up to 10 m deep and the initial failure slab up to 10,000 m^2. These are a very dangerous form of failure as they can bring down 100 times the initial failure volume of snow (Smith 2013).

Once the snow avalanche has started moving three types of motion have been identified:

(a) Powder avalanches (Fig. 2) – these take the form of an aerosol of fine, diffused snow that behaves like a dense gas. These are a very hazardous form of movement that at the leading edge can have speeds up to 70 m/s. The movement tends to keep to well-defined deep channels, but their passage is not affected by obstacles in their path. As an example of the scale of the phenomena the leading powder wave of the avalanche that struck the village of Galtür in Austria in 1999 was 100 m high and this event killed 31 people as well as demolishing seven modern buildings.

Mass Movement, Fig. 2 A typical powder avalanche in Wyoming, USA (Photo credit Wikipedia Commons)

(b) Dry flowing avalanches – these are formed of cohesionless snow grains <0.2 m in diameter that follow well-defined channels. On the ground the speed of a dry snow can be up to 60 m/s, but if they become airborne they can reach speeds of 120 m/s.

(c) Wet flowing avalanches – in terms of movement these failures resemble a debris flow (Type 22 in Table 1) and are composed of dense wet snow formed of rounded particles 0.1 m–2 m + in diameter. These flows tend to keep to stream channels with speeds of 5–30 m/s and cause considerable erosion along the avalanche track.

Causes of Snow Avalanches

Snow avalanches are caused by: heavy snowfalls increasing the load on the snowpack; rain or thaw increasing the water content and reducing the strength of the snowpack; or a transient increase in dynamic loading that can either be natural (earthquakes) or artificial (skiers crossing a marginally stable area or use of explosives).

Engineering Geological Considerations

As with landslide studies the key aspect of snow avalanche investigations is to assess the hazard. As snow avalanches tend to occur in the same or similar sites in an area it is possible to identify the likely avalanche tracks and either avoid these if planning new developments or employ a range of mitigation measures to protect existing infrastructure, including ski runs. Many of the same techniques used to assess landslide hazard (Lee and Jones 2014) can be employed in avalanche hazard investigations. However, it should serve as a warning that the village of Galtür in Austria affected by the 1999 disaster had previously been considered to lie in a low hazard zone. Once the hazard has been identified, the decision can be made on whether or not there is a risk that needs mitigation. Mitigation measures can be divided into two main types (Smith 2013):

(a) Artificial release – the use of explosives to initiate an avalanche at a controlled time. This allows for snow clearance measures to be in place, people can be kept away from the areas affected, and the failure can be set off before the snowpack gets too large that might otherwise result in an uncontrolled avalanche.

(b) Defense structures – this is the most common way of dealing with avalanches and where engineering design has an important role to play. There are four main types of structures: retention, designed to trap and retain snow on a slope thus preventing initiation or escalation of small failures; redistribution structures designed to prevent snow accumulation through drifting; deflectors and retarding devices that are placed in the avalanche track and runout zone; and direct protection such as avalanche sheds and galleries typically built over railways and roads.

Conclusions

The expression "mass movement" has not been widely adopted in engineering geology with the concentration being on the subcategory of "landslides." Nevertheless, mass movement is a better way to describe the range of failures that can occur on slopes and which may require an engineering geological input. It is appropriate to note that in a study of the fjords of northwestern Iceland Decaulne (2007) recognized it was the combined hazard created by snow-avalanches and debris-flows that required mitigation and prevention and the approach to assessing the hazards was the same.

For engineering geologists working in environments where mass movements of any type are possible it is beholden on them to ensure all the hazards are identified. The methods of investigation for all forms of mass movement are essentially the same and require the creation of an adequate ground model that will enable the nature and scale of any hazards and risks to be identified and quantified. The ground model can then be used in the design of any measures needed to mitigate the risk, whether this is to existing or proposed infrastructure development. Note that the ground model is not a simple definitive construct as it will develop and improve when new data become available during investigations and any subsequent construction. These developments of the ground model must be incorporated into any design process.

There is a wealth of literature and on-going research into the nature and causes of mass movement. Nevertheless, it is still not possible to state with any certainty where or when a slope failure will occur and what size it will be nor the extent of the runout. Although snow avalanche research has probably advanced further in establishing these issues than landslide studies the Galtür disaster in Austria in 1999 remains as a salutary lesson showing there are still many unknowns. Until these questions can be answered mass movement research must continue to ensure there are no more disasters like the 1963 Vaiont Reservoir landslide.

Cross-References

▶ Avalanche
▶ Climate Change
▶ Engineering Geomorphological Mapping
▶ Factor of Safety
▶ Glacier Environments
▶ Hazard
▶ Hazard Assessment
▶ Landslide
▶ Mountain Environments
▶ Risk Assessment
▶ Shear Strength
▶ Shear Stress
▶ Site Investigation
▶ Stabilization

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Mechanical Properties

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Definition

Physical properties that determine the behavior of ground that is under mechanical stress.

Mechanical properties of ground determine the behavior of the ground under stress in a mechanical way; for example, settlement of ground under a foundation, subsidence due to underground excavation or extraction of gases and fluids, tunnel and slope stability, and “breaking” of rock or cemented soils. Deformation is the change in volume or shape of ground under stress in which rheology and viscosity may be important. Strength commonly denotes a stress condition at which the ground fails (breaks) when a threshold in stress conditions is exceeded. Constitutive models are the aggregate term for relations that describe the chemical-physical-mechanical behavior of ground.

Ground materials are diverse and may be gases, fluids, solids (i.e., minerals, grains, and aggregates of grains or minerals) and any mixture of these and also include man-made ground, such as fills and waste dump materials. Ground is commonly differentiated between soil and rock, soil being an aggregate of loose or weakly bonded particles and rock consisting of particles cemented or locked together, giving rock a tensile strength. Soil and rock are, by some, differentiated based on a compressive strength difference with soil being weaker than 1 MPa and rock being stronger. A differentiation is made between “intact” and “discontinuous” ground, that is, ground without and with, respectively, distinct planes of mechanical weakness (discontinuities) such as faults, joints, bedding planes, fractures, schistosity. A groundmass consists of (blocks of) intact ground with discontinuities, if present.

Stress, Deviatoric Stress, Deformation, and Strain

If a normal (σ) or shear (τ) stress is applied on a body of ground, the ground will deform, that is, will be strained, respectively, and change in shape by an angle rotation. The normal stress minus the hydrostatic stress, which is the average of all normal stresses working on a body of ground, is the deviatoric stress.

Total and Effective Stress, and Gas and Fluid Pressure

A porous ground consists of a skeleton of solid particles with pores in-between. The pores may contain gases and fluids. When a load is applied on a body of ground, part of the load is taken by the skeleton resulting in stress in the skeleton, and part by the gas and fluid. The skeleton, gas and fluid will deform. Many mechanical characteristics of the ground depend on the stress between particles; therefore, normally the load is divided into “effective stress,” or the stress in the skeleton, and the stress (pressure) in the gas and fluid. In the case of pores filled by water, this is the “porewater pressure.” Effective stress together with the gas and fluid pressure is the “total stress” which equals the stress from the outside on the body of ground. In a porous ground without any gas or fluid and in a nonporous ground, the effective stress equals the total stress. Total stress may be applicable in situations where the pore gas and fluid pressure cannot or dissipates too slowly, for example, during fast loading of a low-permeable clay (the “undrained” situation). In most cases, the presence of gas is neglected under engineering conditions, but may be important in, for instance, subsidence due to gas and oil exploitation.

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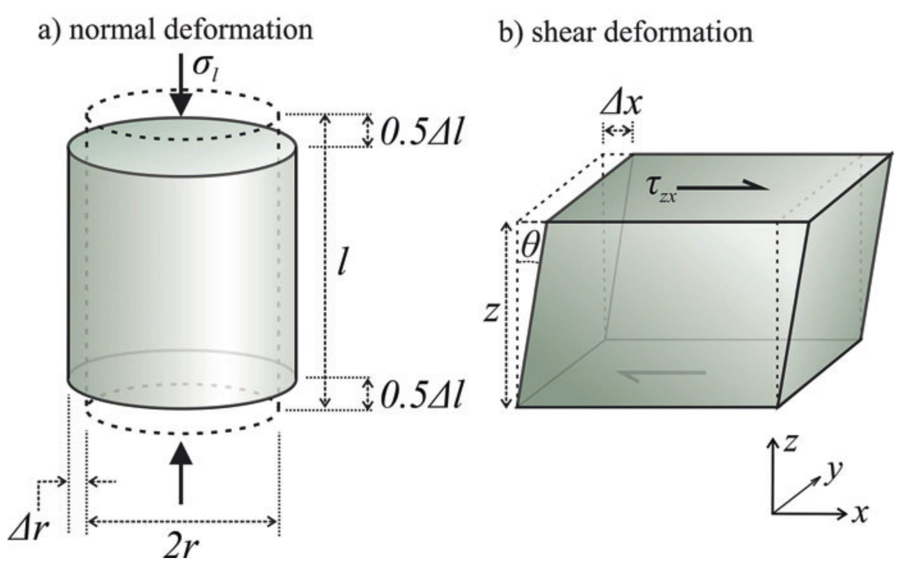
Elastic Deformation

The relation between normal and shear stress and strain for an intact, homogeneous, isotropic, and ideal-elastic body of intact ground is formulated in Eqs. 1 and 2 (Fig. 1). In elastic

Mechanical Properties, Fig. 1 Deformation, stress and strain



deformation, stress and strain are coupled properties; there is no strain without stress and vice versa. Under the influence of a normal stress (σ) in a particular direction, the material becomes shorter in that direction and wider perpendicular to the stress direction. The amount of shortening in relation to the stress is expressed by the elastic deformation or Young's modulus (E). The amount of widening in one direction related to the shortening in the other direction is expressed by the Poisson's ratio (υ).

$$E_l = \frac{\sigma_l}{\varepsilon_l} \quad \varepsilon_l = \frac{\Delta l}{l} \quad \varepsilon_r = \frac{\Delta r}{r} \quad \upsilon_{lr} = \frac{\varepsilon_r}{\varepsilon_l} \tag{1}$$

E_l = elastic Young's modulus of deformation in l direction [Pa]
υ_{lr} = Poisson's ratio of expanding in r direction due to stress in l direction
σ_l = stress in l direction [Pa]
l, r = length respectively radius of body [m]
Δ_l, Δ_r = deformation in l respectively r direction [m]
$\varepsilon_l, \varepsilon_r$ = strain in l respectively r direction

Shear stress (τ) causes a deformation in shape governed by the shear modulus (or modulus of rigidity). The shear modulus is defined as follows:

$$G_{zx} = \frac{\tau_{zx}}{\gamma_{zx}} \quad \gamma_{zx} = \frac{\Delta x}{z} = \tan\theta \tag{2}$$

G_{zx} = shear modulus in z direction due to shear stress in x direction [Pa]
τ_{zx} = shear stress in x direction on plane with normal in z direction [Pa]
γ_{zx} = shear strain
l, r = length respectively radius of body [m]
Δx = shear deformation in x direction [m]
z = height of body [m]

For an intact, homogeneous, isotropic, and ideal-elastic solid, the Young's modulus, Poisson's ratio, and shear modulus are related following:

$$2\,G(1+\nu) = E \tag{3}$$

Deformation characteristics are often expressed in terms of Lamé parameters (also named "constants" or "coefficients") λ and μ:

$$\lambda = \frac{\nu E}{(1+\nu)(1-2\nu)} \quad \mu = G = \frac{E}{2(1+\nu)} \tag{4}$$

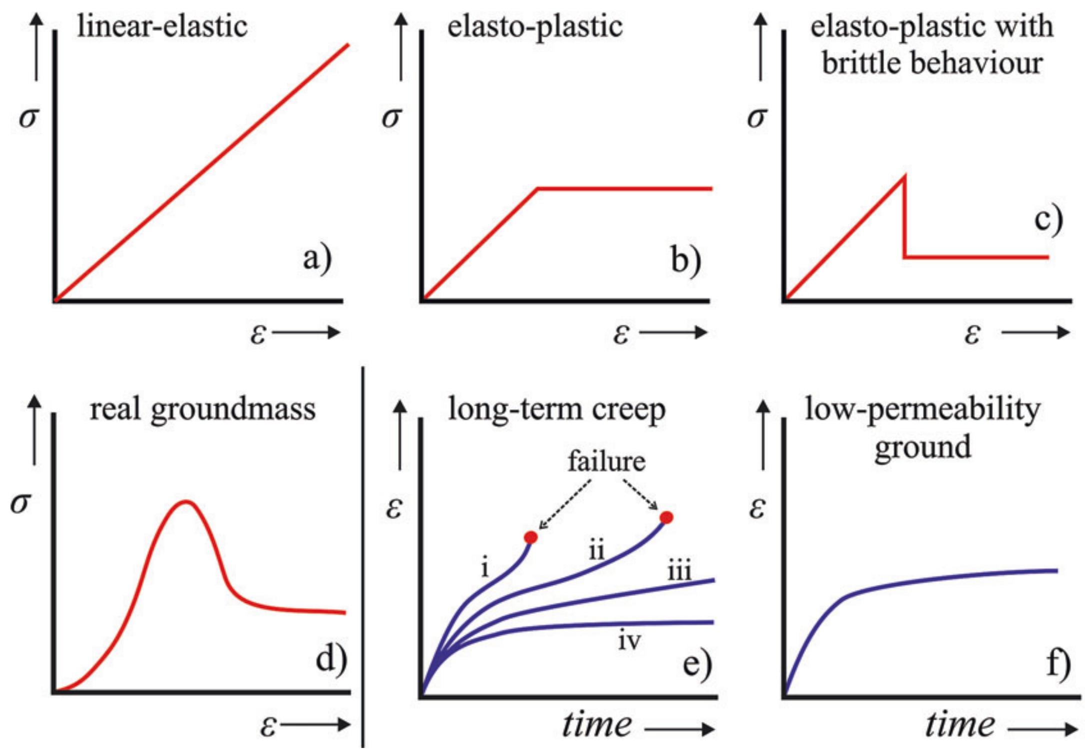
Moduli and Poisson's ratio are anisotropic for most intact ground and most groundmasses, that is, the values vary with direction.

Non-elastic Intact Ground and Groundmass

Most intact ground and virtually all groundmasses do not deform in an ideally elastic manner, and the normal and shear deformation moduli are not elastic or only partially elastic (Fig. 2). Ground may deform as a combination of plastic, elastic, and brittleness, properties which may also depend on factors such as time, temperature, confining stress, presence of gases and fluids, and nuclear radiation. Brittleness means that the intact ground fails, that is, breaks or fractures. Figure 2a shows a linear–elastic deformation; on release of the stress, the sample will return to its original volume and shape. In Fig. 2b the material behaves elasto-plastically; the first part is elastic deformation, whereas in the second (plastic) part the deformation increases under constant stress; the ground does not return to its original volume and shape when the stress is released; Fig. 2c is similar to Fig. 2b, but at the boundary between elastic and plastic, the material fails (brittleness). Figure 2d shows the deformation behavior of most real ground which is a combination of elastic, plastic, and brittle deformation. Intact ground, in particular rock, may deform more-or-less elastically for stresses up to 50–80% of the unconfined compressive strength (UCS) value (see below). Groundmasses are seldom intact but mostly contain discontinuities. Discontinuous groundmasses virtually never behave elastically, but mostly deform permanently, thus plastically, with shear displacements along discontinuities. Under higher confining pressure or temperature, most materials show a more gradual deformation without brittleness, such materials are said to undergo ductile deformation. As most intact ground and groundmasses do not or not fully behave elastically, many engineers prefer the letter "D" to denote the deformation modulus rather than "E." Table 1 lists values for D-moduli and Poisson's ratios for various intact grounds and groundmasses. Note the enormous variation in values even for geologicallly or lithologically similar ground.

Consolidation and Compaction

Under influence of stress, and over time, ground changes; gases and fluids may be expelled, grains and particles re-arrange, and the constituents of the ground may change structurally and chemically. Generally, this results in a decrease of volume. The expressions "consolidation" and "compaction" are used interchangeably, but consolidation is more often used for soil materials with low permeability and compaction for permeable granular soil, for rock, and for soil becoming rock. In soil mechanics, it is customary to differentiate between primary and secondary consolidation. The first is mainly related to expelling gases and fluids from pores in the ground, whereas the second is more related to



Mechanical Properties, Fig. 2 Various stress–strain (deformation) behavior; σ = stress; ε = strain; (**e**) Modified after Dusseault and Fordham (1993). Note that in figures (**a–d**) the horizontal axis is strain while in (**e**) and (**f**) it is time

re-arrangement of grains and changes in material, the latter particularly in organic soils, such as peat (Fig. 2f). Mathematically this is characterized by the "coefficient of consolidation" (c_v – a smaller value indicates that more time is required for consolidation) for short-term primary deformation and by the "coefficient of secondary consolidation" for long-term deformation (C_α – a larger value indicates more consolidation in a given time span) (Bodó and Jones 2013). Short and long-term deformation under near-surface conditions may be centimeters to many decimeters per year per meter thickness of ground, the latter especially for ground such as peat and household waste.

Compaction of granular soil-type material under stress involves re-arrangement of the grains resulting in a smaller volume. Compaction of rock and soil becoming rock involves expelling of gases and fluids, re-arrangement, and structural and chemical changes of particles, grains, and minerals. Compaction rates for ground under near-surface conditions are very variable ranging from centimeters per year to millimeters per millions of years or longer per meter thickness of ground.

Time Effects, Creep, and Temperature

Deformation of intact ground and groundmasses is time dependent. There are several reasons that cause this dependency, which can be divided into four phases with increasing timespan: (1) No strain can be instant after applying a stress. Instant strain would require an infinite velocity of the material particles, which is impossible. The particles have a certain mass, and displacement requires a certain timespan. If stress is applied, shock waves of stress-strain will travel through the ground. It will take some time, albeit in engineering terms very little, mostly in the order of microseconds to minutes, before stress and strain are again in equilibrium throughout the ground. Under slow application of stress, the shock wave effect may be minimal, but it will still take time before equilibrium between stress and strain is established. (2) All ground shows some (sometimes limited) effects of longer-term deformation. This is called "*creep*." Figure 2e shows various options for strain versus time for long-term deformation under constant stress. Some materials deform with an increasing deformation rate leading to rapid failure (curve i in Fig. 2e). Others deform very slowly, and deformation rates may attenuate (curve iv), be steady state (curve iii), or re-accelerate after a long steady state period resulting in failure (curve ii). Creep is responsible for the delay in, for example, collapse of underground excavations. When a groundmass is loaded with a new stress environment due to excavation, the stress levels may not exceed the strength of the groundmass. Hence, the excavation will not fail. However, when stresses are near to the maximum stresses the

Mechanical Properties, Table 1 Example of deformation values

Material		Deformation modulus (D) (GPa)[1]	Poisson's ratio (ν)[1]
Soil			
Doha marine loose sand[2,a]		0.02	
Sand (Amsterdam)[b]		0.035–0.04	0.2
Residual soil & fill[b,c]		0.01–0.04	0.15
London clay (drained; depending on depth and direction)[3,d]		0.007–0.2	0.125
Aeschertunnel glacial till[4,e]		0.08	0.2
Clay (Amsterdam)[b]		0.01	0.15
Peat (Amsterdam)[b]		0.002	0.15
Frozen dense sand (artificially frozen, T ~ −10 °C) (short/long-term)[5,f]		0.75/0.33	0.3[g]
Frozen stiff clay (artificially frozen, T ~ −10 °C) (short/long-term)[5,f]		0.3/0.12	0.006–0.13[h]
Ice (natural fresh water ice; T ~ −5 °C)[5,i]		10	0.33
Man-made material			
Concrete (regular commercial, Portland cement, 28 days cured)[6,j]		27–35	0.2
Iron/steel[k]		200	0.3
Intact rock			
Hawkesbury sandstone[l]		6–14	0.15
Falset carboniferous sandstone[m]		35–60	0.1–0.2
Vinalmont limestone[n]		70	0.31
Sibbe limestone[n]		1.2	0.25
Königshain granite[7,o]	Slightly weathered	50	
	Moderately weathered	25	
	Highly weathered	15	
Äspö slightly fractured diorite and granite[p]		69–79	0.21–0.28
Basalt[8,q]		78	0.25
Gorleben salts (uniaxial/triaxial short-term laboratory tests; average of different formations)[r]		25/33	0.25–0.32
Rock mass			
Hawkesbury sandstone[9,l]		0.05–2.5	
Sheared flysch[9,v]		0.433	
Marly shale (standard zone)[w]		1.8	
Mu-cha tunnel fault (sheared sandstone and shale in clay matrix)[9,x]		0.2	0.3
Sydney-Gunnedah Basin coal[9,y]		2.5	0.24
Äspö slightly fractured diorite and granite[9,p]		55	0.26
Basalt[8,9,q]		10–40	0.3
Gorleben salts (short-term dilatometer tests; average of different formations)[10,r]		19	

Notes: Values reported are for normal (near-) surface engineering conditions. [1]"*D*" is the deformation modulus and "ν" the Poisson's ratio; values for 50% of the failure strength if reported. [2]"Loose to dense" refer to the packing of the particles. [3]Undrained and drained refer to the dissipation of pore gas and fluid pressures during loading; generally undrained applies to fast loading situations and drained for slow. [4]Glacier deposit: clayey sand and silt, with gravel and isolated boulders. [5]Values indicative only; strongly dependent on test conditions, deformation rate, compaction, temperature, structure, and quantity. [6]Indicative only depending on type of concrete, aggregate, and cement type. [7]Weathering description follow BS 5930: 1999 (1999). [8]Summary literature typical values based on different basalts. [9]Properties determined by rock mass classification and/or back analyses from tunnel construction. [10]Dilatometer tests. Data from: [a]Chen (2010), [b]Bosch and Broere (2009), [c]Chan and Stone (2005), [d]Karakus and Fowell (2005), [e]Coulter and Martin (2004), [f]Jessberger et al. (2003), [g]Kirsch and Richter (2009), [h]Lee et al. (2002), [i]Schulson (1999), [j]Bamforth et al. (2008), [k]Ashby and Jones (2012), [l]Pells (2004), [m]Kouokam (1993), [n]Swart (1987), [o]Thuro and Scholz (2004), [p]Andersson (2010), [q]Schultz (1995), [r]Bräuer et al. (2011), [v]Marinos et al. (2009), [w]Alejano et al. (2008), [x]Yu (1998), [y]Sainsbury (2008)

groundmass can sustain, small microcracks may develop with time. The number of cracks will increase over time, and grow together, until, after some time, the groundmass fails. Similarly, in slope stability, loading a discontinuity with a shear stress may cause asperities on the plane to be stressed such that no immediate failure occurs, but with time microcracks form in the asperities which break after some time. The length of the period between stress loading and failure may range from seconds to many years. (3) All grounds show a very long-term creep effect that is often also strongly dependent on temperature and confining stress. The ground deforms and after some time the ground may fail even if the ground is stressed well below the maximum sustainable stresses. The mechanisms for this effect are largely unknown, but it is thought that re-crystallization under stress and weathering may play a role. Long-term creep is likely responsible for some collapses of excavations after long-time spans, sometimes after many hundreds of years. (4) On geological timespans, ground may show viscous flow.

Volumetric (Hydrostatic) Deformation

A body of ground will first compress due to setting and closure of fissures (small cracks) if it is under equal stress from all directions (Fig. 3), then a second phase of elastic deformation occurs for the particle skeleton. This is followed by a collapse of the pore structure in porous ground which effectively also destroys the skeleton structure. With continuing increase in stress, the particles in the ground will interlock and the amount of volume change with stress increase strongly reduces, resulting in a strong increase of the volumetric deformation modulus. Further increase in stress will cause the structures of particles and minerals to be destroyed and, at even higher stress levels, molecular and atomic structural changes. This high level of stress does not normally occur in engineering but may occur during underground nuclear tests. The bulk modulus (K) of volumetric deformation of ground is defined as:

$$\begin{gathered}\text{bulk modulus [Pa]} = K = \frac{\sigma_{\text{hydrostatic}}}{\Delta V/V} \\ \sigma_{\text{hydrostatic}} = \text{hydrostatic stress } [Pa] \\ \Delta V = \text{change in volume } [m^3] \quad V = \text{volume } [m^3]\end{gathered} \tag{5}$$

For an intact, homogeneous, isotropic, and ideal-elastic solid, the bulk modulus is related to Young's and shear moduli as follows:

$$K = \frac{EG}{3(3G - E)} \tag{6}$$

During compression, gases and fluids are expelled from the ground if possible. If not, part of the stress is taken up by the gases and fluids which will increase the overall deformation moduli and may prevent pore collapse. Note that a ground under equal stress is not really failing in the sense that a failure plane and a strength can be defined as discussed below.

Unconfined and Confined Compressive Strength

The compressive strength is the compressive stress at failure on a sample under a deviatoric normal compressive stress (σ_1). Compressive strength of ground material can be tested under different stress configurations. Depending on the type of test undertaken the compressive strength is denoted as Unconfined Compressive Strength (*UCS*), (normal) triaxial compressive strength, or true-triaxial compressive strength. Note that the tests are measuring the compressive strength but that the failure mode is actually due to stresses in the sample exceeding the shear or tensile (the latter sometimes referred to as splitting or bending) strength. Normally also the change in dimensions of the sample are measured during the test to obtain deformation characteristics. Triaxial and true-triaxial tests are mostly completed with pore pressure transducers allowing measurement of gas and fluid pressure in the sample during the test.

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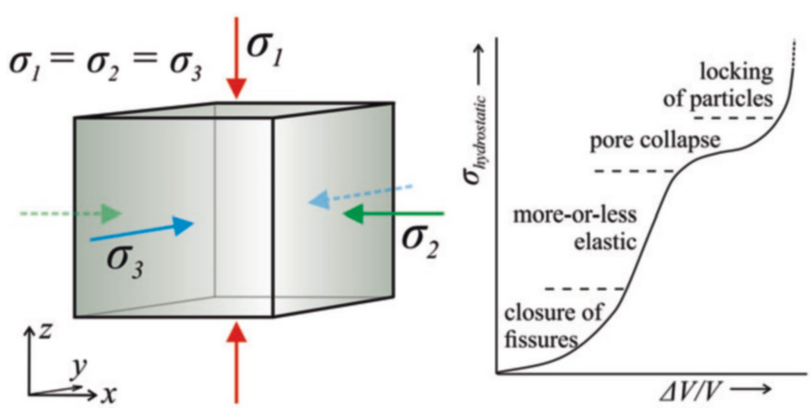
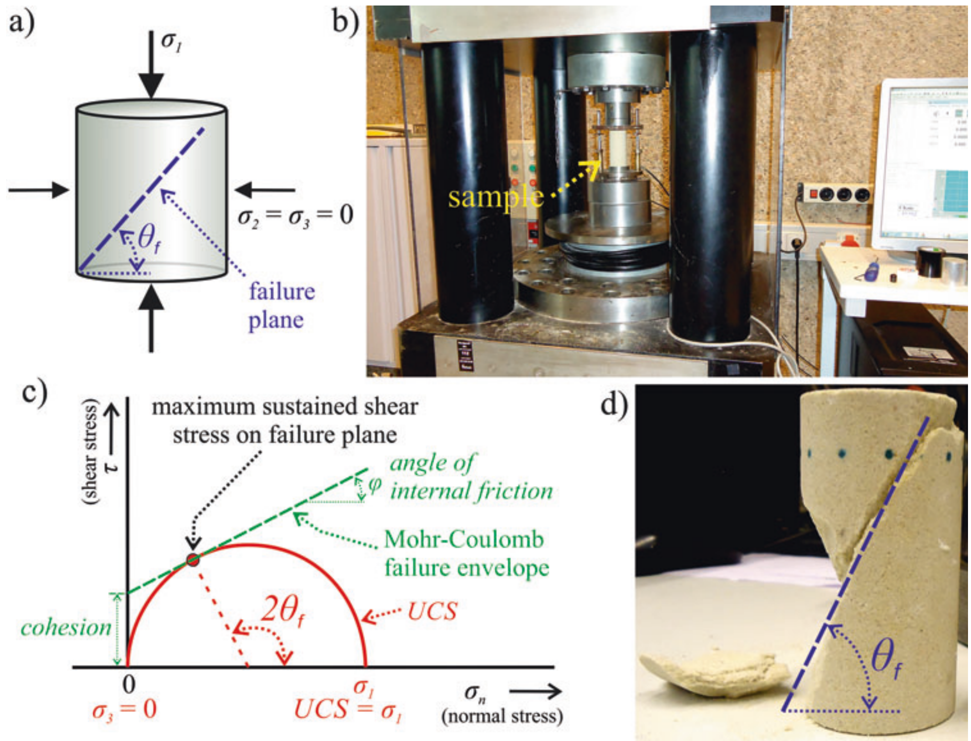
Unconfined Compressive Strength (*UCS*)

The Unconfined Compressive Strength (*UCS*) (also uniaxial strength) is the compressive stress (σ_1) measured at failure on a sample of ground under the condition that the confining pressure is zero ($\sigma_2 = \sigma_3 = 0$) (Fig. 4). The test is normally done on a cylinder sample, but can also be done on a cubic sample. Soil samples should have some form of attraction or gluing effect between particles; otherwise, the sample falls apart under its own weight. Alternatively, a sleeve or jacket of, for example, rubber is used to maintain sample integrity. The approximate failure plane is indicated in Fig. 4d; the



Mechanical Properties, Fig. 3 Volumetric stress-strain of porous ground



Mechanical Properties, Fig. 4 Unconfined Compressive Strength (UCS) test on a cylinder sample; (**a**) sample and stress configuration; (**b**) test equipment; (**c**) stress configuration in a Mohr-circle diagram; (**d**) test result (Photos courtesy W. Verwaal, TU Delft, 2017)

angle may be slightly different from the failure plane angle indicated in Fig. 4c due to changes in failure plane area during the test. Figure 4c shows the failure state in the Mohr-circle diagram with the Mohr-Coulomb failure envelope. Examples of *UCS* values are listed in Table 2. Test standards and procedures are given in ASTM D7012-10 (2008) and Ulusay and Hudson (2007).

Triaxial Compressive Strength

In a triaxial test, the compressive stress (σ_1) is measured at the point failure of a sample that is under confining pressure (Fig. 5). The test is normally done on a cylinder sample in a sleeve of a foil of rubber (for soil) or metal, such as, copper or steel (for rock). The sample with sleeve is positioned in a pressure cell in which water or oil gives the confining pressure on the sleeve (Fig. 5b, c, e). The test is denoted a "normal triaxial test" but more commonly just "triaxial test". Example values in terms of the Mohr-Coulomb failure envelope parameters are listed in Table 2. Test standards are given in ASTM D7012-10 (2008) and Ulusay and Hudson (2007). The confining pressure cell is made of glass or another transparent material for low-pressure tests, for example, on soil and very weak rock, and of steel for high-pressure tests on rock.

True-Triaxial Compressive Strength

The compressive stress (σ_1) is measured at the point of failure of a sample that is under confining pressure. In a true-triaxial test, the confining pressure is not equal in the x and y directions ($\sigma_1 \neq \sigma_2 \neq \sigma_3$) (Fig. 6). The test is done to investigate the influence of the intermediate principal stress (σ_2). A true-triaxial test apparatus is technically highly complicated. The test is normally done on cube or rectangular prism samples, where platens compress the sample from three perpendicular directions independently. The whole test setup can be placed in a pressure cell to allow for pore pressures. Various solutions are used to solve technical problems that arise from the reduction in size or change in shape of the deforming sample, whereas the test platens cannot easily change size or shape.

Tensile Strength; Direct Tensile Strength (*DTS*) and Brazilian Tensile Strength (*BTS*)

Intact ground with some attraction or gluing between grains or particles has a tensile strength. The tensile strength can be established by a confined or unconfined Direct Tensile Strength (*DTS*) test (Fig. 7a, d) or by an indirect tensile strength test, including the Brazilian (or indirect or splitting) Tensile Strength (*BTS*) test (Fig. 7b, e). The *DTS* can be done in a confining pressure cell. Standards and test procedures are given in ASTM D3967-08 (2008), ASTM D2936-08 (2008), and Ulusay and Hudson (2007). The tensile strength plots negative in the Mohr's circle diagram (Fig. 7c). The *BTS* is established by compressing a disk of the ground. The disk will fail by induced internal tensile stress. The Brazilian Tensile Strength (*BTS*) is then:

$$\sigma_{t\ \mathrm{Brazilian}} = \frac{2P}{\pi D t}$$
$$\sigma_{t\ \mathrm{Brazilian}} = \text{Brazilian Tensile Strength } (BTS)\ [Pa] \qquad (7)$$
$$P = \text{load on sample at failure } [N]$$
$$D = \text{diameter } [m] \quad t = \text{thickness } [m]$$

Estimation of the tensile strength of ground is notoriously unreliable. Small inhomogeneities or small cracks, which are often invisible, may decrease the tensile strength considerably. Additionally, failure in a tensile stress environment is a mechanism propelling itself. If failure starts, the tensile force must be taken by the remaining not yet failed part, increasing the tensile stress in that volume. This volume is stressed even more and fails faster. Tensile failure is a (very) rapid process and happens normally with little warning. For these reasons, the tensile strength of intact rock is not or only partially considered in design in rock mechanics. Examples of intact tensile strength are listed in Table 2.

Mechanical Properties, Table 2 Examples of strength values

Name	UCS (MPa)	TS (MPa)	φ′ (degrees)	cohesion′ (MPa)	range of confining pressure (MPa)
Soil					
Sand (rounded particles) (loose to dense)[1,a]			27–35	0	<0.5
Sand (angular particles) (loose to dense)[1,a]			33–45	0	
Sandy gravel (loose to dense)[1,a]			35–50	0	
Residual soil & fill[b,c]			25	0	<0.1
Clay (undrained) (very soft to hard)[2,3,d]			0	0.01–0.3	<0.5
Clay (drained)[3,a,c]			20–35	0–0.15	
Peat (drained)[3,c]			20	0.006	<0.15
Frozen dense sand (artificially frozen, T~−10°C) (short/long-term)[4,e,f]	7/4	20–50% of UCS	38/22	2/1.4	<1.4
Frozen stiff clay (artificially frozen, T~−10°C) (short/long-term)[4,e,f]	6/1.5		1.5/7.5	0.8/0.6	
Ice (natural fresh water ice)[4,5,g,h]	1–18	1.3	25–48[i]	0.25[i]	<0.2[i]
Man-made material					
Concrete (regular commercial, normal strength, Portland, 28 days cured)[6,j]	15–40	2–5	43[7,k,l]	5.5[7,k,l]	<17
Iron (yield strength)[m]	50	200			
Steel (low-alloy) (yield strength)[m]	500–1,980	680–2,400			
Intact rock					
Vindhyan sandstone[n]	102	6.9	37	33	<15
Hawkesbury sandstone[o]	21–60	3.5	47	4.2	<20
Eagle Ford Shale[p]	2.1	0.93	24	0.41	<3
Yucca Mountain-Topopah Spring Tuff[q]					
(TSw2/Tptpmn)	187	11.6	48	40	<15
(TSw3/Tptpv)	16	4.0	47	3.5	<10
Vinalmont limestone[r]	190	7			
Sibbe limestone[r]	3.5	0.38			
Basalt[8,s]	266	14.5	31	66	3.4–34.5
Königshain Granite[9,t]					
Slightly weathered	185	18			
Moderately weathered	38	3.5			
Highly weathered	13	1			
Completely weathered			25	0.015	<0.4
Residual soil			35	0.025	
Gorleben salts (short-term laboratory tests; average of different formations)[u]	26	1.6			
Rock mass					
Sheared flysch[10,v]	4.5		16	0.073	~ 2.5
Mu-Cha Tunnel Fault (sheared sandstone & shale in clay matrix)[10,w]	2.6		28	0.1	~ 3.5
Basalt[8,10,s]				0.6–6	3.4 – 34.5
Deriner Granodiorite[10,x]	80	16	40	0.35	<1.5

(*continued*)

Mechanical Properties, Table 2 (continued)

Name	UCS (MPa)	TS (MPa)	φ′ (degrees)	cohesion′ (MPa)	range of confining pressure (MPa)
Falset Granodiorite[9,11,y]					<0.6
Fresh (zone 1)	175	10.2	47	0.017	
Slightly weathered (zone 1–2)	110	4.1	46	0.016	
Moderately weathered (zone 3)	80	2.7	38	0.014	
Highly weathered (zone 4)	3		17	0.008	
Completely weathered (zone 5)	0.5		6	0.003	
Falset Lower Muschelkalk Limestone[11,12,y]					
Large blocky	80	8[z]	62	0.027	
Small blocky	70	8[z]	18	0.007	
Sydney-Gunnedah Basin coal[10,aa]		0.8	38	1.9	~ 12.5

Notes: Note the large variation between different materials, the wide variation within the same material due to different states of weathering, and the large influence of block size on mass properties while the intact material strengths (*UCS* and *TS*) are about the same. Values reported are for normal (near) surface engineering conditions. For groundmasses, the *UCS* and *TS* are of the intact ground. φ' and *cohesion*′ are effective Mohr-Coulomb failure envelope parameters with the range of confining pressures as the properties are validated within the given range only. Not all literature reports whether effective or total φ and *cohesion* are reported, but the test conditions imply effective, except where otherwise indicated. [1]"Loose to dense" refers to the packing of the particles (following BS5930), which influences the φ' values. [2]"very soft to hard" refers to the consistency of the clay BS 5930: 1999 (1999). [3]"Undrained" and "drained" refer to the dissipation of pore gas and fluid pressures during loading; generally, undrained applies to fast loading situations and drained for slow. Values reported for undrained cohesion are "*Su*" (or "*cu*") and φ and *cohesion* are total stress Mohr-Coulomb failure envelope parameters (i.e., without accent). [4]Values indicative only; strongly dependent on test conditions, deformation rate, compaction, temperature, structure, and quantity. Strength is the highest stress sustained in the test; highest values for fast loading. [5]*UCS* and *TS* at temperature −1°C to −16°C; shear properties at −2°C; shear samples contain ground fragments and air bubbles. [6]Concrete strength values are indications for *"regular"* commercially available concrete without additives or reinforcement. *UCS* of commercially available "*high-strength*" concrete is 40–150 MPa, up to 200 MPa with *TS* up to 9 MPa for "*ultra-high strength*" concrete (UHC 2006). *UCS* for special purposes concrete (e.g., military) may be over 800 MPa. [7]*Cohesion*′ and φ' depend on the strength combination of aggregate and cement matrix; i.e., does the material shear through low-shear strength aggregate or are high-shear strength aggregate particles overridden. The values in the table are therefore only indicative of a particular combination of aggregate and cement matrix. The values are based on combined regression of data from various authors. [8]Summary literature for typical values based on different basalts. [9]Weathering classifiers indicating an increasing grade of weathering from fresh to slightly weathered, etc., follow ISO 14689-1: 2003 (2003). The zones follow BS 5930: 1999 (1999) and not the replacement standard ISO 14689-1: 2003 (2003) as the replacement is at present considered by some to be inferior to the BS 5930 (Price et al. 2009). [10]φ' and *cohesion*′ determined by rock mass classification and/or back analyses from tunnel construction. [11]φ' and *cohesion*′ back analyzed from slope engineering. [12]"Large blocky" implies that most of the blocks in the rock mass are about equi-dimensional with sides between 0.6 and 2 m, while "small blocky" implies equi-dimensional with sides between 6 and 20 cm (ISO 14689-1: 2003 2003). Data from: [a]Craig (2004), [b]Chan et al. (2005), [c]Bosch and Broere (2009), [d]BS 5930: 1999 (1999), [e]Jessberger et al. (2003), [f]Zhang et al. (2007), [g]Schulson (1999), [h]Gagnon and Gammon (1995), [i]Arenson et al. (2003), [j]Bamforth et al. (2008), [k]Sonnenberg et al. (2003), [l]Wong et al. (2007), [m]Ashby and Jones (2012), [n]Dubey (2006), [o]Pells (2004), [p]Hsu and Nelson (2002), [q]Ciancia and Heiken (2006), [r]Swart (1987), [s]Schultz (1995), [t]Thuro and Scholz (2004), [u]Bräuer et al. (2011), [v]Marinos et al. (2009), [w]Yu (1998), [x]Cekerevac et al. (2009), [y]Hack (1998), [z]Koukam (1993), [aa]Sainsbury (2008)

Point Load Strength (*PLS*)

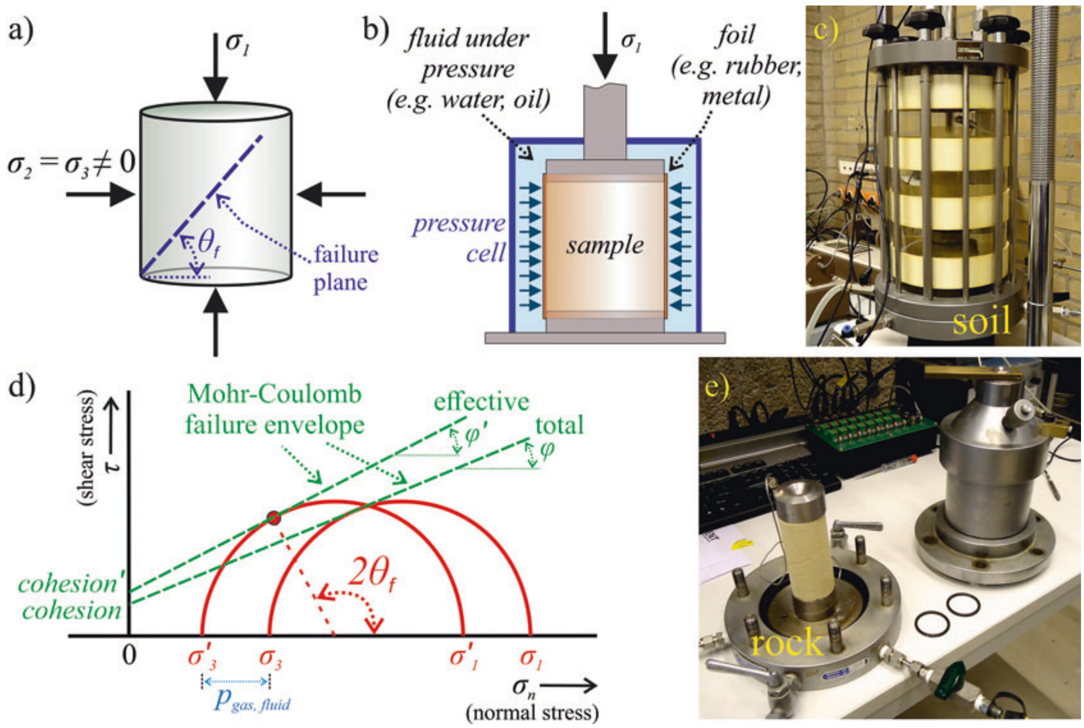
Point Load Strength (*PLS*) tests have been developed to get an idea about the strength of a piece of rock with little effort. The test can be done on lumps of rock or borehole cores with a size of about 50 mm. The sample is placed between two standardized steel "*points*" (Fig. 8). The two points are moved together by mechanical or hydraulic loading until the sample breaks. The maximum force (P) at failure divided by D^2 is "*Is*." The *Is* value has to be corrected if the sample size differs much from 50 mm. The procedure can be found in ASTM D5731-08 (2008) and Ulusay and Hudson (2007). Normally the result is denoted "Is_{50}" or "*PLS*." The *PLS* test is not intended as a replacement for Unconfined Compressive Strength (*UCS*) testing. The test is very sensitive for irregularities and inhomogeneities in the sample, such as discontinuities. In particular, if these are present near one of the points, the *PLS* value is significantly reduced. Notwithstanding this, various relations have been proposed between *PLS* and *UCS* (Rusnak and Mark 2000); a commonly used, but in the opinion of the author unreliable, relation is (Bieniawski 1975; Broch and Franklin 1972):

$$UCS = 24\ x\ Is_{50} \tag{8}$$

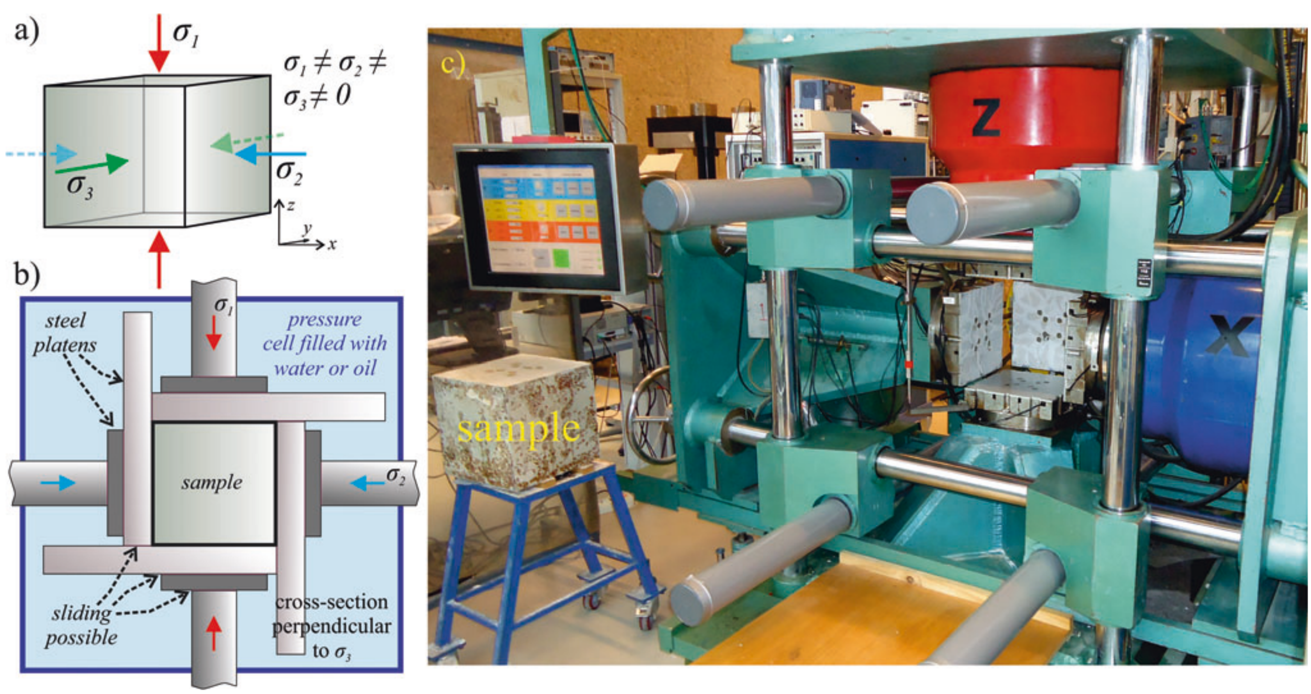
The author does not regard the *PLS* test as particularly useful, and not much if any better than the faster, and simpler to execute, rebound and "*Simple Means*" tests below.

Impact and Rebound Tests; Schmidt Hammer, Ball Rebound, and Equotip

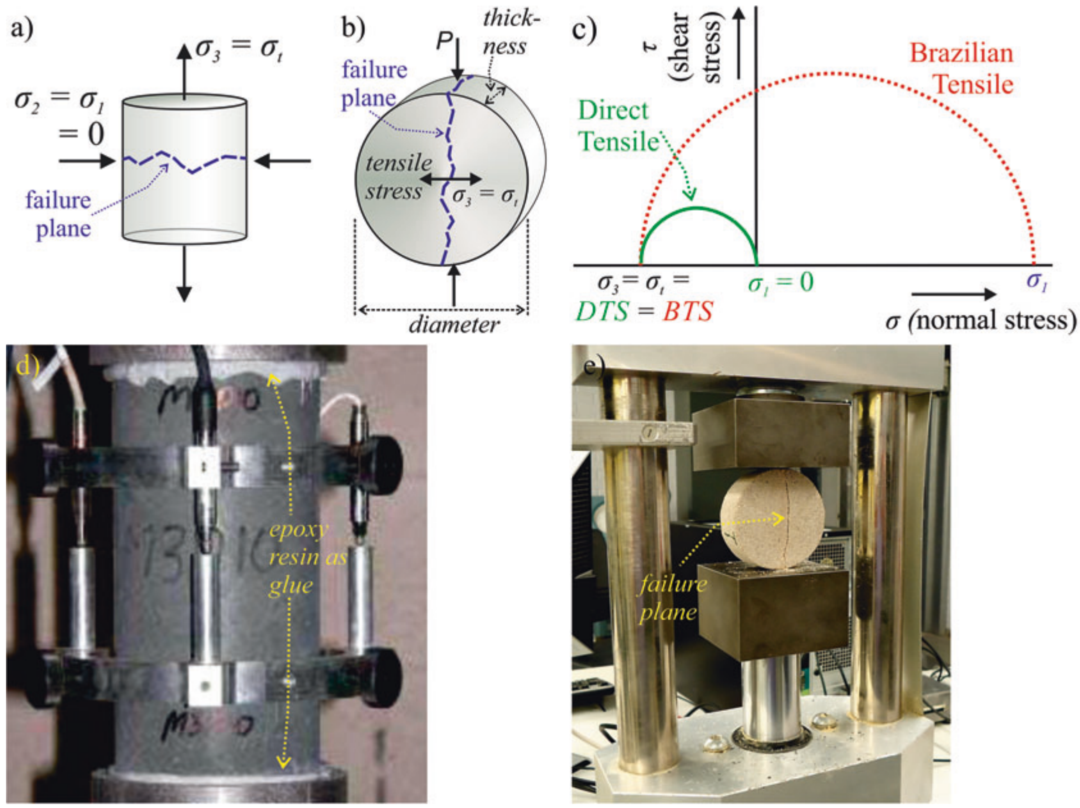
Rebound measurements are based on a piston or ball that drops from a certain height onto the surface of the material



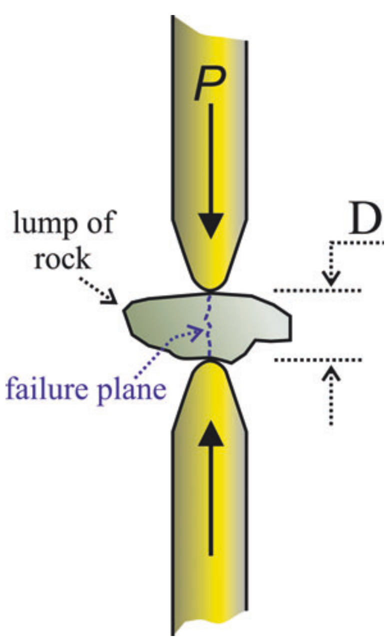
Mechanical Properties, Fig. 5 (Normal) Triaxial test (**a**, **b**) sample and stress configuration; (**c**) transparent pressure cell for soil; (**d**) stress configuration in a Mohr-circle diagram in terms of total and effective stresses; (**e**) disassembled pressure cell for rock, left bottom plate with sample, right steel upper part (Photos courtesy W. Verwaal, TU Delft, 2017)



Mechanical Properties, Fig. 6 True-triaxial test; (**a**, **b**) sample and stress configuration; (**c**) test equipment (x and z are two of the three pressure jacks) (Photo courtesy W. Verwaal, TU Delft, 2017)

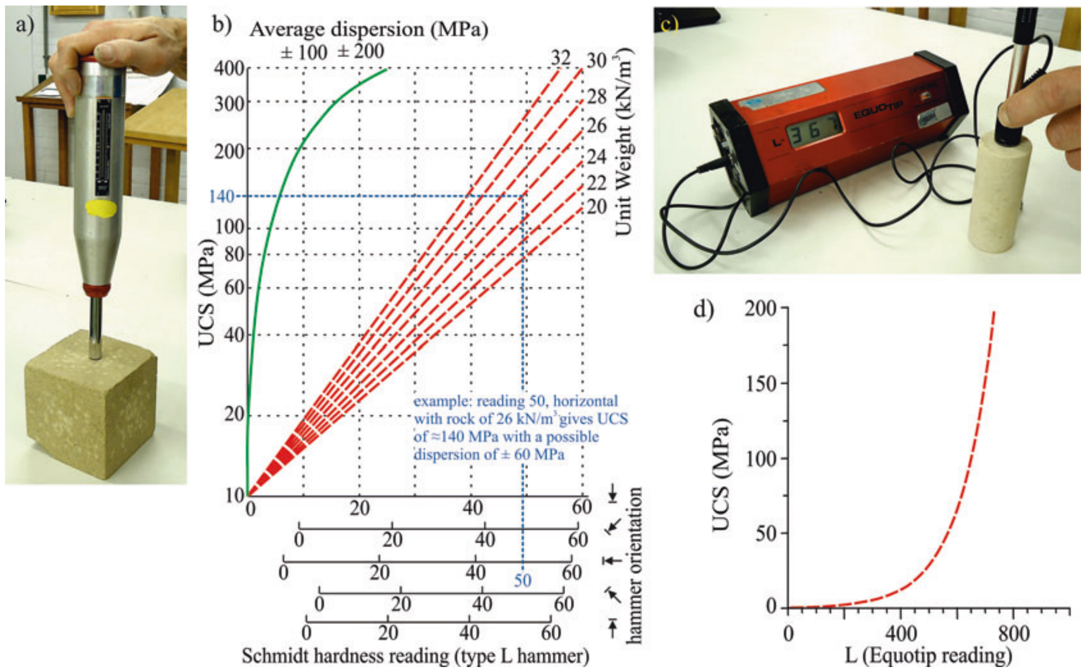


Mechanical Properties, Fig. 7 Tensile strength; (**a**, **d**) Direct Tensile Strength (DTS); (**b**, **e**) Brazilian Tensile Strength (BTS); (**c**) stress configuration in a Mohr-circle diagram (Photos courtesy W. Verwaal, TU Delft, 2017)



Mechanical Properties, Fig. 8 Point Load Strength (PLS)

to be measured. The rebound of the piston or ball after hitting the surface depends on the elastic parameters of the tested material and on the strength of the material at the surface. The crushing of surface asperities and surface material, which dissipates energy, causes this latter effect. Rebound measurement apparatus are the Schmidt Hammer which was originally developed for testing concrete quality (Fig. 9a, b) (Schmidt 1951; ASTM C805/C805M-13 2013; ASTM D5873-14 2014), the Equotip developed for steel testing (Fig. 9c, d) (Equotip 2018), and ball rebound (Pool 1981). The rebound values on rock surfaces have been correlated with intact rock strength (Fig. 9b, d) (Deere and Miller 1966; Stimpson 1965; Pool 1981; Verwaal and Mulder 1993; Hack et al. 1993; Hoek 2017; Ulusay and Hudson 2007). The execution of the test damages the rock at the impact point; asperities are crushed, and generally, the rock material will be compressed. Therefore, repeated impacts on the same location show increasing values. The tests are also influenced by local differences in structure and texture, presence of fluids (water), asperities (roughness of the surface), loose material on the surface, and in particular, a discontinuity beneath the



Mechanical Properties, Fig. 9 (**a**) Schmidt Hammer; (**b**) conversion Schmidt Hammer to UCS (Modified from Hoek 2017); (**c**) Equotip equipment; (**d**) Equotip conversion to UCS (Modified from Verwaal and Mulder 1993) (Photos courtesy A. Mulder, TU Delft, 2017)

surface. Schmidt hammer values are influenced by the material to a fairly large depth (order of centimeters) beneath the surface whereas ball rebound and Equotip release considerably less energy and are influenced by a thinner layer of material (order of millimeters).

"Simple Means" Intact Rock Strength Field Estimates

"Simple means" field tests make use of hand pressure, geological hammer, etc., to estimate the strength of cohesive soil and intact rock in classes following the British and ISO standards (BS 5930: 1981 (1981); BS 5930: 1999 (1999); ISO 14689-1: 2003 (2003)) (Table 3). Extensive numbers of tests allowed a thorough analysis of the accuracy and reliability of the simple means field tests for estimating intact rock strength (Fig. 10).

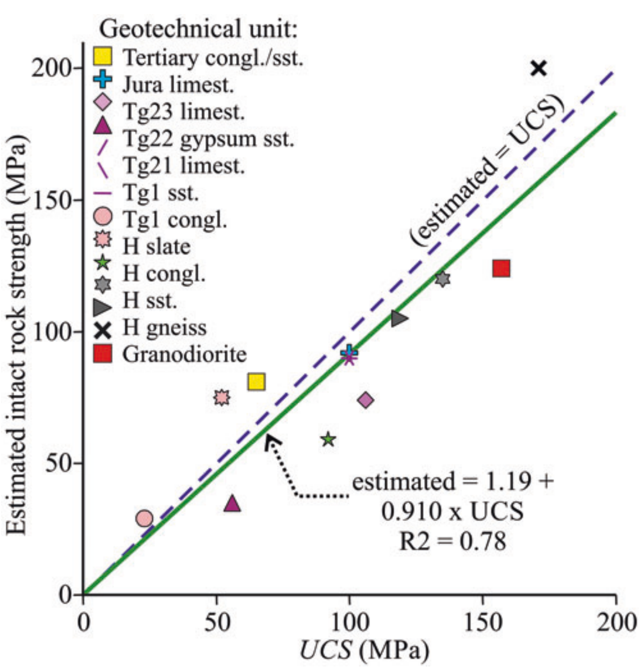
Shear Strength Tests

Direct shear strength of discontinuities and ground material can be established in a "*Golden shear box*," respectively, a "*direct shear testing apparatus*" (Fig. 11). The sample is mounted in two half steel boxes and opposite forces are applied to the two steel boxes. The horizontal displacement ($\delta_{\text{horizontal}}$) and vertical opening ($\delta_{\text{vertical}}$) between the two steel box halves are measured during the test. This allows the "angle of roughness" of the failure plane of a discontinuity, or the "dilatancy" under shear displacement of intact ground material to be established. Standards for testing are ASTM D3080M-11 (2011); ASTM D5607-08 (2008) and to be found in Ulusay and Hudson (2007). The shear test can also be executed as a "*ring* shear test" (Bromhead 1979; ASTM D6467-13 2013). In the latter test, the two halves of the test box are not translated but rotated. The advantage of the ring over a direct shear test is the potentially unlimited displacement. This makes the test particularly suitable for measuring residual shear strength properties or for determining how the shear strength changes during transition from undisturbed to remolded material. The Golden, direct, or ring test equipment can be placed in a fluid (e.g., water, oil) tank with fluid under pressure and a pore pressure transducer incorporated in the sample near to the discontinuity or the expected failure surface to measure the pore pressure. Shear strength of intact rock that is more than "*very weak*" in strength is seldom established by direct or ring-shear, but mostly by triaxial or true-triaxial testing, because the necessary stresses are so high that testing equipment is very heavy and expensive. For further details, refer to chapters on Rock Field Tests and Soil Field Tests.

Mechanical Properties, Table 3 "Simple means" field test for estimating strength (ISO 14689-1: 2003 2003; BS 5930: 1999 1999)

Cohesive soil (BS 5930: 1999)			Intact rock (ISO 14689-1 2003)		
Term	Field identification	Undrained compressive strength (kPa)	Term	Field identification	Unconfined compressive strength (MPa)
Very soft	Finger easily pushed in up to 25 mm	0–20	Extremely weak	Indented by thumbnail	<1
Soft	Finger pushed in up to 10 mm	20–40	Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1–5
Firm	Thumb makes impression easily	40–75	Weak	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5–25
Stiff	Can be indented slightly by thumb	75–150	Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25–50
Very stiff	Can be indented by thumbnail	150–300	Strong	Specimen requires more than one blow of geological hammer to fracture it	50–100
Hard or very weak mudstone	Can be scratched by thumbnail	>300	Very strong	Specimen requires many blows of geological hammer to fracture it	100–250
			Extremely strong	Specimen can only be chipped with geological hammer	>250

Notes: Some extremely weak rocks will behave as soils and should be described as soils. Geological hammer should be about 1 kg. Blows by the point of a geological hammer should be done with care as the hammer may break on impact with an unexpected harder item (e.g., a piece of quartz)



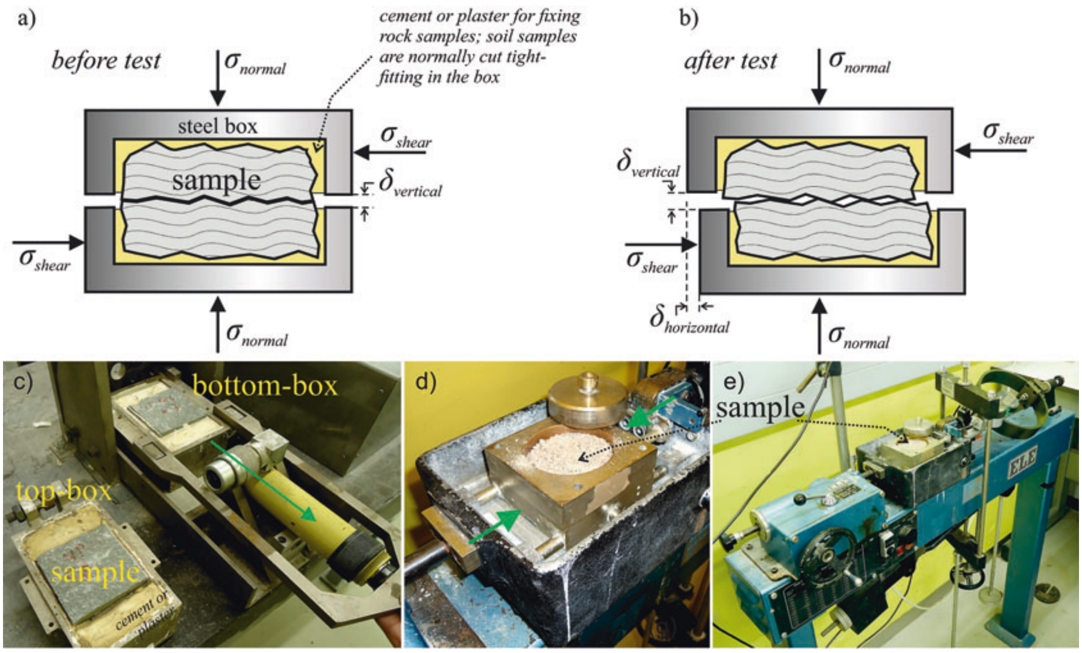
Mechanical Properties, Fig. 10 Correlation between "simple means" estimate and UCS (Modified from Hack and Huisman 2002)

Constitutive Models

Constitutive models are the aggregate term for relations that describe mathematically the chemical-physical-mechanical behavior of the ground or groundmass, normally the relation between the parameters stress, strain, strength, time, and temperature (Wang and Huang 2009; Yang et al. 2013). They may also encompass parameters such as electricity, magnetism, and nuclear radiation. Equations 1 and 2 are examples of simple constitutive models as is the Mohr-Coulomb failure envelope. Many hundreds of constitutive models for ground and groundmasses are defined; some highly complicated, for example, time and temperature-dependent viscous behavior of an anisotropic discontinuous groundmass with gases and fluids in which nuclear material is stored (e.g., Karato 2012; Cai and Horii 1992; Wang and Huang 2009). Many models for groundmasses are (in part) based on rock mass classification (Marinos and Hoek 2000; Hack et al. 2003; Barton 2002; Bieniawski 1989; Price et al. 2009). Description of these is outside the scope of this chapter.

Test Procedures, Standards and Codes of Practice

The results of the various tests mentioned in this chapter depend often on factors such as sample size and form (i.e., cylindrical, rectangular, or cubical), methodology of testing (e.g., rate of stress increase), environment temperature, and gas or fluid content. The dependencies mean that results can be compared and be applied in design of engineering structures only if the tests are done strictly following a particular



Mechanical Properties, Fig. 11 (**a**–**c**) Golden shear box test for shear strength of a discontinuity; during the test the top-box is installed on top of the bottom-box and fixed in horizontal direction, the bottom-box is pulled from underneath the top-box (green arrow); (**d**, **e**) Direct shear box test with detail of the direct sample box (Photos courtesy W. Verwaal, TU Delft, 2017)

test procedure as described in *"standards"* or *"codes of practice,"* for instance, standards of the International Standard Organization (ISO). Often test results depend to a certain extent on the method of testing; therefore, the test conditions should be as much as possible similar to those that will exist where applied, for example, the confining stress in a triaxial test should be similar to the stress environment *in situ* where the test result is applied.

Summary

Mechanical properties of ground are diverse and numerous. Although tests have been designed for all properties, inhomogeneity of the ground, restricted size of laboratory samples, and high costs of full-scale testing often prohibits a complete characterization of the ground by tests alone. Expert judgement and estimation of ground characteristics and properties is often as, if not more, important than testing.

Cross-References

- ▶ Angle of Internal Friction
- ▶ Biological Weathering
- ▶ Chemical Weathering
- ▶ Consolidation
- ▶ Deviatoric Stress
- ▶ Dilatancy
- ▶ Mohr Circle
- ▶ Mohr-Coulomb Failure Envelope
- ▶ Physical Weathering
- ▶ Rock Field Tests
- ▶ Rock Mass Classification
- ▶ Shear Modulus

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Metamorphic Rocks

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Definition

Rocks derived from other pre-existing rocks that, in the course of geological processes, have undergone mineralogical, chemical, and structural changes in the solid state, in response to the changes in physical and chemical conditions existing at depth.

Formation of Metamorphic Rocks

Metamorphism is a process in which pre-existing rocks are transformed into other rocks by increases in temperature and pressure causing changes in the mineral association, texture, and structure. Note that these changes take place in the solid state.

The composition of the rock resulting from a metamorphic process depends essentially on the original composition, the conditions of temperature and pressure, and the presence and activity of fluids. The metamorphic processes range from recrystallization, which involves the increase in size and/or the change in the external form of the original minerals, to metamorphic reactions that result in the development of new minerals in stable equilibrium under the new prevailing conditions.

The most common minerals in the rocks in the Earth's crust are plagioclase, K-feldspar, quartz, pyroxenes, amphiboles, micas, and clay minerals. Concerning metamorphic rocks, other minerals are typically represented such as garnet, chlorite, serpentine, epidote, talc, and polymorphs of Al_2SiO_5 (kyanite, sillimanite, and andalusite).

Metamorphic processes mainly occur in association with tectonic processes at plate boundaries, in three major environments: subduction zones, collision zones, and mid-ocean ridges.

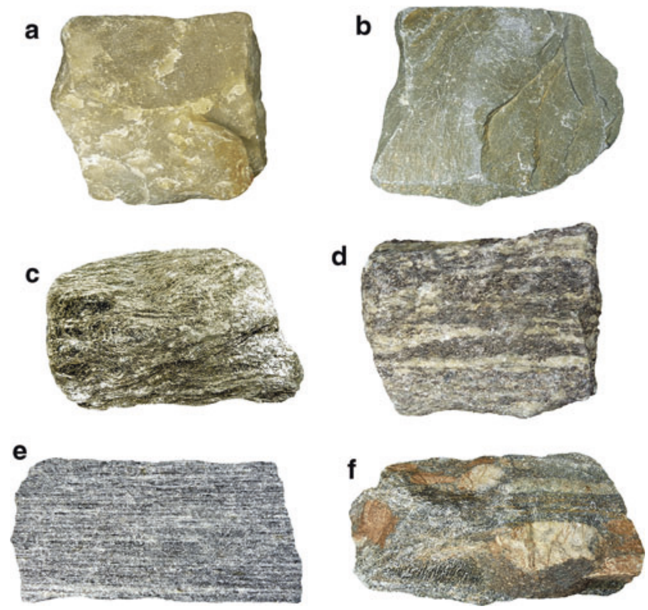
The determining factors for metamorphism are temperature, pressure, the presence of fluids, and the duration of the process.

Temperature is the most important agent, promoting the recrystallization of more stable minerals. The heat originates from magma and the geothermal gradient, which is the temperature increase with depth in the crust of the order of 20–30 °C/km. Metamorphic transformations depend on the precursor rock (or protolith), start at 150–200 °C, and end when the rock melts, transforming into magma and then igneous rock.

Fluids assist the recrystallization of precursor minerals, which occurs due to the migration of ions. The main fluid is water with dissolved salts and volatile components. In sedimentary rocks, fluids are found within their pores; in igneous and metamorphic rocks, they are present in fractures and faults. In metamorphic processes, the water may also originate from the alteration of hydrated minerals (clay minerals and micas) that constitute the original rocks.

Pressure also plays an important role and increases with depth at a gradient of 1 kilobar for each 3 km. There are two types of pressure – lithostatic and directed.

Lithostatic pressure is the same in all directions in the rock massif, not causing significant mechanical deformation and resulting in essentially equigranular mineral fabrics. Directed pressure is generated by the movement of lithospheric plates, acting vectorially and producing deformation and mineral orientation (especially those with tabular or prismatic habits) perpendicular to the maximum pressure. Directed pressure is responsible for the formation of oriented and folded structures.



Metamorphic Rocks, Fig. 1 Examples of metamorphic structures: massive (**a**), foliated (**b**), schistose (**c**), gneissic (**d**), mylonitic (**e**), and relict (**f**)

The most common structure of a metamorphic rock is oriented, but massive structures also occur, mainly in monomineralic rocks such as marbles and quartzites. Examples of metamorphic structures are foliated, schistose, gneissic, cataclastic, mylonitic, and relict (Fig. 1).

Types of Metamorphism

There are several types of metamorphism, depending on the predominating agent (temperature or pressure).

If temperature is dominant, there is contact or thermal metamorphism, which occurs when a body of magma intrudes into the parent rock, forming a metamorphic halo at its boundary. In this case there is no severe deformation, and the result is a fine-grained rock of massive structure, called hornfels.

If pressure is more important, there is dynamic or cataclastic metamorphism, which occurs in the vicinity of shear zones or faults. In this case, directed pressure causes movement and ruptures in the crust, producing mylonites (ductile deformation) and cataclasites (brittle deformation), depending on the depth of the crust level where these deformations occur:

– At more surficial (crustal) levels, purely mechanical forces predominate near faults; shear is essentially brittle, causing the fracture and fragmentation of the rock, producing the cataclasites and tectonic breccias.

– Deeper in the crust, temperature starts to act together with deforming forces, and the shearing process becomes predominantly ductile (Fig. 2), which can completely destroy the original textural arrangement of the precursor rocks (igneous or metamorphic), resulting in structures with plastic deformation.

Metamorphic Rocks, Fig. 2 Lenticular deformation of amphibolite in mylonitic gneiss in a shear zone

If temperature and pressure are equally important, there is regional or dynamothermal metamorphism, responsible for producing large volumes of metamorphic rocks, and associated with the formation of mountains at plate boundaries. It occurs over extensive regions and reaches deep crustal levels. The resulting rocks tend to be foliated, and the most common varieties are slate, phyllite, schist, and gneiss.

Two or more successive metamorphic events can occur, that is, polymetamorphism. These events may be of a higher or lower grade than the previous metamorphism. If temperature and/or pressure increase, there is progressive metamorphism, which forms minerals of higher metamorphic grade in relation to the minerals already present. If temperature and/or pressure decrease, there is retrograde metamorphism, which forms minerals of lower metamorphic grade in response to the new physical conditions.

The relative position of the metamorphic rocks in the P-T field is presented in Fig. 3.

The minerals that constitute the original rock respond differently to these processes. Quartz readily undergoes intracrystalline deformation, showing microscopic deformation, such as undulatory extinction. After the finalization of forces, recrystallization occurs, and a mosaic of recrystallized grains can occupy the location of what previously was a single grain of quartz.

The presence of undulatory extinction and very fine quartz grains (<0.15 mm) are criteria for assessing the suitability of crushed rock (coarse and fine aggregate) for use in concrete,

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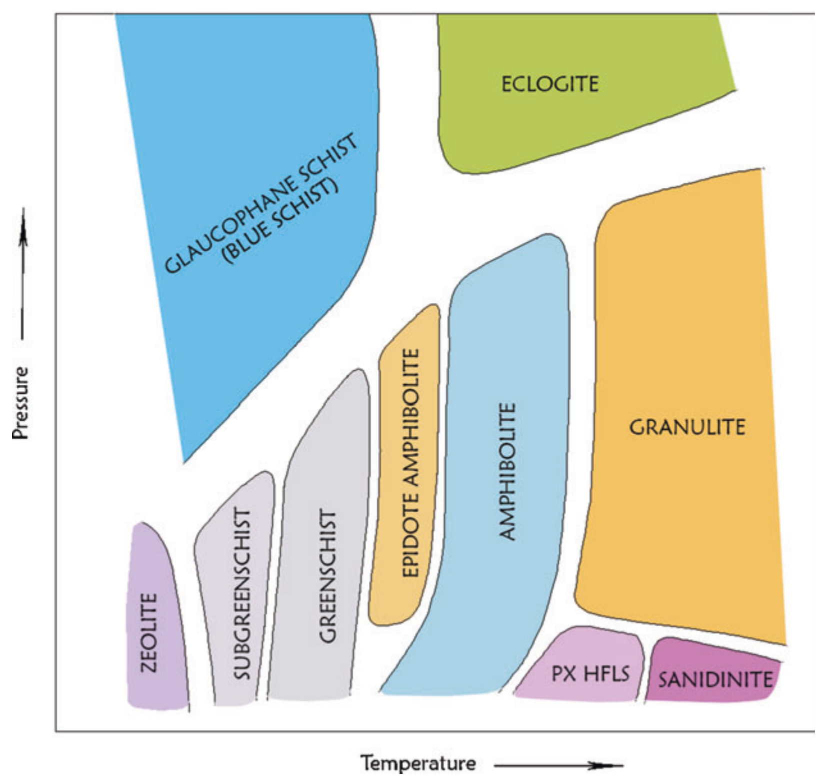
Metamorphic Rocks, Fig. 3 Diagram showing the P-T fields of metamorphic rocks. PX HFLS = pyroxene hornfels



because a direct relationship has been found between these features and the potential for alkali-silica reactions in concrete, forming expansive compounds that could damage and destroy the structures of civil engineering works, such as dams.

Other minerals, such as feldspars, rarely exhibit intracrystalline deformation. In general, only their edges deform and tend to consist of rounded relict crystals (porphyroclasts), also known as augen.

Classification of Metamorphic Rocks

Metamorphic rocks can be classified based on three characteristics – structure, mineralogy, and their protoliths.

The structural classification reflects the arrangement and grain size of the constituent minerals, which characterize the metamorphic grade. At a low metamorphic grade, the rocks formed are oriented and finely granulated, in which the minerals are not visible to the naked eye. They thus have the foliated structure of slate and phyllite. At a higher metamorphic grade, the minerals become micaceous or prismatic, resulting in the schistose structure characteristic of schists. A further increase in the metamorphic grade promotes the segregation of minerals into bands with larger grain size than observed in schists, resulting in the gneissic structure characteristic of gneisses. In the higher metamorphic grade granulite appears, a rock with Fe-Mg silicates (dominantly pyroxene).

In fault regions with a strong action of directed pressure, mylonitic or cataclastic structures form, with elongated or fractured mineral grains, respectively.

Migmatitic structure is peculiar to a hybrid rock, consisting of igneous and metamorphic portions due to partial melting and may or may not exhibit fold features.

The absence of orientation of the mineral grains results in the massive structure that is common in rocks of a high metamorphic grade.

The mineralogical classification is most commonly used for monomineralic rocks, and the presence of orientation is associated with the constituent mineral. Marble, quartzite and amphibolite are some examples of rocks in this category.

Classification based on the protolith is used when the rock retains relict structures that allow the original rock to be recognized, usually in rocks of a low metamorphic grade. Their names have to include the prefix *meta*, for example, metaconglomerate.

Some Types of Metamorphic Rocks

Slates, phyllites, and schists: characterized by high content of micaceous minerals and well-developed foliation. These are the metamorphic products of pelitic sedimentary rocks consisting mainly of clay minerals or clay or silt-sized grains.

– Slate: very fine-grained rock exhibiting a strong planar orientation, called slaty cleavage. It consists mainly of sericite and quartz. Its principal characteristic is fissility – the property of a rock to fracture easily along fine cleavage or stratification planes, a property that can favor the occurrence of landslides, slippages, and other phenomena. On the other hand, this characteristic favors the extraction of slabs that are widely used as flooring and roofing materials in countries with cold climates because of their mechanical strength (under bending forces) and also thermal insulation able to withstand snow.
– Phyllite: very fine-grained and strongly foliated rock composed mainly of sericite and quartz, and as accessory minerals graphite, chlorite, feldspars, and other minerals. Phyllosilicates confer a characteristic silky lustre on the rock.
– Schists: medium-to-coarse grained rocks, generally visible to the naked eye, with strongly planar or linear preferential arrangement. They typically consist of phyllosilicates (muscovite and/or biotite) and quartz, usually accompanied by metamorphic minerals characteristic of the P and T ranges in which they formed, such as garnet, sillimanite, staurolite, among many others, often constituting porphyroblasts or poikiloblasts.

Gneisses and migmatites: resistant rocks suitable for most engineering purposes, unless they contain foliation planes (especially rich in micaceous minerals, such as biotite) in quantities and dimensions that can constitute discontinuities or sites conducive to landslides/slippages.

– Gneisses: usually quartz-feldspathic rocks, medium-to-coarse grained, and showing moderate-to-strong planar orientation provided by the iso-orientation of tabular or prismatic minerals, called gneissic structure or foliation. They may be derived either from the deformation of granitic rocks or from the total mineralogical and textural reorganization of rocks, especially pelitic rocks, under high-grade metamorphic conditions (high P and T), resulting in a mineral association of quartz, K-feldspar, and plagioclase, with garnet, cordierite, aluminosilicates, and muscovite.
– Migmatites: rocks of heterogeneous compositions and structures (called *migmatitic*), usually medium-to-coarse grained, which often occur in terrains of high metamorphic grade. Megascopically, migmatites consist of light-colored (leucocratic) portions of a low-mafic, granitic (quartz-feldspathic) composition interlayed in dark-colored (melanocratic) portions that are generally foliated and have mafic minerals in their composition, with gneissic structures.

Quartzite: rocks formed almost exclusively from recrystallized quartz in an arrangement called *granoblastic*, generally derived from siliceous sediments – quartz sandstones or cherts. These are often white in color, with variations toward red (due to the presence of iron hydroxides) or yellow tones. They are very hard rocks, with high resistance to crushing and cutting with diamond saws, producing great wear in equipment. They are also very resistant to both weathering and hydrothermal alteration. Mica quartzites, also called flagstones, are widely used for flooring.

Marbles: rocks consisting of more than 50% carbonate minerals, more specifically calcite and/or dolomite, formed by the metamorphism of calcitic and/or dolomitic sedimentary rocks. They are of massive structure and varied grain size (from fine to coarse) and of different colors – white, pink, gray, green, etc. The texture is typically granoblastic and, in addition to carbonates, they may contain talc, amphibole (tremolite), pyroxene (diopside), and olivine (forsterite), among others.

Amphibolites: dark-colored (dark-green to black) rocks, fine-to-medium grained, consisting primarily of hornblende and plagioclase, usually with opaque (magnetite) and titanite accessories. In general they are products of the metamorphism of basic rocks (basalt). The metamorphism of basic rocks can also result in the formation of dark-green colored and fine-grained rocks, rich in actinolite, epidote, and chlorite. When their structure is oriented, they are generically referred to as *greenschists*, and when massive they are known as *greenstone*.

Additional information about metamorphic rocks is found in Yardley (1989), Philpotts (1989), Yardley et al. (1990), Bucher and Frey (1994) and Winter (2010).

Applications

Metamorphic processes lead to the production of metallic and nonmetallic mineral resources including large deposits of gold and iron.

Metamorphism is also responsible for the origin of some industrial minerals (wollastonite, asbestos, graphite) and gemstones (garnet, andalusite, ruby/sapphire, emerald, diamond).

Metamorphic rocks are of great importance in the construction industry, either in the form of crushed stone or dimension stone.

The majority are used as a facing material, with emphasis on the gneisses and marbles. Marble has been used for floors for thousands of years and is widely used due to its high workability and aesthetic diversity. An internationally famous example is the marble from Carrara (Italy).

Summary

Metamorphic rocks are formed from precursor rocks modified by increase of pressure and temperature, leading to changes in the mineral association, in texture, and in the structure of the rock – processes that occur in the solid state. Their characteristics depend on the protolith (structure, texture, and composition of the original rock), the combined action of pressure and temperature, or the predominance of one of these factors, the time lapse of the metamorphic processes, and the presence or absence of fluids. Examples of metamorphic rocks include slates, phyllites, schists, gneisses, migmatites, marbles, amphibolites, and quartzites.

Cross-References

- ▶ Aggregate
- ▶ Aggregate Tests
- ▶ Alkali-Silica Reaction
- ▶ Alteration
- ▶ Armor Stone
- ▶ Bedrock
- ▶ Building Stone
- ▶ Cap Rock
- ▶ Classification of Rocks
- ▶ Crushed Rock
- ▶ Geochemistry
- ▶ Hydrothermal Alteration
- ▶ Igneous Rocks
- ▶ International Society for Rock Mechanics (ISRM)
- ▶ Limestone
- ▶ Mechanical Properties
- ▶ Petrographic Analysis
- ▶ Rock Field Tests
- ▶ Rock Laboratory Tests
- ▶ Rock Mass Classification
- ▶ Rock Properties
- ▶ Sedimentary Rocks

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Mine Closure

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Definition

The process for ending the operation of a mine.

Mine closure is the process for ending the operation of a mine. It is commonly embodied in a closure plan developed as part of the operations plan for a particular mine. Robertson and Shaw (2006) articulate four key objectives to be considered when closing a mine from operation:

1. Protect public health and safety.
2. Alleviate or eliminate environmental damage.
3. Achieve a productive use of the land or a return to its original condition or an acceptable alternative.
4. To the extent achievable, provide for sustainability of social and economic benefits resulting from mine development and operations.

Orderly mine closure depends on planning for providing the details on design and costs to achieve these key objectives (ICMM 2008; Australian Government 2016).

Mine Closure for Active and Abandoned Mines

It may seem counterintuitive to consider closure during the development and opening of a mine. But there are several advantages to engage, at some level, in planning the life of a mine from start to finish (ICMM 2008; Australian Government 2016). First, there is an economic advantage to eliminating or minimizing future liability and risk including those associated with mine closure (Robertson and Shaw 2006). For example, returning the mined site to a productive use or its original condition might be done in sequence with expansion of the mine during its operational life. This permits land reclamation to incorporate waste rock and subeconomic ore in a manner that minimizes the expense of operational handling and, also, overall reclamation costs. Phased reclamation has the additional advantage of reducing the cost of reclamation bonding (depositing money in advance to ensure availability of sufficient funds) compared to deferring the reclamation until the mine is to be closed. Second, establishing the broad outlines of mine closure at the mine's start means adjustments to achieve planned actions may require only small incremental changes when the mine's operational plan is later modified in response to market forces or physical conditions on the ground (ICMM 2008). Third, closure plans are expected to be modified as additional data are developed during the life of a mine. The initial closure planning can draw upon the environmental assessment data completed for the mining effort. Whereas the elements of the closure plan are more conceptual at the start-up of the mine, these elements become increasingly detailed over the operational life of the mine. Consequently, the closure plan over the years becomes more relevant to the conditions present when closure takes place (ICMM 2008; Australian Government 2016). Lastly, closure planning is commonly part of the regulatory environment associated with mine operations. Storm water discharge, hazardous substance storage compliance, and water quality monitoring are just some of the areas typically having these mandated requirements. The closure plan provides a means for making certain there is compliance with applicable laws and regulations which may call for a completion plan, changes in monitoring, or transitioning to different sets of requirements (Australian Government 2016).

In contrast to the orderly approach to mine closure expected for current mines, abandoned mines from past mining booms are found in many regions. For example, an estimated 500,000 abandoned mines are present in the United States, especially the western states (BLM 2017; De Graff 2007). Such historic mines require closure actions because they pose:

1. Hazards to public health and safety
2. An adverse impact to the environment
3. Prevent or limit the productive use of the land

The intervening years or decades between the end of active mining and today allowed changes which make abandoned mine closure more difficult and expensive. Deterioration of underground works or above ground access may present worker safety and equipment suitability issues which increases implementation costs (De Graff 2007). For example, underground works can become flooded and oxygen-deficient areas can develop. Depending on the type of metal or mineral being mined, explosive gases can accumulate. Both underground and above ground, workers may encounter deteriorated explosives originally intended for mine development.

Deterioration or loss of mine roads can result from seasonal or storm damage, degrading their usefulness. Changed land use, such as a new canal or wilderness designation, can render them inaccessible. Where road repair is too costly, some equipment may be unsuitable for the closure work because it cannot negotiate the damaged road or is too large or heavy for helicopter lifting (De Graff 2007). Closure at a mine may require removing buildings which supported the mine operations or house the facilities for processing ore (Fig. 1). Building

Mine Closure, Fig. 1 The office, workshop, and storage facility for the Evening Star Mine, a gold mine in the southern Sierra Nevada, California (USA). Removal of this structure also included its contents, draining and disposing of fuel in the above ground tanks and disposing of metal and other solid waste debris around the buildings

Mine Closure, Fig. 2 Rinconada mill and mine site in the Central Coast Range north of the City of San Luis Obispo, California (USA). The cinnabar-rich ore extracted from underground works in the hill adjacent to the mill was roasted and the mercury vapor cooled in condensers for collection. The red, unvegetated slopes below the mill are covered with roasted tailings still containing some mercury

demolition, solid waste from operations, and disposal of unused fuel and similar chemicals may be required.

Where ore processing took place, there may be hazardous materials from the beneficiation process or due to environmental exposure of the tailings. Hazardous materials are typically highly regulated as to handling, transport, and disposal (De Graff 2007). The Rinconada Mine in Central Coast Range near the City of San Luis Obispo, California (USA), illustrates this situation (De Graff et al. 2007; Fig. 2). The mine extracted mercury for processing at a small mill at the mine site. The tailings accumulated on the slope below the processing mill and extended into a dry creek bed. Closure actions for the mine required blocking access to multiple openings into the underground works. By comparison, the

Mine Closure, Fig. 3 A view of the tailings at Tiptop Mine in the Central Coast Range of California, a World War II-developed mine to produce chromite. Seasonal erosion of the steeply sloped, unreclaimed tailings introduces gravel to fine-grained sand into the downstream channel. These additional sediments can potentially be transported into the coastal estuary at Morro Bay, California (USA), a few miles to the west of the mine

closure actions for the mill required draining and disposing of leftover diesel fuel, removal of some surface soil and tailings with high levels of mercury present to specialized disposal facilities, capping of remaining tailings with imported clean soil, erosion control measures to stabilize the creek and adjacent slope, and removing or making inaccessible mercury-contaminated parts of the mill equipment. Even when tailings do not contain or generate hazardous substances, they can be a source of excessive sediment affecting normal stream flow, impairing aquatic habitat, and adversely affecting aquatic species (Fig. 3).

At abandoned mines, funds for mine closure are absent or any bonded funds available are insufficient resulting in the need for governmental expenditures. Implementation of the closure at Rinconada Mine site cost more than $800,000 which was provided by participating US government agencies. The expense of abandoned mine closure is a primary reason for prioritizing where closure actions should be undertaken in an orderly fashion (e.g., Kubit et al. 2014). This common circumstance of abandoned mine closure highlights the benefit of integrating mine closure into present-day mine operations.

Engineering Geology in Mine Closure Planning

Engineering geologists play a significant role in mine closure planning and implementation. They provide geologic characterization of waste rock and other material to be used in reclamation. This data serve as a basis for avoiding release of undesirable chemicals into the fluvial or

Mine Closure, Fig. 4 An interior view of the Tiptop Mine, a combined underground and open pit operation. The mine disturbed area intercepts normal drainage from the adjacent hill which collects in the bottom of the pit. Also present is a high nearly vertical headwall with rockfall hazard potential

Mine Closure, Fig. 5 A gated portal for an adit (horizontal underground mine feature) at the Blue Light Mine located in the mountains east of Irving, California (USA). The gate prevents public access to the unsafe, flooded part of the underground works. The visible fractures and discontinuities in the rock were evaluated to ensure the long-term integrity of this structure preventing public access

groundwater environment as a result of weathering and erosion. For design purposes, revegetation will be more successful with geologic characterization data identifying the fertility and hydrologic capability of materials exposed at the mine site (Fig. 3). The engineering geologist will be instrumental in designing stable slopes with rock or soil and with modification of potentially unstable high walls created during the extraction process (Fig. 4). Restoring natural drainage disrupted by the mining operation or ore-processing activities can be a challenge to overcome with geologic information on the existing undisturbed drainage network in the area and finding ways to design topographic reclamation to reconnect to this network (Fig. 4). The engineering geologist also provides characterization around shaft, adit, and tunnel entrances to underground works. These data are needed to identify a suitable closure method. The use of steel grates on adit and tunnels and anchored screens over shaft openings requires a strong rock mass to ensure their integrity (Fig. 5). Without this anchoring capability, other measures would be more likely to achieve the public safety goal of preventing entrance to the underground works.

Summary

Mine closure ensures public safety, protects the environment, facilitates restoration of the site to productive land use, and contributes to economic and social sustainability. Engineering geologic expertise is an important factor for effectively implementing many mine closure actions. The cost of necessary work to close older, abandoned mines demonstrates the financial benefit of having mine closure incorporated into the overall operation plan at new mines.

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Cross-References

- ▶ Acid Mine Drainage
- ▶ Environmental Assessment
- ▶ Erosion
- ▶ Excavation
- ▶ Gradation/Grading
- ▶ Land Use
- ▶ Mining
- ▶ Sand
- ▶ Tailings

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Mineralization

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Definition

From an engineering geological perspective, *mineralization* is the chemical alteration, replacement, and enrichment of minerals.

Mineralization takes place in (1) voids, veins, pipes, stockworks, and structural reefs (Evans 1993; Robb 2005); (2) disseminated and concentrated in porous and fractured host rocks by deep circulating groundwater and hydrothermal action (Pirajno 2012); (3) in unconsolidated deposits by evaporation (Warren 2016), weathering, erosion, deposition, and precipitation (Edwards and Atkinson 1986); and (4) by replacement of organic matter by inorganic material leading to fossilization (Donovan 1991). Exploration, mining, and remediation of mineralized terrain are fields of interest for engineers and geologists.

Mineralization includes chemical alteration, *replacement*, and *enrichment* of minerals within igneous, metamorphic, and sedimentary rocks. *Replacement* is a process whereby one system component is progressively substituted by another, for example, sulfide mineralization during hydrothermal alteration of limestone by replacement. *Enrichment*: results from mechanical or chemical processes by which the relative amount of a constituent mineral or element in a rock body is increased, either by selective removal from within or introduction from an external source. Respective examples include the concentration of heavy minerals (e.g., gold and cassiterite) in placer ore deposits and removal of iron ions by downward percolating groundwater increasing copper grade in porphyry ore deposits.

Mineralization processes include:

1. *Precipitation* from magmatic fluids and gases confined to chambers, voids, veins, pipes, stockworks and structural reefs., or chemical reactions between circulating groundwater and country rock – changes in pressure and temperature, and evaporation cause minerals, for example, calcite and quartz, to precipitate from water solution
2. Dissolution, *dissemination,* and concentration in porous and fractured host rock by deep circulating groundwater and hydrothermal action – *dissemination* is the infilling of rock pores by minerals or replacement of existing pore-filling material, for example, a porphyry deposit containing fine particles of copper sulfide minerals dispersed through the enclosing rock
3. Concentration of minerals in unconsolidated deposits by evaporation, weathering, erosion, deposition, and precipitation
4. Replacement of organic matter by inorganic material leading to fossilization

Ore bodies are mineralized bedrock, unconsolidated surface deposits, and soils containing metal-bearing and nonmetallic minerals with socio-economic value. During exploration and mining, lithological descriptions, field relationships, core samples, borehole logs, geochemical analyses, geophysical data, geomechanical testing, and chronostratigraphic cross-sections are evaluated to identify mineral-bearing units, their bounding surfaces, and *ore grade* which is the concentration of ore minerals and metals contained in the Earth material (bedrock, surficial deposits, or soil) being mined. Style of mineralization and variations in ore grade will influence extraction options, economic value, and life span of a mining operation.

Geological factors controlling selection of mining method include the nature of the intrusion, chemistry of mineral-bearing fluids, style of mineralization, lateral variations in rock type, faults and other structures, and presence of groundwater. For surface pits and underground mines, site damage, overburden failure, and loss of life are minimized when competent rock or weak beds, faults, fractures and folds, permeable, and porous units are identified through detailed surface and subsurface mapping, stratigraphic analysis, and sequence modelling. Petrology, lithology, structural geology, mineralization type, surface hydrology, groundwater distribution, contamination migration paths, and natural background conditions influence environmental restoration and remediation options for open pits, underground workings, solution mines, mills, processing plants, waste rock piles, tailings ponds, and related infrastructure (e.g., bridges and roads).

Cross-References

- ▶ Acid Mine Drainage
- ▶ Aeromagnetic Survey
- ▶ Alteration
- ▶ Bedrock
- ▶ Biological Weathering
- ▶ Borehole Investigations
- ▶ Cap Rock
- ▶ Chemical Weathering
- ▶ Conductivity
- ▶ Contamination
- ▶ Cross Sections
- ▶ Dams
- ▶ Dissolution
- ▶ Engineering Geological Maps
- ▶ Evaporites
- ▶ Geochemistry
- ▶ Geological Structures
- ▶ Geophysical Methods
- ▶ Groundwater
- ▶ Hydrology
- ▶ Hydrothermal Alteration
- ▶ Igneous Rocks
- ▶ Metamorphic Rocks
- ▶ Mine Closure
- ▶ Mining
- ▶ Mining Hazards
- ▶ Petrographic Analysis
- ▶ Physical Weathering
- ▶ Residual Soils
- ▶ Rock Mechanics
- ▶ Sedimentary Rocks
- ▶ Sequence Stratigraphy
- ▶ Soil Properties
- ▶ Subsurface Exploration
- ▶ Voids
- ▶ Volcanic Environments

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Mining

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Synonyms

Abstracting; Cutting; Digging; Dig-out; Excavating; Extracting; Hollowing; Quarrying; Removal; Scooping; Unearthing

Definition

Mining is the extraction or abstraction of mineral deposits from the surface or subsurface of the Earth.

Overview of Mining, Minerals, and Society

"If it can't be grown, it has to be mined"...! Almost every item we use contains minerals. Society has been and continues to be dependent on minerals produced by mining. It could be argued that every material item in society is produced from a mineral or derived from a mineral product. Mineral resources are therefore likely to remain a fundamental necessity for society into the future.

Mining, like agriculture, represents one of the earliest activities of *Homo sapiens*. Evidence (stone tools) for mining extends back to Paleolithic times (ca. 450,000 yr. BP). Surface/underground mining in parts of Africa (e.g., Swaziland) dates back to 40,000 yr. BP. Fired clay pots in Czechoslovakia date back some 30,000 yr. BP, and gold and copper were being worked in some parts of the world as long ago as 18,000 yr. BP. Since then, mining has continued to expand and helped advance and develop human civilization, through the Bronze Age (4000–5000 yr. BP), Iron Age (1500–1780 yr. BP), Steel Age (1780 BP–1950 AD), and the Modern Age (1950–present) (Agricola 1556). Mining has taken place throughout the world for hundreds and, in some localities, thousands of years. In the UK, for example, evidence for mining dates back to the Neolithic where flint nodules contained in chalk were mined for tool making (e.g., Grimes' Graves, Norfolk, UK), and coal was mined by the Romans as long ago as 122 AD. Vast fortunes, economic wealth, prosperity, and growth have resulted from mining and the mineral commodities produced. The Industrial Revolution in Britain was founded upon coal and associated mineral resources, resulting in the establishment and expansion of industrial cities. Mineral commodities are the end product of mining. Modern society would not function as we know it, without mining and the raw materials and commodities associated with mining. Minerals are required for tools, utensils, building and construction, food, weapons, ornamental jewelry and cosmetics, currency, energy to produce heat and power, industrial machinery, electronics, and nuclear fission (Table 1).

Clearly, there are many benefits of mining as minerals contribute significantly to the economic development of countries, ease poverty, and directly or indirectly improve people's lives around the world. Conversely, the collapse of mining driven by economic recession, falling commodity prices, or geological complexities has resulted in entire communities or towns becoming adversely affected causing economic and social deprivation or environmental hazards. As such, mining is a "boom-and-bust" industry.

The Life Cycle of a Mine

Geological exploration and engineering geological and geotechnical investigations to locate, define, and characterize the deposit precede a mine. Next technological, financial, and social evaluations determine if the mine is economically viable. The mineral(s) is then extracted from the ground by mining engineers before mineral processing engineers (metallurgists) prepare the mineral into a higher-quality product, concentrating and refining the minerals (or rock) for marketing and sale. Finally, all mines eventually close and are abandoned, and this can leave behind a legacy of hazards and liabilities that may detrimentally impact the economy, environment, and society. Geologists are likely to be involved in almost every stage of a mine's life, although the majority of the engineering geological requirements will be during exploration, mine design, operation, and abandonment. A mine therefore has a "life cycle" as shown in Fig. 1 and outlined below.

Prospecting and Exploration

All mining operations start with prospecting and exploration. The content, scale, and costs of duration of exploration programs vary considerably. Exploration is carried out and funded by geological surveys, minerals agencies, mining companies, consultants, or contractors. The design of an exploration program is influenced by the political and security regime within a particular country, remoteness of location, available infrastructure, land ownership, world commodity markets, and weather. Generally, exploration aims to locate, define, and characterize an economically viable mineral, within a prospect (also known as a lease, tenure, or tenancy). Raising finance and securing a mineral prospect are the first stage of exploration, known as prospecting (i.e., exploration limited in scope), which might lead to a discovery. Securing the legal rights to prospect and explore often requires collaboration with a geological or resources ministry and the award of an exploration license. This will define the area where exploration can be legally conducted, by whom, the permissible techniques, duration, and terms of the tenure. The distinction between prospecting and exploration is ill-defined, this is normally transitional with exploration referring to an increase in the scale of exploration operations.

Exploration may take place in an entirely new geographical location (grassroots exploration) to define the type, size, geometry, quality, quantity, and grade of the mineral(s) or to extend an existing mine or known deposit (mine-based exploration). It is time consuming and expensive, and the costs for exploration can be in the order of several millions of dollars. Exploration budgets are generally reviewed by the investors on a regular basis. Returns on exploration investment may take years, if and when the mine goes into operation. Exploration is a high-risk business, and not all exploration targets develop into a mine. Exploration surveys may be halted when it can be shown they are not economically viable, although they may reopen in the future if the controlling factors (economic, technical, security, or political) change.

Desk Study and Target Evaluation

Exploration begins with planning, the commissioning of a desk study, and development of an exploration strategy (Marjoribanks 2010; Moon et al. 2006). The relevance and veracity of any existing data and information must be diligently assessed. For example, if there are airborne geophysical data with obvious anomalies, those with an increased likelihood of containing economic minerals are selected for further consideration and targeted exploration. Evaluation may begin with a review of the following (if available):

- Geological and geomorphological maps, reports, and analytical data from a geological survey
- Satellite imagery and air photographs (including stereographic pairs)
- Airborne geophysical data
- Geochemical data
- Reporting of past exploration results
- Technical and assay reports for soil and rock sampling from exposures, trenches, and drill core and preliminary drilling

Ultimately, a conceptual geological model should be developed to characterize and categorize the type and style of mineralization and identify data deficiencies and gaps in knowledge about the deposit.

Geological Mapping

Geological mapping, often initiated during prospecting, involves the detailed assessment of the mineral deposit including the structures, mineralization, alteration, geomorphology, and the collection of soil and rock samples for further

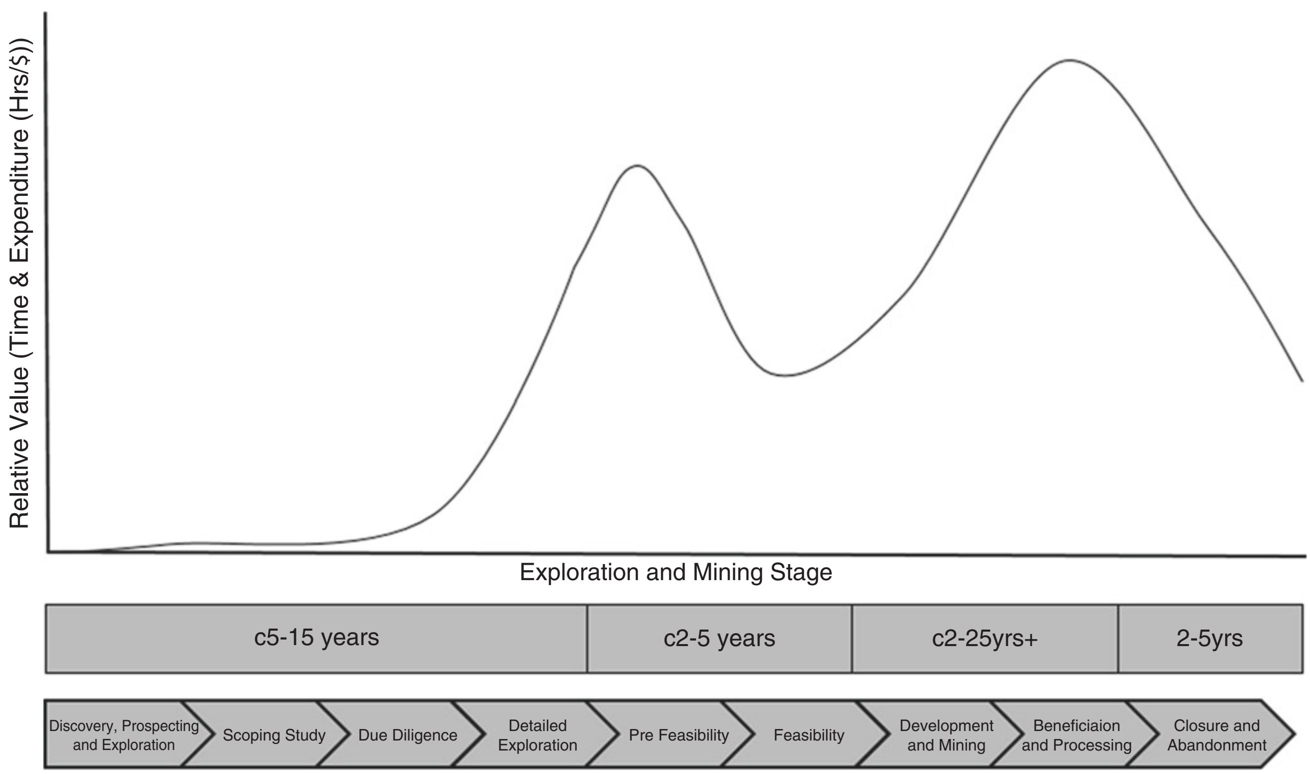
Mining, Table 1 Minerals, commodities, and their common uses

Mineral	Commodity	Uses
Bauxite	Aluminum	Vehicles, packaging, building construction, electrical, machinery
Beryllium	Beryl	Light, strong alloys, aircraft industry, fluorescent lamps, X-ray tubes, gemstones (emerald and aquamarine), computers, telecommunication, aerospace, defense, medical equipment
Cobalite	Cobalt	Superalloys, aircraft gas turbine engines, cutting tools, wear-resistant applications, chemicals (paint dryers, catalysts, magnetic coatings), permanent magnets
Stibnite	Antimony	Lead batteries, cable sheaths, bearing metal, type metal, solder, collapsible tubes, foil, pipes, semiconductor, flame retardant, fireworks, rubber, chemicals, textiles, medicine, glassmaking
Feldspar	Feldspar	Glass, ceramics, enamelware, soaps, abrasive wheels, enamel, insulating compositions, fertilizer, roofing materials, textiles and paper filler, pottery
Zinc and bauxite ore	Gallium	Circuits, light-emitting diodes (LEDs), photo detectors and solar cells, treatment of cancer, defense applications, computers, telecommunications
Gypsum	Calcium sulfate	Prefabricated wallboard, building plaster, cement, agriculture
Zinc ores	Indium tin oxide	Electrical conductivity, liquid crystal displays (LCDs), solders, alloys, compounds, electrical components, semiconductors, research
Galena	Lead	Batteries, tanks, solders, seals, electrical, TV tubes, glass, construction, communications, weights, ceramics, crystal glass, X-ray and gamma radiation shielding, soundproofing, ammunition
Pyrolusite	Manganese	Iron and steel, ferroalloys, construction, machinery transportation
Molybdenite	Molybdenum	Car parts, construction equipment, gas transmission pipes, stainless steels, tool steels, cast irons, superalloys, chemicals, lubricants, light bulbs
Polite	Silica	Building, construction, fillers, horticulture aggregate
Apatite	Phosphate	Phosphoric acid, phosphate fertilizers, feed additives for livestock, phosphate chemicals for industrial and domestic uses
Quartz	Silica	Semiprecious gem stone (amethyst, citrine, rose quart, smoky quartz, agate, jasper, onyx); piezoelectric properties (pressure gauges, oscillators, resonators, computer chips, glass, refractory materials); ceramics, abrasives, water filtration, hydraulic cements; cosmetics, pharmaceutical, paper, insecticides, foods, paints; thermal, photovoltaic cells. China is the leading producer
Argentinite	Silver	Coins; medals; electrical; electronic devices; jewelry; silverware; photography; electronics water distillation; catalyst; mirrors; silver plating; table cutlery; dental, medical, and scientific equipment; bearing metal; magnet windings; brazing alloys; solder; catalytic converters; cell phone covers; electronics; circuit boards; bandages; batteries
Columbite-Tantalite	Tantalum	Electronic component (capacitors, circuitry), auto electronics, pagers, personal computers, telephones, superconductors, high-speed tools, catalyst, sutures body implants, optical glass, electroplating devices
Scheelite	Tungsten	Cemented carbide; cutting; wear-resistant materials; construction; metalworking; mining; oil and gas drilling; high-density electrodes; filaments; wires; electrical, electronic, heating, lighting, welding applications; steels; supperalloys
Magnetite	Vanadium	Alloys, iron and steel, sulfuric acid
Zeolites	Zeolite	Animal feed, cat litter, cement, water softener, purification, odor control, radioactive ions from nuclear plant effluent
Coal	Carbon	Thermal power generation, steel making, chemical, household coal briquettes, cement manufacturing
Barite	Barium	Oil well drilling, paper, rubber, cloth, ink, plastics, radiography, deoxidizer for copper, sparkplugs in alloys, white pigments
Clay	Bentonite	Floor and wall tiles, absorbent, sanitation, mud drilling, foundry sand bond, iron pelletizing, bricks, aggregate, cement, drilling mud, pet waste absorbent
Chromite	Chromium	Chemicals, ferroalloys, stainless and heat resisting steel products
Chalcopyrite	Copper	Construction, electronic products (cables and wires, switches, plumbing, heating), transportation equipment, roofing, chemical and pharmaceutical machinery, alloys (brass, bronze) castings, electroplated protective coatings.
Fluorite	Fluorspar	Ceramics, optical, electroplating, plastics industries, smelting, carbon electrodes, emery wheels, electric arc welders, toothpaste, paint pigment
Gold	Gold	Jewelry, dentistry, medicine, coins, ingots, scientific and electronic instruments, electrolyte
Halite	Sodium chloride	Human and animal diet, food seasoning and preservation, chemicals, ceramic glazes, metallurgy, curing of hides, mineral waters, soap manufacturing, water softeners, photography
Hematite	Iron oxide	Steels manufacture, vehicle auto parts, catalyst, medicine tracer element in biochemical and metallurgical research, paints, printing inks, plastics, cosmetics, paper dyeing, polishing
Spodumene	Lithium	Ceramics, glass, batteries, lubricating greases, rocket propellants, vitamin A synthesis, silver solder

(*continued*)

Mining, Table 1 (continued)

Mineral	Commodity	Uses
Mica	Mica	Paints, cement, agent, well-drilling muds, plastics, roofing, rubber, welding
Pentlandite	Nickel	Stainless steel, aerospace, transportation, fabricated metal products, electrical equipment, household appliances
Platinum Group Metals	Various	Control of car and plant emissions, jewelry, catalysts, chemicals pharmaceuticals, glass fibers, fiber-reinforced plastics, capacitors, electronic circuits, in dental alloys (e.g., platinum, palladium, rhodium, iridium, osmium, and ruthenium)
Pyrite	Iron sulfide	Sulfuric acid and sulfur dioxide; to recover iron, gold, copper, cobalt, nickel; to make jewelry
Rare Earth Elements	Various	Petroleum fluid cracking catalysts, metallurgical additives, alloys, glass polishing, ceramics, permanent magnet, phosphors (e.g., lanthanum, cerium, praseodymium, neodymium, promethium, samarium, europium, gadolinium, terbium, dysprosium, holmium, erbium, thulium ytterbium, and lutetium)
Trona and Nahcolite	Sodium carbonate	Glass, fiberglass liquid detergents, medicine, food additive, photography, cleaning
Palladium	Titanium	Pigments, producers, welding rod coatings, armor, chemical processing, marine, medical, power generation, sporting goods
Uraninite or Pitchblende	Uranium	Nuclear generation, medicine, atomic dating, powering nuclear submarines and other defense applications
Sphalerite	Zinc	Galvanizing, alloys, agriculture, chemical, paint, rubber
Limestone	Calcium carbonate	Quicklime, slaked lime building material, to purify iron in blast furnaces, glass manufacturing, cement and concrete
Oil and Gas	Various	Fuel energy, plastics, chemicals, electronics, clothing, medical and sports equipment, sports items, vehicle accessories



Mining, Fig. 1 Conceptual life cycle of a mine

laboratory analysis. Geological maps provide the basis for the location of future drilling campaigns. Due consideration must be given to the scale of mapping and the appropriate use of software for the production and interrogation of the maps produced. These may be large-scale (1:1,000, 1:2,500, or 1:5,000) or small-scale (1:100,000 or 1:250,000). Mapping techniques may vary from basic field observations combined with the use of a compass-clinometer and tape measure to the deployment of highly sophisticated mapping techniques such as LiDAR, InSAR, and GPS combined with GIS. The mapping coordinate system should be carefully considered as this varies across the world (e.g., Geographic, Cartesian, or Universal Traverse Mercator (UTM)). Geological mapping may also take place in exploratory trenches, open-pit mines, and underground mineworkings, often at mapping scales of 1:100 or 1:10.

Geochemistry

Systematic geochemical sampling of soils is undertaken to identify the relationship between mineralized rock and residual soil deposits. These surveys normally take place over large areas of many square kilometers, along streams and small rivers or in trenches to obtain bulk samples or chip samples. The sample numbers, locations, frequency, and time available will depend on the exploration budget and mineralization type. The geostatistical analysis of the soil chemistry and the resultant anomalies provide an indication of where the mineralization occurs, by comparison to the background values in unmineralized areas. Further and denser sampling may then need to be undertaken.

Geophysics

Geophysical surveys detect the differences in physical properties of the mineral deposit and the host rock. These may be deployed remotely, from the air or from the ground. The most suitable suite of geophysical techniques must be chosen to increase the probability of geophysical anomalies being detected. Geophysical surveys may occur at a single point, as a series of traverses along the ground surface or covering a designated area. Computer software and GIS can assist in processing and producing a 2D or 3D geophysical interpretation of the geology. The depth of the surveys can vary from less than 1 m to thousands of meters depending on the instrument and technique used. The choice of geophysical methods must be designed on the basis of an understanding of the geology. Ideally, geophysical surveys are deployed in a phased manner with successive focus on relatively smaller areas to more accurately delineate the mineral deposit. The appropriate choice in instruments will also depend on environmental considerations and in particular any geophysical "noise." All geophysical anomalies require validation by drilling and the use of borehole geophysical methods. Typical geophysical methods used in mineral exploration are examined in (Kearey et al. 2005).

Rotary Core Drilling

Rotary core drilling is required to delineate the mineral deposits and determine the size, geometry, grade, tonnage, and engineering characteristics. A preliminary and wider-spaced drilling grid is followed by a secondary and much denser array of boreholes. If required, advanced stages of drilling may take place (at the feasibility stage). Drilling provides an exploration and resource geologist with the opportunity to quantify the mineralization and the engineering geologist with the opportunity to evaluate the engineering characteristics of the minerals, host rock, and overburden rock mass (Fig. 2).

The engineering logging of the drill core takes place simultaneously with lithological, stratigraphic, and mineralogical logging. A degree of collaboration and coordination is required to ensure optimum data and observations are obtained from the drill core. Whereas the exploration geologist may be primarily interested in the drill core obtained from the mineralized areas, the engineering geologist will be interested in both the mineralized and barren areas. These

Mining, Fig. 2 Exploration drilling rig observations will be important to assist with the acquisition of geotechnical parameters for the design of the open-pit or underground mine. Alternatively, a designated number of targeted engineering geological (geotechnical) and hydrogeological boreholes may be required. It is important to note that the drilling of boreholes to obtain engineering geological parameters must include the orientation of the drill core to ensure the rock mass discontinuities are correctly aligned to represent in situ rock mass. Typical engineering geological investigations may include consideration of the following:

- Drilling parameters (e.g., type, layout, flush, setup, contractors, time of drilling, depth from, depth to, meters drilled, run length, core recovery, rock types drilled, length of drill, bit diameter, and internal barrel diameter).
- Core handing procedure including washing and transportation to the logging facility.
- Core logging facility and samples security.
- Engineering description and classification of lithologies.
- Core orientation (e.g., ACT II® Electronic Core Orienter).
- Rock quality designation (RQD).
- Fracture index (FI).
- Solid core recovery (SCR).
- Fracture frequency (FF) (0–30°, 30–60°, 60–90°).
- Intact rock strength (hardness).
- Geological strength index (GSI).
- Rock mass quality (Q-value).
- Discontinuities (type (i.e., joints, bedding, faults), classification, infilling mineralization, roughness, joint wall strength, veinlets, drilling-induced fractures, macro and micro-roughness, orientation, length, and azimuth).
- Degree and intensity of weathering and alteration.
- Determination of tectonic structure, rock mass structures, microfracturing, and fabric.
- *In situ* testing and monitoring boreholes may include the use of a high-pressure dilatometer (stiffness).
- *In situ* hydraulic fracturing (rock stress).
- Lugeon, slug, and pulse tests and the installation stand pipes and piezometers to monitor the groundwater regime.
- Cone penetrometer testing (CPT).
- Downhole surveying (diameter, trajectory, azimuth, and inclination).
- Geophysical borehole surveys might also be deployed to investigate faults (e.g., acoustic or optical televiewer, verticality, caliper, sonic, P-S suspension, natural gamma, gamma-gamma (density), neutron, fluid flow, and temperature logging).
- Core photography (with appropriate lining, color chart, and scale).
- Core logging (including a check that the markers in the core box correspond to the drillers' logs, date of logging; shift, drill runs "from" and "to," core recovery, lithological descriptions, rock types, rock descriptions including color, texture, structures, weathering, alteration, fractures and contacts, and mineralization type and percentage) and comparison with the drillers' logs.
- Lithological reference samples.
- Core logging databases and database management.
- Core splitting, sampling procedure, labeling, sample preparation for assays, or mineral/rock quality/grade.
- Long-term core and sample storage, chain of custody of samples, and security.
- Metallurgical and chemical laboratory test work schedule, controls, duplicate, and repeated samples.
- Geotechnical laboratory test work schedule (e.g., uniaxial compressive strength, triaxial strength, or sear strength).

Quality Assurance and Quality Control (QAQC)

Quality assurance and quality control (QAQC) peer review audits of mineral exploration are good practice. This enables the exploration and engineering geologists, client, and investors to have a high degree of confidence in the results. The purpose of the QAQC audit is to ensure that all procedures and assays are reliable, accurate (i.e., how close are the assays to the true metal content in the samples), precise (i.e., how repeatable are the values from the samples), and relevant to the exploration programs and mineral type.

Databases

Exploration, resource and geotechnical databases, and geographical information systems (GIS) are an important part of modern mineral exploration programs. They may include commercial or proprietary in-house exploration software. Databases enable efficient manipulation of huge volumes of data. Furthermore, databases facilitate geological interpolation and extrapolation, the interrogation of geospatially referenced different data sets, statistical analyses, calculation of resources and reserves, and the production of 2D and 3D geological cross and longitudinal sections through the mineral deposit. Validation of digital databases is important to reduce the risk for errors and to ensure the data are properly managed and suitably backed up and secured.

Geological Model

A geological model may be a fundamental requirement for the reporting of mineral resource and reserve. Modelling begins with the collation and review of historical and newly obtained exploration and engineering geological data. A database should then be constructed and validated to identify errors. Geological modelling provides the following:

- The cost-effective management of large volumes of data.
- The 2D or 3D spatial assessment and characterization of the mineral deposits.
- The interpolation of geological data between boreholes and Points of Observation (PoO).

- Stratigraphic modelling identifies the geometry and thickness of the mineral deposits or the top and bottom of each coal seam and each geological unit.
- Classification of resources into measured, indicated, and inferred (see below).
- Facilitation of audits and due diligence.
- Provision of the understanding of mineral properties for beneficiation and processing.
- Provision of information for mining engineers to assist with the design, planning, and scheduling of a mining operation.

Reporting of Exploration Results, Resources, and Reserves

In 1995, Bre-X Minerals Ltd., a major Canadian-based gold mining company reported a significant gold deposit at Busang, in Borneo, Indonesia. This announcement increased significantly the share stock price on the Toronto Stock Exchange (TSE). However, in 1997 the gold samples were investigated by an independent consulting company, and previously tested samples were found to be fraudulent. Alluvial placer gold grains that had been introduced were not consistent with the gold found in the host rock. This caused the collapse of the Bre-X share price and significantly upset the TSE.

Fraudulent claims, mining scandals, and corruption helped provide the basis for the development of international reporting standards to ensure consistency in the reporting of mineral resources and reserves, in a common style and language, across borders, and in a manner that is understandable by investors and other non-technical persons. Commonly used commercial, international reporting codes and standards have therefore been produced by professional organizations and/or government agencies. "Codes" must be followed, whereas "guidance" documents provide advice and information on best practice. Each standard generally defines resources and reserves (Fig. 3).

The first mining code was the "Code and Guidelines for defining and Classifying Mineral Reserves, Mineral Resources and Exploration Results," known as the "JORC Code". The JORC Code was originally published in 1989 and updated in 1992, 1993, 1996, 1999, 2004, and 2012 (JORC 2012). The JORC Code can be applied to all mineral deposit, and there is a separate JORC guideline document for coal. These guidelines contain definitions of various categories of resources and reserves, Points of Observations (PoOs) (e.g., boreholes, underground exposures, surface exposures, geochemical data), and recommendations for minimum spacings of PoOs for various categories of geological confidence (e.g., measured, indicated, and inferred). Generally, the lower the quantity and quality of data, the lower the classification. Fundamentally, JORC and other standards and codes and assessments must be carried out by an appropriately qualified and suitably skilled Competent Person (JORC Code) or "Qualified Person" (NI43-101) with a minimum of 5 years experience relevant to the style of mineralization and deposit type being considered and a member of a professional body that has an enforceable code of conduct. The principles governing the operation and application of the JORC Code are as follows:

- "Transparency": The provision of clear and ambiguous data and information.
- "Materiality": All relevant and reasonable data information are presented to allow informed judgments to be made.
- "Competence": Member of a professional organization and at least 5 years relevant experience.

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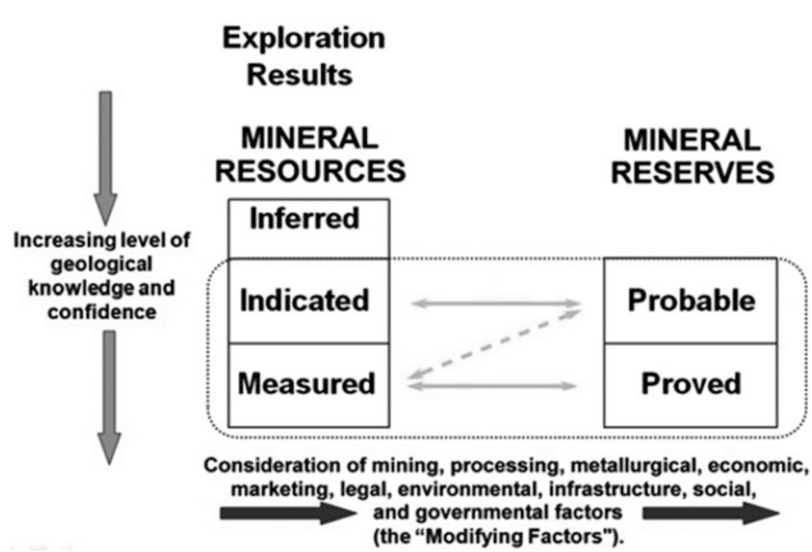
Mining, Fig. 3 Classification of Resources and Reserves (After CRISCO 2013)



JORC was followed by the publication of other international reporting standards including the following:

- South African Minerals Code (SAMREC) (South Africa).
- National Instruments NI 43-101 (Canada).
- AusIMM (Australia).
- Australian Institute of Geoscientists (Australia).
- The Pan-European Reserves and Resources Reporting Committee (PERC) (Europe).
- Alternative Investment Market (AIM) (London, UK).
- The Chilean Mining Code (Chile).
- Canadian Institute of Mining, Metallurgy and Petroleum (CIM) (Canada).
- The Institute of Materials, Minerals and Mining (IOM3) Code (UK).
- The Peruvian Mining Code (Peru).
- The Philippines Mining Code (Philippines).
- The South African Institute of Mining and Metallurgy (SAIMM) (South Africa).
- The Society for Mining, Metallurgy & Exploration (SME) (USA).
- US Securities and Exchange Commission (USA).
- GKZ State Commission on Mineral Reserves (Russia and CIS).
- The Polish Mining Code (Poland).
- The Ukrainian Mining (Ukraine).
- Chinese Resource and Reserve Classification (China).

The results from mineral exploration do not define a Resource or Reserve. This may be due to insufficient or noncompliant Points of Observation (PoO) and insufficient data to enable the tonnage, grade, or volume of mineralization to be determined.

Mineral resources are an *in situ* estimate of the grade and tonnes of mineralization with a realistic prospect of eventual economic extraction. Mineral resources can be demonstrated by technical evaluation and economic analysis to be likely mineable, treatable, and saleable.

Mineral reserves are derived by the application of "modifying factors." Whereby, the classification of a resource on a geological basis is converted to a reserve by prefeasibility and feasibility study that evaluates the "modifying factors" to demonstrate that mining could reasonably and economically take place at the time of reporting.

Some codes have different terms to describe "Resources" and "Reserves." For instance, the Ukrainian standard was inherited from the Soviet centrally planned economies where assessments were made by state-owned geological institutes. These methods were rule-based and prescriptive and different from many of the international reporting codes noted above. The Russian GKZ system reports Prognostic Resources (P1, P2, P3) from exploration results and deposit definition and Off-Balance Reserves and Balanced Reserve (C2, C1, B, A) which have increasing levels of geological knowledge and confidence. Industrial Reserves were considered available for mining, whereas Operational Reserves are minerals recoverable from the mining layouts. Note that these codes do not assess the different categories of Reserves depending on economic viability and the international term Resource is not recognized. The alignment between different codes may be complex, and it is not accurate to simply transfer a category of reserve from one of these systems into the resource and reserve classification of the JORC code.

There is no single international code for the global mining industry. In 1999, the Council of Mining and Metallurgical Institutions (CMMI) and United Nations Economic Commission for Europe (UNECE) agreed on the same definitions and the application of the United Nations Framework Classification for Solid Fuels and Mineral Commodities (UNFC). In 2006, the Committee for Mineral Reserves International Reporting Standards (CRISCO) became the successor to CMMI and published a template for the International Reporting Template for Exploration Results, Mineral Resources and Mineral Reserves. This is not mandatory, and it does not replace existing codes and standards but provides a basis for resource and reserves classification for countries that do not have existing codes or standards. It provides a generic international standard without the legal and regulatory requirements unique to each country. This is aimed at investors to help decide the viability of a future prospect and to assist with decision making for mergers and acquisitions (CRISCO 2013).

Scoping Study

A scoping study is carried out at an early stage to identify if a mining project may be technically feasible and economically viable. This may be used as a basis to attract funding and investment or for the purposes of obtaining an exploration license. The geology of the deposits and deposit type are characterized, and mining options are evaluated. In addition, hazards, constraints, and fatal flaws are identified as part of a risk assessment. A scoping study aims to have an accuracy of approximately +/− 30%, and the results determine if a prefeasibility should be carried out.

Prefeasibility Study

A prefeasibility study is carried out to assess the technical aspects of a mining project and to help investors or the mining company decide if a project is potentially profitable. Typically, a prefeasibility study evaluates the geology, mine design, mine planning, mineral processing, economic viability, risks and uncertainties, and whether the project can justify additional expenditure on mineral exploration, mineral processing, experimental test work, mine planning, and ultimately a feasibility study. A detailed geological assessment and resource estimate determine the quality and quantity of the mineral deposit. A recognized international resource reporting code or standard will be required. A 3D geological model, combined with geostatistical modelling, helps to define the resources. Other items that may be required to be assessed include the mining method options, mining scale, mineral processing, metallurgical and beneficiation requirements, preliminary civil engineering, geotechnical and infrastructure requirements, mining waste management, environmental impact assessments (EIA), social impact assessments (SIA), groundwater and hydrogeology, and financial and cost estimates. A Prefeasibility study aims to have an accuracy of approximately +/− 20% to 25%, and the results determine if a feasibility study should be carried out.

Feasibility Study

The Feasibility Study is sometimes also known as a Bankable Feasibility Study (BFS) (although this can be misleading as it does not actually mean the report is "bankable") or Definitive Feasibility Study (DFS). The objective of a feasibility study is to evaluate and assess the technical, economic, and environmental feasibility of a mine and to identify any fatal laws and constraints as part of a risk assessment. Essentially, a feasibility study should determine a project design and economic viability. The results of a feasibility study can also be used to provide confidence to investors and may have a role in equity raising and securing debt finance. The feasibility study can also be used to assess other projects on an equal basis. This focuses on the most technically and economically favorable option; it assesses all risks to the mining project, removes or reduces uncertainty, increases confidence for the investors and/or shareholder/stakeholders, and presents the data and information that can be audited as part of a due diligence. Ultimately, the feasibility study is aimed at demonstrating with a high degree of confidence that the mining project can be constructed in an economically viable and technically feasible way. The data and information contained in the feasibility study provide the basis for detailed civil engineering design of mine buildings and associated infrastructure. Finance may also be based on the findings of a feasibility study and can provide the basis for any permitting approvals or regulatory requirements. A feasibility study has an accuracy of +/− 10% to 15%. Engineering geologists are likely to be involved in the analysis and design of the following:

- Engineering characterization of drill core.
- Mine infrastructure, processing plant, and foundations.
- Pipeline and utilities routes.
- Haul roads, access roads, cutting, embankments, and retaining walls.
- Diggability and trafficability of large plant.
- Drilling and blasting requirements.
- Underground rock mechanics (roadways, shafts, adits, inclines, stopes) and strata control.
- Open-pit optimization, rock mechanics, soil mechanics, and slope stability.
- Tailings dams, sediment ponds, water supply reservoirs, and impoundments.
- Waste rock or colliery spoil tips.
- Heap leach pads.
- Evaluation of mining hazards.

Surface Mining and Quarrying

Minerals can be mined by underground or open-pit methods (Fig. 4) depending on the following (Fig. 5):

- Geological and resource factors: deposit type, depth, geometry, structure (faults, dykes, folds, and discontinuities), grade variations, surface constraints, *in situ* stress orientation, hydrogeology, and environmental impacts
- Mine factors: ventilation, geological hazards, geotechnical parameters, and engineering behavior of the rock mass or soil
- Other factors: development and capital costs, production costs, reserve recovery, assessment, socioeconomic or political issues

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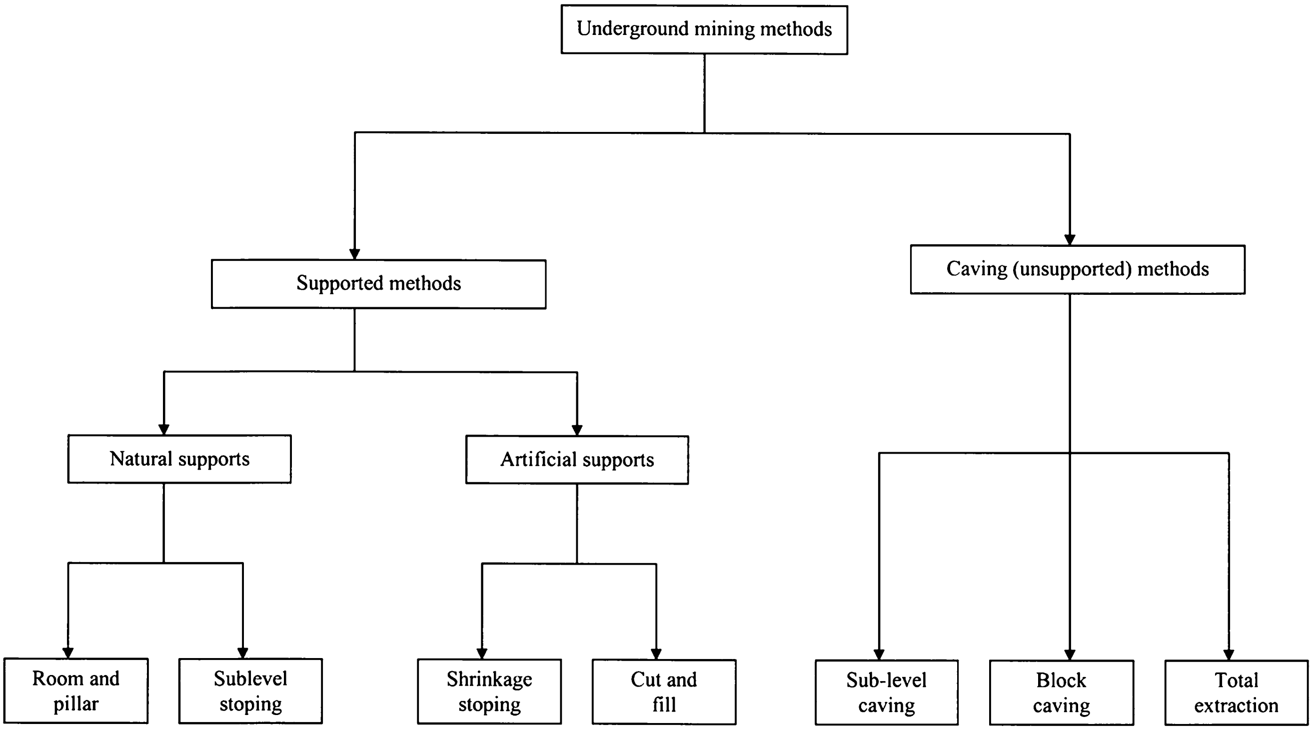
Open-Pit Mining and Quarrying

Surface mining (also known as open-pit or open-cast mining) and quarrying are used to extract stratified mineral resources that are close to the ground surface, for example, lignite, coal, metalliferous minerals, industrial minerals, and aggregates (Read and Stacey 2001; Smith and Collis 2001). A principal limiting factor is the stripping ratio (the amount of overburden to be removed) as well as the hydrogeology. If groundwater is present, dewatering will be necessary. The impact of dewatering on neighboring properties and the potential for the groundwater to be polluted should be assessed. Groundwater monitoring is required in boreholes during exploration to identify if the surface mine will extend below the water table. The stability of the open-pit walls, slopes, and benches require engineering geological evaluation with particular emphasis on the strength of the rock mass and the characterization of the rock mass discontinuities. The direction of dip and strike of strata in relation to the geometry and orientation of quarry and open-pit faces will also influence rock stability. These may also be investigated by the engineering geological logging of drill core during the exploration phase combined with slope stability analysis. If the strata daylight (dip into) the open pit/quarry, sliding failure may occur. Where the



Mining, Fig. 4 Mining methods (after Bell and Donnelly 2006)

Mining, Fig. 5 Typical open-pit and underground mine

strata dip steeply and cannot be worked against the dip, the face should be parallel to the bedding. The relationship between the angles of shearing resistance along a joint when sliding occurs under gravity and the dip of the joint will be relevant. Strong rock may fail suddenly and violently if the peak strength is exceeded on a high or steep rock face slope. Weaker rock, with marginal differences between the peak and residual strengths, fails gradually. The length and inclination of joints, friction and shear strength (frictional resistance), joint roughness, and moisture (effective stress) are relevant rock mass parameters for medium and high slopes, whereas unit weight parameter is important for small slopes. Where rock slopes become higher, the water pressure is relevant, and cohesion is less important. Some quarries may have a short-term and higher-working face and a shorter, longer abandoned face. Thin, intercalated beds of mudrock or shale and the rock mass discontinuities may also promote rock falls (e.g., ravelling, toppling, wedge, sliding, or planar failure). Loose overburden may fail by slipping or flowing into open pits/quarries by retrogressive rotational slip (Matheson and Reeves 2011). Active and abandoned quarries, including rock faces and tips, will be subjected to weathering, erosion, and failure and should be subjected to regular appraisals and assessments (Anon 1987, 2013).

Dimension stone may be worked by the drilling of closely spaced boreholes or the use of a mechanized diamond and tungsten carbide-impregnated chain saws, wire saw, or wedges to split the rock. Rotary percussion drilling and blasting are a common method for the extraction of metalliferous and industrial minerals and the removal of overburden. Usually, the open pit is operated from a series of benches that enable extraction and drilling and blasting to take place simultaneously. The fracture index and engineering properties of the rock help to determine the blasting fragmentation.

Contour Mining

Contour mining (also known as "mountain removal mining" or "mountaintop mining") is a coal mining method that involves the topographic removal or alteration of a ridge or summit. The entire coal seam is excavated by the removal of the overlying overburden. First, the overburden is removed by draglines or truck and shovel. Secondly, the uppermost coal seams are removed, and transported to the processing plant. The mining waste often may be used to infill adjacent valley floors (known as "holler fills" or "valleys fills"). Thirdly, draglines excavate the lower coal seams and intervening overburden, and these are placed into stock piles and valley fill, respectively. Finally, rehabilitation and vegetation take place. Due to deforestation and huge environmental impacts, this is a controversial mining method that takes place in Kentucky, West Virginia, Virginia, and Tennessee, in the Appalachian Mountains, USA.

Strip Mining

Strip mining is a type of open-pit coal and lignite mining, which starts by the cutting of an initial box-cut using face shovels. A dragline or bucket wheel excavator is then used to remove the overburden, which is subsequently stockpiled. In strip mining the waste overburden is placed by a dragline in parallel rows. Face shovels, dump trucks, and scrapers then work the exposed rock and associated minerals. The removed waste may be used as back fill, for rehabilitation, restoration, and landscaping as the highwall face advances. Mining wastes may be stored at their angle of repose if they are temporary slopes but may need profiling for longer-term stability. The failure of mining waste in 1996, at Aberfan, in South Wales, destroyed a school killing 116 children and 28 adults. The maximum depth for open-pit coal mining depends on several economic and geological factors, but it is in the order of approximately 100 m (although some mines exist that are deeper) and stripping ratios up to 25:1. Open-pit mines can be large with faces 3–5 km long. The reach of a dragline can be as much as 45 m and highwall faces may be 100 m high. The diggability of rock and method of excavation (e.g., digging, ripping, or blasting) can be determined by evaluating the unconfined compressive rock strength, rock mass discontinuities, block size (joint spacing and bed separation), and weathering (Scoble and Muftuoglu 1984), although other methods are also available using seismic velocities of the rock mass to determine diggability.

Auger Mining

Auger mining is a method for the extraction of coal by the boring of an auger into a coal seam. This is usually associated with open-pit or strip mining when it becomes uneconomic to recover coal due to excessive depth and overburden removal. The use of an auger is controlled by the dip of a seam and the engineering properties of the coal. Auguring requires a coal seam to be horizontal or slightly dipping and to have reasonable strength.

Highwall Mining

Highwall mining, like auger mining, is associated with open-pit coal mining, whereby a seam of coal is excavated at outcrop. This method uses a continuous miner operated by a hydraulic boom with a shearer and cutter-head boom that can be extended for approximately 300 m into a coalface.

Alluvial Mining

Placer minerals, such as gold, platinum, tin, diamonds, gemstones, ilmenite, zircon, and monazite, are concentrations of minerals that have been eroded from source and then deposited elsewhere by gravity. These are common in alluvial, fluvial, lacustrine, glacial, glacio-lacustrine, aeolian, eluvial, and residual sedimentary settings. Alluvial mining of placer deposits involves the surface excavation of loose sediments from stream, river, lake, estuary, or beach deposits. This can include small-scale panning or the use of a sluice box to trap heavy minerals. Alluvial mining can have severe environmental consequences since large volume of silt, sand, or gravel often may be processed to recover the economic minerals (Fig. 6).

Dredging

Dredging is used to recover relatively dense minerals such as cassiterite, rutile, ilmenite chromite, and scheelite that are under a natural or artificial body of water by the use of a floating vessel, with associated processing equipment, sumps, waste disposal, and storage facilities. Alternatively, the excavated sediments can be processed onshore.

Sand, Gravel, and Clay Pits

Sand, gravel, and mudrocks are excavated from surface workings for the production of building and civil engineering raw materials (e.g., aggregates, tiles, sewer drains, bricks, mortars, cement, plaster, and renderings). The deposit commonly occurs as river, beach, marine, glaciofluvial, and aeolian deposits. The degree of sorting, composition, and characteristics can be variable depending on the depositional environment. Bimodal, glaciofluvial deposits contain gravel and sand

Mining, Fig. 6 Alluvial placer mining for gold

grade materials. Some alluvial gravel may contain detrital carbon (coal), shale, shell fragments, or deleterious salts that could be detrimental to the desired products, whereas aeolian deposits tend to be well sorted.

Clay used for brickmaking must have suitable chemical and physical properties including moderate plasticity, suitable workability, high strength when dry, low shrinkage, long vitrification, and low sulfide minerals. The environmental impacts of working sand and gravel must be carefully considered, for instance, to ensure the removal of the deposit doesn't cause or exacerbate flooding or coastal erosion. The intact strength, bulking factor, and natural moisture contents will influence diggability and the types of mining equipment. These factors may be determined during exploration by engineering geological evaluation and geophysics. The stability of these deposits needs to be estimated to assist with the design of the pit walls. Displacement takes place when the shear stresses exceed the shear strength of the deposits. Generally, slope angles of between 35° to 45° tend to be used in sand and gravel pits and 30° and 45° in clay soils, but this depends on the deposits' (soil) engineering characteristics and geomechanical behavior. Mining may take place by the use of a dragline or mechanical excavator that loads the materials onto trucks or a conveyor for sorting, washing, and grading to produce finer and coarse grades. Flooding is often required to be controlled, and upon abandonment sand and gravel pits may be rehabilitated to form artificial lakes and wetlands.

Underground Mining

Underground mining methods may be classified depending on whether supports are required. Where supports are used, these are further categorized if rock or artificial support is installed. Some of the main underground mining methods are described below (Hustrulid and Bullock 2001).

Artisanal and Small-Scale Mining

Artisanal mining includes both surface and underground operations. Typically, this takes place in some less developed countries (Fig. 7). There are an estimated 13 million people involved in artisanal and small-scale mining (ASM), in approximately 30 countries, and up to 100 million people are dependent on ASM (Hentschel et al. 2003). Minerals that are extracted by ASM include gold, silver, zinc, tin, copper, precious and semiprecious gemstones, coal, bauxite, limestone, and other industrial minerals. ASM often takes place with the following characteristics:

- No or little appreciation of the geology.
- No or poor prospecting and exploration strategies.
- No or low mechanization.
- Labor intensive.
- Insufficient regulations and policies.
- Low levels of health and safety.
- Unskilled and inexperienced work force.
- Rudimentary mining and mineral processing techniques.
- Low productivity and low salaries.
- Non-continuous mining controlled by seasons, weather, and clime or market conditions.
- Poor capital investments.
- Little regard for the environment and future sustainability of the mineral resource.

ASM may involve vulnerable members of society, including children, women, and the elderly. What is more, ASM may operate outside the law and may be associated with fraud, corruption, criminal gangs, land and mineral right conflicts, and the displacement of local tribes and indigenous people. ASM can lead to adverse environmental damage including pollution of surface water courses and aquifers, contamination, unstable waste, subsidence, fires and spontaneous combustion, and dereliction. ASM can result in mineral

Mining, Fig. 7 Artisanal mining in Colombia

resources depletion or lead to the sterilization of some deposits since it is not possible to accurately quantify resources.

Outcrop Workings

It is likely that minerals were originally mined where they cropped out. This was almost certainly the case in historical times when the target mineral was observed at or near the ground surface. Early surface workings were probably rudimentary with little or no understanding of the geology, groundwater, roof support, and ventilation.

Drifts

When mineral extraction becomes too deep, adits are driven into hillsides until rock failure or lack of ventilation prevents further advancement. By comparison, modern drift mining may be highly mechanized and may be horizontal, inclined up to approximately 45°, or helical.

Soughs

Soughs are horizontal tunnels excavated to drain groundwater to enable mineral resources to become mined. In the UK, soughs date back to the fifteenth century where rudimentary pumping methods were developed using, for example, horses and simple winding mechanisms.

Bell Pits

Bell pits, although still used in some less developed countries, were a historical mining method that started in parts of Europe in the thirteenth or fourteenth century (possibly earlier) to extract mineral deposits that were located no deeper than approximately 7 m below the ground surface. Coal and chalk were commonly mined using bell pits, the latter known as “deneholes.” Fires were used for ventilation circuits, although this caused explosions in some coal mines. Bell pits were commonly used to extract coal, sandstone, flint, clay, and semiprecious and precious gemstones. A vertical shaft was excavated, and then the bottom of the shaft was extended once the mineral had been intersected. An adjacent bell pit was sunk, at a distance of up to about 10 m, once the mineral had been exhausted and within the limitations of the roof support and ventilation. Bell pits may be exposed in modern-day open-pit workings.

Partial Extraction: Room and Pillar Workings

Stratified mineral deposits can be mined by partial extraction methods where part of the mineral is sacrificed and left in the mine to support the roof. Room and pillar workings (also known as pillar and stall, post and stall, stoop and room, and bord and pillar) replaced bell pits and comprised a series of radiating rooms, supported by *in situ* pillars. Historical room and pillar workings were unplanned. Often, mine abandonment plans do not exist and the location, depth, and geometry of old mine may not be known. Support pillars may vary considerably in size and extraction ratios vary from 30 to 70%. Unmined minerals may also have been left in the roof and floor to promote stability of the voids. Groundwater is managed by pumping or natural drainage. Two or more shafts were constructed to facilitate ventilation along with underground fan pit installations and to provide alternative means of access and egress. Waste rock may have been stored in the mine workings (stowing) to reduce the labor time and costs of surface disposal and to prevent subsidence. Stress concentrations on old pillars may cause these to fail, which can trigger collapses of adjacent pillars. In other situations pillars may have been "robbed" at the closing stages of mining which can influence the stability of mine roof and increase the possibility for future subsidence. Pillars may also become destabilized by mine water ingress. Where the floor strata are weak, pillars can punch into a weaker floor, causing ground subsidence. This may generate collapses that can chimney to the ground surface to generate a surface collapse or crown hole (Piggott and Eynon 1978). Collapse of these mine workings and void migration will be determined by the unsupported span width, extraction height, and engineering properties including shear strength and discontinuities in the cover overlying strata, dip, depth of superficial deposits, and groundwater regime. Modern room and pillar mines for halite, gypsum, limestone, chalk, and sandstone may be of considerable size, systematic, and mechanized.

Early Longwall Mining

An adaptation of longwall mining developed in the Midlands coalfields in the UK during the 1600s (Shropshire Longwall) consists of two parallel roadways (gates) driven about 10 m apart to excavate a coal seam. The gates advanced as the coalface was mined, and these also permitted labor, equipment, and a ventilation circuit to be established. Firstly, the bottom of the coal seam was undercut using hand-held tools and using timber support. This was followed by a vertical cut. Iron wedges inserted into the top (later replaced by explosives) of the exposed seam allowed several meters of the seam to drop, where it could be manually loaded into wagons for transportation along the gates to the surface. This method of working generated subsidence and influenced groundwater in overlying aquifers.

Modern Longwall Mining

Mechanized longwall mining developed in the deep coal mines of Europe from around the early to middle part of the twentieth century. Two parallel roadways, some 200–300 m apart, were excavated for a distance of up to several kilometers. These were joined at rights angles to form a working face and to provide ventilation. The coal extraction was automated using a rotating armored plated coalface shearer or plow where the seams were thin (less than ca. 1 m) and which cut the entire length of the face in a single run. The roof was supported by hydraulic props which advanced causing the roof behind to collapse into the goaf, reducing the compressive stress on the operating face. The excavated coal fell onto conveyor belts that transported the coal to the surface for processing. In retreat longwall mining, the two roadways were driven along the full extent of the seam being extracted, which ensured there was geological continuity, before the shearer then extracted the intervening coal. Longwall mining generates subsidence, influences aquifer permeability, and produces large volumes of waste that were disposed as spoil heaps. Longwall mining was not permitted beneath some water bodies unless there was at least 105 m of rock cover between the roof of the mine and the water bed (Whittaker and Reddish 1989).

Shortwall Mining

Shortwall mining uses narrow panels, up to approximately 45 m long, and a single roadway in an attempt to control subsidence. Modern shortwall mines have fully automated face equipment, ventilation systems, and gas monitoring devices. Due to the short faces, the goaf and any support pillars may not completely converge with the floor leaving voids.

Wongawilli Mining

The Wongawilli mining method was developed in Australia for the extraction of coal. This comprises two or three headings that divide the coal panel into smaller panels. The panel may be up to 1 km long and up to 200 m wide and are excavated using mechanized coal cutting, support, and transportation systems.

Sublevel Stoping

Sublevel stoping is a method of mining for metalliferous deposits. Various levels are driven into the zone of mineralization, and then the rock is removed between these levels by blasting and gravity, before being removed to a haulage level. In cut-and-fill stoping, horizontal slices of the ore body are excavated in the stope and waste rock is used to backfill the void. Each slice is driven into the footwall, with crosscuts, and the rock is systematically removed and transported to the ground surface for processing. Hydraulic backfilling may take place as the mining advances, controlled by dams and barricades. This type of mining must consider the engineering properties of the host rock and the groundwater to ensure failure and folding do not occur.

Shrinkage Stoping

Shrinkage stoping shares similarities with the cut-and-fill mining method, but the broken rock is not extracted and remains in place to support the walls and to provide a working floor to enable the upward progression of the roof.

Sublevel Caving

Sublevel caving methods allow the ore body and host rock to cave into the space generated by the extracted mineral. This caving takes place in a controlled manner and cannot be used in areas where subsidence is prohibited or there are aquifers. A network of shafts, development drifts, crosscuts, and haulage levels develop a mine complex. This mining method takes place in steeply dipping or vertical mineral deposits.

Block Caving

Block caving is used to mine massive, steeply dipping or tabular dipping mineralized rock. A large rectangular block is drilled, blasted, and undercut to cause the block to fail. The mineralized rock and waste collapse before being removed via chutes, drawpoints, and crosscuts to a haulage level. The subsided ground may contain scarps, fissures, and broken and loose material.

Solution Mining

Soluble rocks, such as halite, can be mined with injection of water and solvents in lined and cased boreholes to dissolve the salt. Brine returns to the surface and the salt is extracted. The size and geometry of the voids created are controlled to prevent collapse and subsidence.

Unconventional Mining

Some mining methods require the development of new and innovative methods of mineral extraction. This includes for example:

- Shale gas: The extraction of natural gas found in shale. To extract the gas, deep and horizontal boreholes are hydraulically fractured to enhance the permeability of the shale enabling the gas to flow for extraction.
- Coal mine methane (CMM): The abstraction of methane gas, via boreholes, that has accumulated in voids associated with past and abandoned coal mines.
- Coal bed methane (CBM): The abstraction of methane gas, via boreholes, in areas of unworked coal.
- Underground coal gasification (UCG): The extraction of syngas from coal seams by the underground controlled ignition of the seam via boreholes.
- Heat pump technology: The use of mine water in shallow, abandoned, flooded mine workings to heat or cool buildings.

Mine Ventilation

The manager of an underground mine must ensure that all parts of a mine are adequately ventilated. A Mine Ventilation Officer may be appointed by the Mine Manager, although this does not relieve the manager of any statutory duty. Ventilation must dilute or remove flammable and noxious gases so that they are harmless and do not exceed safe operational limits. These include, for example, methane (CH_4), radon (Rn), carbon dioxide (CO_2), carbon monoxide (CO), hydrogen sulfide (H_2S), oxygen (O_2), and stythe gas (air enriched in nitrogen and depleted in oxygen). Air entering a mine is subsequently distributed within the mine complex via a network of internal raises, ramps, steel ducting, and regulators to generate auxiliary ventilation systems. Exhaust systems draw out the contaminated air from the mine. Ventilation cells in the mine must provide no less than 19% oxygen (by volume) in the general body of the air. The mine ventilation system must also ensure that the mine atmosphere is reasonable in terms of the temperature, humidity, and dust. Miners may be evacuated if methane levels exceed 2%, shot firing may not proceed, and electricity supplies might be closed down. Methane levels become explosive if there is a source for ignition in concentrations of 5–15% by volume. In the UK, ventilation in mines may be provided by exhaust systems as this maintains the pressure inside a mine at a lower level than outside. Therefore, if the fans ceased operating, fresh air would be sucked into the mine. In coal mines, coal is usually wound up the downcast shaft to prevent a plug of methane following coal travelling along a conveyor, although this differs around the world. Drill holes may be drilled into the roof of some mines to control hazardous gas accumulations. Automated gas monitoring systems are also used to replace conventional methods of gas detection such as the yellow canary or Davy lamp.

Open-Pit Stability, Tip Stability, and Strata Control

Strata control considers how the strata may be controlled in the direct vicinity of the mine opening, including mine shafts, roadways, stopes, and operational faces. However, mining subsidence is generally not considered (see section on ► "Mining Hazards"). Engineering geologists, geotechnical engineers, and mining engineers will be involved in the evaluation of how the rock mass may behave during and following underground excavation and what methods and techniques may be available to cost-effectively prevent and control strata failure around the mine openings. Parts of the mine may require stabilization for a specified and limited time period, such as the roof and goaf in a longwall coal mine, whereas roadways and shafts bases may require longer-term stability. Generally, the engineering properties of the strata will require assessment, including the rock mass characteristics, density, tensile and compressive strength, Poisson's ratio, Young's modulus of elasticity, geological structure, and in particular the rock mass discontinuities, the three-dimensional stress field, and groundwater. Where these parameters cannot be measured in laboratory, by *in situ* tests or by monitoring or modelling, it may be necessary for these to be conservatively estimated.

Rock mass classification methods may be used to assist with stability analysis in mining and tunnelling. Since 1946, these have been used to evaluate the empirical relationships between rock mass parameters and the engineering design. Several different rock mass rating systems are available including rock mass rating (RMR), Q-system, mining rock mass rating (MRMR), and New Austrian Tunnelling Methods (NATM), and others exist for rock slopes (Bieniawski 1989; Fig. 8).

Conventional support methods include *in situ* pillars or timber props and insets. Steel arches with metal lagging may be used in roadways, which may be semicircular, combined radius arch, or Gothic arch. Hydraulic face supports move forward as a longwall face advances. Steel or fiber glass roof bolts are increasingly used in mines as a cost-effective and efficient method of strata control.

Mine Closure and Abandonment

Mines can close due to a variety of reasons. These include geological constraints, mineral(s) becoming exhausted or uneconomic, reduced demand for the mineral product, and political or environmental reasons. Some mines close and then reopen at some future stage perhaps under more favorable economic conditions. Interestingly, some mines remained open in the former Soviet Union even though there was no or little demand for their products and they were working at an economic loss. This was driven by the fact they sustained the lives of several thousands of people who lived in the mining towns in remote and challenging geographical locations. Mine closure can have an inevitable destructive and detrimental impact on the local or national economy and the society which was dependent on the mine, sometimes leading to a "ghost town." Some old mining sites have historical and archaeological value or contain protected flora and fauna (e.g., kestrels nesting on quarry faces, bats in mine entries, or rare plants on mineral waste tips).

Mining, Fig. 8 Engineering characterization and geological logging of drill core for the design of an open-pit mine and underground (L J Donnelly and WAI)

Serious environmental consequences can also be caused by mine closure. The extent of the impact will depend on the mineral worked, the number of years over which mining took place, processing and beneficiation, the management of mining waste, and so on. Today, mining projects consider the environmental and social impact of mining well in advance of any mine closure proposals; this can be undertaken as part of an Environmental Impact Assessment (EIA) and Social Impact Assessment (SIA) usually at the prefeasibility and feasibility stages of the project.

The planned and phased closure of a mine may assist to ensure the mine is left in a secure state and the associated hazards, constraints, and social and environmental impacts are properly addressed, managed, and mitigated. If mine closures are appropriately implemented, the maximum value of the land and future redevelopment options can be evaluated. The planning for mine closure will require due consideration of several multidisciplinary issues such as geological, technical, environment, financial, legal, and socioeconomic. The mine closure plan should commence well in advance of the actual closure of the mine taking place and to ensure there are sufficient financial resources to enable the liabilities, hazards, and associated geotechnical risks to be evaluated.

Engineering geological investigations determine the treatment and long-term stabilization of mine entries (shafts and adits), support pillars, roadways, subsidence prevention, and the stability of rock slopes in open-pit mines and on spoil tips. A hydrogeological evaluation of mine water, mine water rebound and mine effluent discharges may also be required.

In some countries there exist environmental legislation and regulations to identify the responsibilities of the mine owners, operators, investors, and stakeholders. An environmental audit is recommended before mine closure is finalized. This should include a review of the mine infrastructure, buildings, mine roadways, shafts, utilities, spoil, dams, lagoon, and hazardous substances (e.g., explosives, polychlorinated biphenyls (PCBs), fuel, asbestos, and grease). Monitoring could be required for surface effluent, mine water, sewage, mineral processing sites, noise and dust, etc. A mine closure plan should also consider the salvage, reuse, or sale of mining equipment and machinery and considerations of a potential buyer for the abandoned mine. If the decision to close a mine has not been finalized, it may be possible that the mine goes into a period of "care and maintenance" before the cessation of mining operations.

New mines can be controversial, with debates often focusing on the need to generate income, provide resources, and maintain people's livelihoods and the environmental impact of the proposed mine. Mining is often not permitted or strictly regulated and monitored in some locations, such as national parks and areas of outstanding national beauty.

Old mine workings are normally considered to be a liability, but many are protected sites of heritage, archaeological, or geological value. The alternative use of old mines could make them a commercially viable asset. This will depend on the geology, hydrogeology, mine geometry and characteristics, and desired end use. Examples of the possible uses of old mine workings are as follows:

- Old, large, dry, room and pillar mines can be used for storage (e.g., oil, gas and chemicals, data centers, and document) (e.g., Cheshire salt mines, UK and Argentina limestone mine, Kansas City, USA).
- Secure military facilities (e.g., Corsham, UK, now decommissioned, West Poland mines, and Iron Mountain, Pittsburgh, USA).
- Low-level radioactive waste (e.g., salt mines, Hannover, Germany).
- Munitions storage (e.g., Burton, UK).
- Storage of mining waste from coal-fired power plants and mine workings (e.g., Romania, Hungary, and Czech Republic).
- Storm water channels in major cities (e.g., parts of Glasgow, Scotland).
- Storage of foodstuffs (e.g., Carrickfergus, Northern Ireland).
- Scientific research in a controlled atmospheric environment (e.g., abandoned gypsum and anhydrite mines (Midlands, UK).
- Tourism and museums (e.g., National Coal Mining Museum and Sygun Copper mine, UK).
- Heat pump technology from shallow, abandoned, flooded coal mine workings (e.g., parts of the UK).
- Unconventional and alternative energy (e.g., abandoned mine methane).
- Landfill and domestic waste storage (e.g., waste management in Europe).
- Aquatic and wildlife centers (e.g., flooded sand and gravel pits and quarries in Europe).
- Theme parks and leisure facilities (e.g., Alton Towers, Midlands, UK).
- World-class floral garden facility (e.g., Butchart Gardens, British Columbia, Canada).

Abandoned mines can be rehabilitated or restored. Landscaping reduces the visual impact of former mine sites including the profiling of waste tips and landform replication. Embankments and bunds can be constructed and vegetation reestablished, often using the mine waste. Where mine workings exist beneath urban areas or sites where new infrastructure or housing is proposed to be developed, it is important for the hazards and risks to be assessed. When required, voids and shallow mine workings may need to be stabilized by grouting or capping of any mine entries. The potential impact of mining includes the following (see chapter on ► "Mining Hazards") (Bell and Donnelly 2006).

- Disruption to land.
- Alteration of the topography.
- Adverse effects on groundwater.
- Mine water rebound.
- Effluent discharge from mines and acid mine drainage.
- Explosive, toxic, or asphyxiate gas emissions.
- Surface water pollution.
- Derelict land and contamination.
- Mining waste and spoil heaps blighting the landscape.
- Damage to vegetation.
- Spontaneous combustion and fires (coal).
- Abandoned mine entries (shafts, adits, and inclines).
- Mining subsidence and fault reactivation.
- Induced seismicity.
- Noise.
- Dust.

Mine Entries: Shafts, Inclines, and Adits

Modern mines must have two means of access and egress. It could be two drifts, a drift and a vertical shaft, or two vertical shafts. Vertical shafts or horizontal entry adits and inclines are required to provide access to mine workings, provide ventilation, connect mining levels, for mining infrastructure (e.g., fan house, shaft balance weights, windings engines and pumps to remove mine water), enable the supply of labor and equipment, and remove waste and minerals. The age, width, shape, depth, and lining of shafts may vary considerably. Old shafts may be unlined or lined with wood (tubing), brick, or masonry. In the seventeenth and eighteenth century, cast iron rings and rivets permitted shaft construction through water-bearing strata. Since the start of the twentieth century, shafts became lined with reinforced cylindrical concrete rings, pre-cast concrete or steel segments, and/or shotcrete to provide stability and control groundwater ingress where there was a high hydrostatic pressure or high lateral pressures. The geological factors influencing shaft construction include the engineering properties of the rock mass, groundwater, *in situ* stress, and ventilation. The demand for coal during the European Industrial Revolution and the invention of water pumping methods in the eighteenth century (e.g., the Newcomen atmospheric engine in 1712) enabled mineshafts to be constructed to greater depths. In the twentieth century, the development of ground freezing techniques enabled deep shafts to be constructed through aquifers. The shape of mine shafts can be round, oval, square, and rectangular, and their depths vary from less than 10 meters to thousands of meters deep.

Upon abandonment mine shafts may have been backfilled but this was not always the case. The engineering characteristics of backfill materials may be unknown and could be hazardous (e.g., agricultural, industrial, chemical, animal, military, or domestic waste or old mining equipment), explosive, or radioactive. Backfilled materials may move into radiating mine roadways or consolidate under their weight; however, this will be influenced by the density of the material, friction against the shaft lining, and large obstacles in the shaft that cause blockages or arching. Some shafts may have a wooden staging or plug such as a tree, timber, reinforced concrete, or old mining equipment. These often deteriorate over time also leading to collapse and subsidence. Shafts may contain spoils where they have subsided leading to a depression, and this can provide evidence for their existence. Shaft caps include railway sleepers, concrete slabs, brick, or "beehive" masonry domes, or they may be fenced with signage to prevent accidental access (Fig. 9). Metal grills may also be laced over mine entries for ecological reasons (e.g., if they contain bats). Some mine shafts may require to be ventilated to prevent gas accumulations. Where a shaft has been levelled, there may be no visible topographic expression, and their locations often may be unknown.

Hazards associated with abandoned mine entries include subsidence or collapse, gas emissions, and mine water discharges. In areas where shafts and mine entries are suspected, these must be located (e.g., by geological mapping, geophysics, trenching, or drilling) and treated (e.g., by grouting, backfilling, and capping). Industry best practice guidance is available for the assessment and mitigation of mining hazards including the backfilling and capping of mine entries (CIRIA 2018).

Mining Records

Mine plans and reports might be available in some countries and held by a government, state, or federal mining organization, geological survey, ministry, or chamber. For example, in the UK, it became a statutory obligation for mine plans to be produced and submitted on abandonment of a coal mine, but this was not compulsory until 1872. In 1911, the legislation required coal mine owners to maintain accurate plans of a specified scale and revise them every 3 months. However, this was not the case for owners of metalliferous mines. Mine abandonment plans and mining records cannot always be relied on for accuracy, and many mines are unrecorded.

Summary

Mining has taken place throughout the world for thousands of years. The minerals and commodities produced by mining have influenced past civilizations and are likely to continue to remain vital to sustain future generations in both developed and less developed countries. Globally, modern society, economies, and industries are reliant on minerals produced by mining. Mining takes place by underground or surface operations. Mining is highly variable ranging from low investment, poorly planned, small-scale artisanal mines with rudimentary mining methods to highly mechanized,

Mining, Fig. 9 Mine shaft covers including steel beehive, steel grid, concrete cap, and masonry beehive

sophisticated, and complex mining operations. Engineering geologists may be involved in most stages of mining and tend to be most involved in exploration and the logging of geotechnical boreholes, mine design (open-pit stability and the construction of tailings dams and lagoons, stopes, roadways, and portals), infrastructure development (utilities, haul roads, embankments, settings, bridges, tunnels, mine buildings, reservoirs, sewers, processing plant, airports, etc.), mine abandonment, and mining hazards assessments.

Engineering geologists should have a good appreciation of the elements of a mining cycle. The life cycle of a new mine may follow a phased process including, for example, a desk study, scoping study, exploration program, due diligence audits, resource and reserve reporting, prefeasibility and feasibility study, mining operation, mine closure, and abandonment. Often significant financial investments are required to develop or extend a mine over periods from a few years to several decades. Each stage increases geological and geotechnical knowledge and stakeholder confidence and reduces technical, financial, and environmental risks to determine whether there is a realistic and reasonable prospect for eventual economic mining.

Firstly, licenses and rights to explore minerals have to be secured and agreed with the appropriate land and mineral owner or government organization/agency. This is followed by the design, management, and implementation of an exploration and geotechnical program to locate and accurately determine the geometry, size, depth, quantity, quality, and tonnage of the mineralization and to characterize the engineering properties and hydrogeology of the deposits and associated unmineralized rock mass. Geological modelling techniques with integrated geostatistical analysis and geographical information systems facilitate the analysis of historical and newly acquired data. The resultant geological models and mineral resources and reserves may have to be produced compliant with international reporting standards.

Minerals are mined by underground and/or surface methods. This depends on several interrelated factors such as the geology, depth, and geometry of the mineral deposit and engineering behavior of the mineralized and unmineralized host rock and the groundwater regime and relative costs. Surface methods include quarries for industrial minerals; pits for clay, sand, and gravel; and open-pit (open-cast) operations for coal. Underground mining methods historically began from shallow adits and bell pits, later advancing to rudimentary room and pillar workings. Modern underground mines include mechanized room and pillar workings, longwall mining (mainly for coal), and a variety of cut-and-fill or stoping methods (mainly for metalliferous minerals). Once extracted, minerals are processed and beneficiated to remove the waste or provide the concentrate for advanced refinement or mineral commodity (this extends beyond the scope of this chapter).

Engineering geologists may be required to evaluate the impact of mine closure or abandonments. This includes the identification, management, investigation and mitigation of any liabilities and mining hazards, and assessment of their consequences and associated geotechnical and environmental risks (see chapter on ▶ "Mining Hazards").

Cross-References

▶ Acid Mine Drainage
▶ Blasting
▶ Boreholes
▶ Borehole Investigations
▶ Classification of Rocks
▶ Classification of Soils
▶ Cut and Fill
▶ Dewatering
▶ Drilling
▶ Engineering Geology
▶ Engineering Geomorphology
▶ Engineering Properties
▶ Environmental Assessment
▶ Failure Criteria
▶ Faults
▶ Geohazards
▶ Geology
▶ Geophysical Methods
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▶ Groundwater
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▶ Grouting
▶ Hazard
▶ Hazard Assessment
▶ Hydrogeology
▶ Hydrothermal Alteration
▶ Induced Seismicity
▶ Landfill
▶ Landslide
▶ Mine Closure
▶ Mineralization
▶ Mining Hazards
▶ Modelling
▶ Modulus of Deformation
▶ Modulus of Elasticity
▶ Mohr Circle
▶ Mohr-Coulomb Failure Envelope
▶ Poisson's Ratio
▶ Rock Bolts
▶ Rock Field Tests
▶ Rock Laboratory Tests
▶ Rock Mass Classification
▶ Rock Mechanics
▶ Shotcrete
▶ Soil Field Tests
▶ Soil Laboratory Tests
▶ Soil Mechanics
▶ Soil Nails
▶ Soil Properties
▶ Stabilization
▶ Strain
▶ Strength
▶ Stress
▶ Subsidence
▶ Subsurface Exploration
▶ Tunnels
▶ Waste Management
▶ Young's Modulus
▶ Zone of Influence

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Mining Hazards

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Synonyms

Mining catastrophes; Mining disasters

Definition

A mining hazard is a source of danger or potential danger, caused by the working of minerals, which has the potential to cause harm to life or damage to a mine, infrastructure, utilities, land, or the environment.

Environmental Impact of Mining

Mining has generated considerable wealth, reduced poverty, and improved the quality of peoples' lives by the provision of natural resources. Modern society would not function as we know it without mining and mineral commodities. As such, minerals are crucial to society and are likely to remain so into the foreseeable future. However, mining can have a detrimental impact on people, society, and the environment. These impacts manifest throughout the world as mining hazards (Bell and Donnelly 2006; CIRIA 2018; Culshaw et al. 2000; HSE 2014).

Mine Entries

Mine shafts and adits facilitate the extraction of mineral deposits. They provide access, ventilation, remove minewater, waste rock, mineralized rock (ore), and interconnect different parts of a mine. The sinking of mine entries goes back several thousands of years and continues today. In some areas, there may be no requirement or legislation to record the position of mine entries. As such, many mine entry locations remain unknown, until they collapse. In the UK alone, there are an estimated 170,000 recorded coal mine shafts and as many unrecorded mine entries are suspected to exist. This excludes those shafts related to the mining of metalliferous deposits and industrial minerals.

The principal hazards associated with mine entries are: sudden and catastrophic collapse of the ground to generate a crown hole (Fig. 1), mine gas, or minewater discharges. Mine entries, if suspected, must be avoided or located and treated before new structures are built or the sites are redeveloped. Mine entries can be located as part of a ground investigation using geological mapping, evaluation of air photos, drilling, trenching, and geophysical surveys. Mitigation or stabilization options typically include the establishment of a safe stand-off-zone with secure fencing and signage, backfilling, plugging, or capping. Mine entries and shallow workings can be investigated by sophisticated 3D laser scanning; this provides information on their extent, geometry, and condition (Fig. 2).

Mining Subsidence

Subsidence is associated with mining. As noted previously, crown holes are a type of subsidence generated by the collapse of mine entries, shallow partial extraction (room and pillar) workings, mine roadway intersections, or the collapse of the ground into an underground mine void.

Trough subsidence is different and is caused by longwall mining where the ground is encouraged to subside to relieve strain accumulations on the working face. The concepts and principles of mining subsidence are well understood. These have been developed and investigated since at least the nineteenth century (Whittaker and Reddish 1989). Structures, utilities, or land located within the zone-of-influence of an area undergoing subsidence may possibly become damaged. The extent and intensity of this damage will vary considerably from slight and negligible, to severe. The evacuation and subsequent demolition of homes and buildings can occur in some mining subsidence areas. Hazards commonly associated with subsidence include fissures, gas emissions, acidic minewater discharges, flooding, fault reactivation, ground compression, tilt, curvature, seismicity, and vibration.

Trough subsidence associated with longwall mining can be predicted with a reasonable degree of accuracy and precision. Empirical subsidence calculation methods use formulae established from observations. Profile function methods are

Mining Hazards, Fig. 1 Drilling rig being recovered from a collapsed mine shaft, Garscadden No. 3, Achamore Road, Drumchapel, Glasgow, Scotland, November 1995. Permit Number CP18/015 British Geological Survey © NERC 2018.

based on the geometry of an excavation. Influence function methods calculate subsidence at a point on the ground surface. Unlike trough subsidence, the subsidence associated with the collapse of mine entries, shallow workings or other mine-related voids cannot be reliably predicted. This may occur decades or hundreds of years after mining has ceased and often suddenly without obvious forewarning. Furthermore, engineering or building works or changes to groundwater levels can initiate subsidence.

The monitoring of subsidence is possible using, for example, precise leveling from a fixed datum located beyond the area undergoing subsidence. Other methods use global positioning systems (GPS), total station and electronic distance measurements (EDM), and interferometry synthetic aperture radar (InSAR), all of which can detect ground movements associated with subsidence.

Fault Reactivation and Fissures

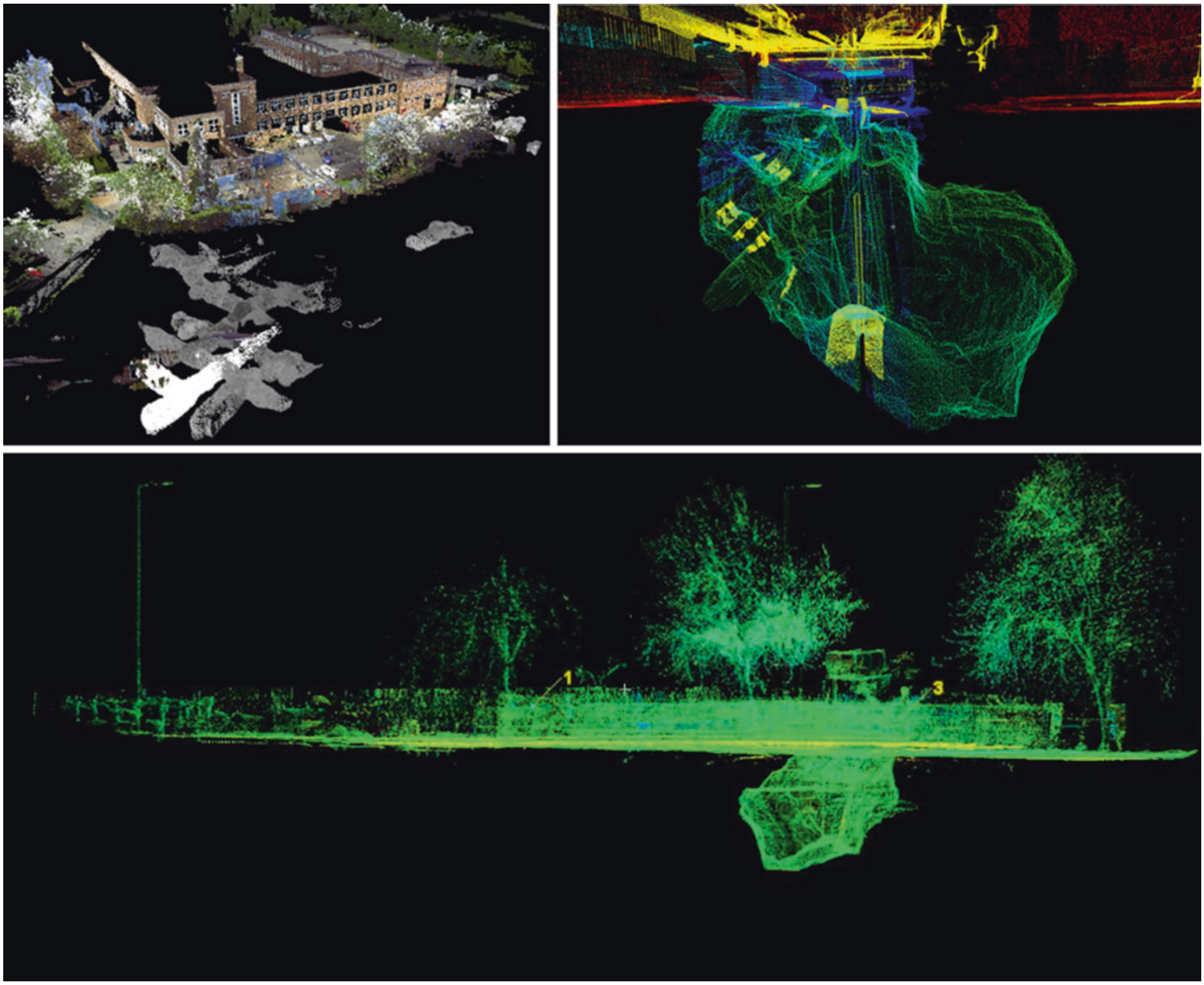
Faults are susceptible to reactivation when subjected to strains and ground movements induced by mining subsidence (Donnelly 2006). This commonly generates a ground rupture (known as a scarp, step or break line) along at the ground surface. These vary considerably in size, up to a meter or so high and several hundreds of meters long, although scarp morphologies can vary considerably. Rarely, mining reactivated faults reach 3–4 km in length and up to approximately 4 m in height. Fissures occur where faults and rock mass discontinuities undergo dilation caused by tensile ground strains. Fissures are usually narrow (less than ca. 1.0 m) but subsequently widen by weathering or collapse of their side walls. Reactivated faults cause damage to homes, buildings, structures, conveyances (e.g., sewers, cables, and pipelines), and agricultural land. Fault reactivation does not continue indefinitely, but several stages of reactivation can take place separated by periods of inactivity (Fig. 3).

Ground containing faults and fissures in areas where longwall mining takes place should be monitored, using the ground deformation techniques noted elsewhere. Fault outcrop positions should be avoided for the siting of new structures until the ground movements have ceased, as determined by monitoring. Fissures may be remediated by choking with a suitable engineered fill material, the placement of a reinforced concrete raft, water jetting and backfilling, or the use of geosynthetic material or geogrids.

Mine Gas

Gases are associated with active and abandoned mines. These have been responsible for widespread loss of miners lives due to explosions or asphyxiation. Gases collect in mine workings, shafts, adits, goaf, and voids where the natural or forced ventilation is no longer operating. Typical mine gases are methane (blackdamp), hydrogen sulfide (stinkdamp), carbon dioxide, carbon monoxide (whitedamp), oxygen depleted air (stythe), and radon.

Methane (CH_4) at concentrations of 4.4 to 17% by volume can explode if there is a source of ignition. Methane is also an asphyxiant. Methane is less dense than air so it tends to collect in the roof or crown of mine roadways. Methane is colorless



Mining Hazards, Fig. 2 3D geo-referenced laser scan survey of abandoned chalk workings (top left) and coal mine voids that collapsed or were about to collapse (Source: Mark Hudson, Geoterra Ltd)

and odorless and cannot be detected by the physical senses. Hydrogen sulfide (H_2S) has a rotten egg odor. At low concentrations, the smell is recognizable by humans, but this obvious odor disappears at high concentrations. It is less dense than air and collects on the floor of mine workings or exploratory trenches. Hydrogen sulfide is toxic to humans, corrosive, and flammable and explosive at concentrates of 4.3–46% by volume. At about 0.01% by volume (100 ppm), it is hazardous to life. At 0.002% (20 ppm), adverse health effects can occur. Carbon dioxide (CO_2) is an asphyxiant. A volume concentration of 5% results in a noticeably increased rate of breathing. Carbon monoxide (CO) is toxic at very low levels. It is colorless, odorless, and tasteless and often associated with combustion. Severe effects on humans will occur following a short exposure time in concentrations as low as 0.1% by volume (1000 ppm). Oxygen (O_2) is colorless, odorless, and tasteless. A total of 24% as opposed to the usual ca. 20.9% can create a potentially dangerous environment that can promote fire. An oxygen deficient mine atmosphere will cause asphyxiation. Radon (Rn-222) is a radioactive gas generated from the decay series of uranium-238 and is a direct decay product of radium-226. Uranium-238 occurs naturally in all rocks and soils. Radon is colorless and odorless and can be inhaled. Inside the lungs, the radioactive elements continue to emit radiation and this causes health problems. Therefore, the main danger from high radon exposure is the long-term increased risk of lung cancer.

Ventilation prevents hazardous gas concentrations from accumulating. Sources of ignition should be eliminated in active and abandoned mines and near mine entries. Multigas detectors should be worn by all mine personnel. An alarm system and evacuation procedure must be in place in the event of gas concentrations. All mine equipment must be intrinsically safe. In abandoned mining areas gas venting may be

Mining Hazards, Fig. 3 Mining subsidence induced reactivation of the Tableland Fault generating a fault scarp, 3-4 m high a c4km long, and the Darren Goch landslide, South Wales

required at mine entries. Gas purging with an inert gas (e.g., nitrogen) can be an effective mitigation measure.

Minewater Rebound and Acid Mine Drainage

Pumping controls minewater levels in both underground and open pit mines. Following mine abandonment expensive minewater pumping may cease. This causes the rebound of the water table towards pre-pumping levels. Consequently, mineworkings may flood until equilibrium is achieved. The duration of rebound ranges from years to decades. Minewater can be high in dissolved metal content or ferruginous, with low pH. Acid Mine Drainage (AMD) occurs when mine water discharges from mine workings (Fig. 4). Where AMD flows into surface water courses, this can cause pollution by the precipitation of ochreous ferric hydroxide. Some mines may discharge water that is corrosive due to high salinity, reducing water quality. Gas emission and subsidence can also be induced by minewater rebound. Sudden mine water blowouts or flooding can occur many years after mine abandonment. In 1992, pumps were turned off resulting in the catastrophic release of contaminated mine water from a mine adit at the Wheal Jane mine, in Cornwall, UK. This resulted in orange discoloration and acid water polluting the River Carnon and Fal Estuary.

Minewater treatment schemes aim at neutralizing acidity of minewater and removing dissolved salts and metals. Passive treatments operate without inputs of energy and chemicals and require less maintenance. These include aerobic wetlands and limestone drains. Active treatment systems require the addition of chemical reagents, tend to be favored less than passive systems, and may be more expensive (Younger and Robins 2002; Metes et al. 1998).

Spontaneous Combustion

Coal seams, coal waste, and carbonaceous material can oxidize when exposed to oxygen in air that in turn can lead to spontaneous combustion (Fig. 5). High rank coal (e.g., anthracite) tends to be more susceptible than low rank coal (e.g., lignite and bituminous). This is associated with the oxidation of pyrite (bacteria or phenols) in coal, which generates an exothermic reaction that can ignite the coal. Consequently, coal fires burn uncontrollably in many coalfields of the world. Coal fires destroy enormous areas of coal reserves and make it difficult for safe extraction. Coal fires produce noxious gas plumes that can sink to the bottom of open pits. These gases have a detrimental impact on fauna, flora, and people who live and work at and in the vicinity of the mines. Coal fires change the geotechnical properties of the rock mass or waste. Residual deposits known as “red shale,” “burnt shale,” or “clinker” show where past burning has taken place. Hot spots may develop with temperature in the range of 600–900 °C. Wild fires may start by spontaneous combustions. Coal fires may generate voids, subsidence, and sinkholes. Steam under high pressure (water gas) can discharge from fissures. Typical areas where coal fires burn

Mining Hazards, Fig. 4 Acid mine drainage from an abandoned mine adit, UK

include USA, South Africa, China, UK, and Colombia. In Pennsylvania, USA, over 200 coal fires have destroyed large areas of vegetation and the ground remains polluted with sulfur (Donnelly and Bell 2010).

Spontaneous combustion is difficult to predict, control, and mitigate. The potential for coal fires to ignite can be investigated as part of a ground investigation by measuring the properties of the carbonaceous deposits. The monitoring of coal fires is possible by ground or air deployed thermal imagery, geophysical surveys, and temperature monitoring in boreholes. In India, coal seams exposed in open pit mines are sprayed with bitumen to seal rock mass discontinuities in an attempt to prevent air ingress. Underground fires may be controlled by purging with nitrogen or water from boreholes. In China, some fires have been extinguished by controlled explosions.

Mining-Induced Landslides

Mining beneath slopes and areas of moderate to high relief can reduce the factor of safety and influence groundwater flow, leading to the initiation or reactivation of landslides (Fig. 3). The Frank slide was a major coal mining-induced rockslide that happened in 1903, in western Alberta, Canada. Approximately 82 million tons of limestone slipped along the flank of Turtle Mountain causing widespread damage to a mine and the Canadian Pacific Railway, including about 90 fatalities.

Open pit mines and quarries are susceptible to failure causing economic loss, and in some cases this may lead to abandonment. In 1984, failure of the St Aiden's open pit, in the UK, caused the River Aire to change course and flood the mine. Insufficient attention was given to the design of the stability of the open pit walls. Consequently, this resulted in the introduction of the Quarry Regulations (HSE 2013).

Waste tips can fail and must be carefully designed and monitored. In 1966, approximately 38,000 m^3 of colliery waste failed and flowed along the western flank of the Taff valley, at Aberfan, in South Wales, UK. The flow slide buried a school and 18 houses. As a result, 144 people died of which 116 of these were children. The flow slide was triggered by over-loading of the tip with colliery waste and mining subsidence that produced tensile strain, opening fissures in the sandstone bedrock, which then enabled groundwater to emerge as springs in front of the tip.

Mining-Induced Seismicity

Induced seismicity is associated with mining, shale gas hydraulic fracturing, enhanced hydrothermal systems, oil and gas exploration and production, groundwater withdrawal, reservoir impoundment, and the disposal of waste into boreholes. In mining, seismicity is generated by changes in the three dimensional stress-field around a mine void. This is also referred to as mining-induced earthquakes or tremors. They tend to occur within the upper 1 km of the ground surface where mining takes place, by comparison to naturally occurring tectonic earthquakes that are much deeper. There are several potential sources of mining-induced seismicity including fault reactivation (although some faults reactivate aseismically); subsidence and bedding plane separation; the collapse of strata into a goaf; mineshaft, stope, or roadway collapse; the shear failure of support pillars; and roof falls, rock bursts, and outbursts. Vibrations can be induced by drilling, blasting, mining machinery, and hydraulic fracturing.

Mining-induced seismicity can cause damage to a mine, well (borehole), or structures located near the source. In addition, the public perception of induced seismicity can be particularly challenging to manage. Ground motions may potentially have an adverse influence on sensitive sites and critical infrastructure such as power stations, hospitals, laboratories, or hazardous waste sites.

Mitigation measures include the establishment of baseline microseismic monitoring program in the vicinity of a mine and surrounding region before mining commences, quantification of the stress regime, and review of background

Mining Hazards, Fig. 5 Spontaneous combustion of a coal seam in India and a coal waste tip in the UK (Source: Donnelly and Bell 2010 and WAI)

seismicity with the installation of surface and a buried (in boreholes) array of seismometers. For sensitive structures, this may require the determination of ground vibration criteria (e.g., peak ground velocity (PGV), peak particle velocity (PPV), peak ground acceleration (PGA); the development of ground motion prediction equations and a shake map; a Deterministic Seismic Hazard Assessment (DSHA) or Probabilistic Seismic Hazard Assessment (PSHA) and the development of a risk based mitigation plan (such as the implementation of a Traffic Light System).

Strata Control

In underground mining, it is essential to control the strata to maintain stability. This requires analysis of the engineering behavior of the rock mass. Typically this involves the provision of geotechnical design parameters, an assessment of the tensile and compressive strength, classification of rock mass discontinuities, Poisson's ratio, modulus of elasticity, geological structures, stress fields, groundwater conditions, and geometry of the mine openings.

In underground mines, strata-controlled hazards include roof falls, rock bursts, outbursts, and floor heave. This also requires an understanding of the three dimension stress fields and how these will be modified by mining. Mine roadways, shafts, adits, haulage, and access routes tend to require permanent rock mass support. Other parts of a mine such as the operational face and goaf only require temporary support. Typical supports are conventional steel arch, mechanized supports, rock bolts, grouting, shotcrete, welded mesh, and silica resins. Other underground mining hazards are associated with fire, flood, inrush, gas, vapors, dust, blasting, machinery, noise, heat, and airborne particulates.

Waste, Dereliction, and Contamination

Mining waste occurs in the form of solids, sludges (slurries), and liquids that can become chemically hazardous. Contamination includes organic and inorganic chemical substances. If present in sufficient quantities and concentrations, it has the potential to cause harm to the environment and to human health. Open pit mines and quarries can fill with non-engineered waste, landfill, or other waste. Dereliction includes the presence of physical objects such as concrete, old foundations, basements, cables, utilities, and reinforcement. However, if open pit backfilling is undertaken with inert waste that is engineered and compacted to a given specification, it may be possible for houses, building, or other structures to be constructed, provided settlement has been achieved and bearing capacity is acceptable. This could include the placement of an engineered clay or geotextile caps to control the environmental migration of leachate and gas. Dust generated from mine tips and open pits can produce localized hazards. This may be mitigated by water spraying. As noted elsewhere, waste tips can fail and all tips must be designed for stability and regularly monitored.

Widespread environmental damage and loss of lives have occurred when tailing dams fail. The standard of inspection and reporting is variable and poor in many countries, with incomplete recordings of past failures. The Bento Rodrigues dam disaster in Brazil in 2015 released 60 million m^3 of iron ore waste. In China, in 1962, the Huogudu tailings pond and dam failure killed 171 and injured another 92. Tailings dams must be properly designed and constructed and subjected to rigorous monitoring and treatment of acid drainage.

Summary

The most common mining hazards include but not limited to ground collapse associated with mine entries (shafts and adits) or shallow, abandoned mine workings; subsidence; fault reactivation and fissures; minewater rebound; acid mine water drainage; mine gas emissions; spontaneous combustion; landslides; seismicity; strata control; waste; dereliction and contamination. Mining hazards, although potentially foreseeable, cannot always be forecasted or predicted in terms of their timing, location, duration, magnitude, and extent. A comprehensive mine closure and abandonment plan allows mining hazards and liabilities to become anticipated and mitigated. Mining hazards do not necessarily occur in isolation but are interrelated and often occur simultaneously. One event may trigger another. A holistic method for risk evaluation should be considered. Monitoring and inspections are recommended to be implemented during, immediately following and long after the cessation of mineral production and mine abandonment.

Cross-References

▶ Acid Mine Drainage
▶ Blasting
▶ Boreholes
▶ Borehole Investigations
▶ Classification of Rocks
▶ Classification of Soils
▶ Cut and Fill
▶ Dewatering
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▶ Zone of Influence

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Modelling

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Definition

Modelling	process of developing a model.
Model	simplified and idealized understanding of a particular phenomenon or physical system. In the context of engineering geology a geo-model is a mathematical representation of a particular phenomenon occurring in the Earth's crust based on geophysical and geological observations made on and below the Earth's surface, built and used to explain and predict its behavior.

Introduction

A geo-model attempts to reproduce the behavior of a particular phenomenon or physical system occurring in the Earth's crust. It usually describes the studied geo-system and its behavior by a set of variables and a set of equations that establish relationships between the variables. The variables represent certain properties of the geo-system. In engineering geology applications, chemo-thermo-hydro-mechanical processes can be modelled and many different variables can be used. For instance, for mechanical processes where motion and behavior of matter through space and time must be described, stress and strain of the involved materials are two important variables that must be addressed.

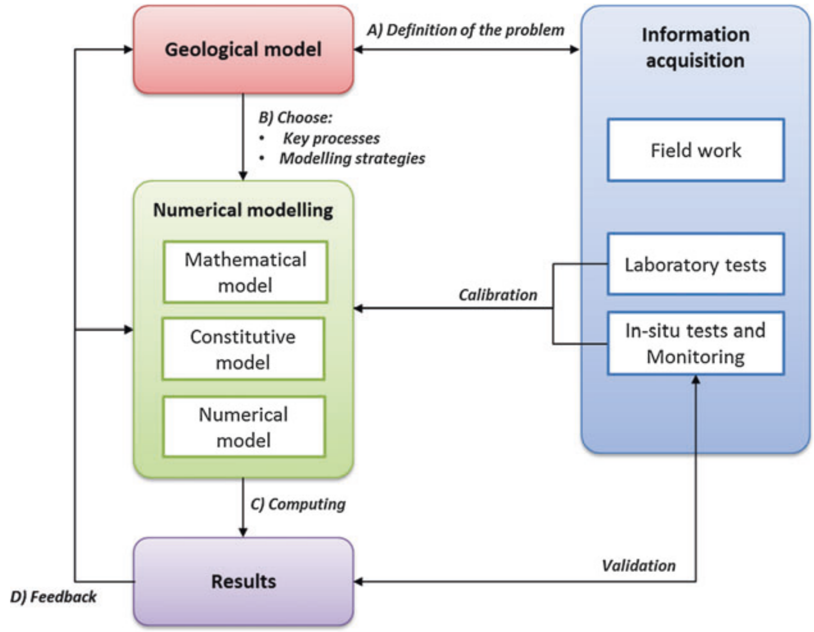
Two kinds of models can be distinguished. First, the phenomenological models that describe an empirical relationship between variables in a manner that is consistent with fundamental theory but is not directly derived from theory. Regression analysis and other statistical processes for estimating the relationships among variables are sometimes used to create statistical models that serve as phenomenological models.

In contrast, physically based models describe a relationship between the variables of the system through accepted physical laws. For instance, for mechanical processes, the set of physical laws describing the motion of bodies (kinematics) under the influence of a system of forces is based on Newton's second law of motion (classical mechanics). Physically based models are sometimes referred to, in a brief way, as "mathematical" or "numerical" models, as these models refer to a mathematical or numerical description of the more relevant physical phenomena which take place in the problem being analyzed. In the following text only physically based models will be discussed and general statements refer to this category.

The main goal of modelling is to provide a tool to understand, define, quantify, visualize, or simulate how a particular phenomenon or geo-system behaves. In some cases the phenomenon is purely natural and modelling is used to identify natural hazards and quantify geological processes. If the proposed model is able to reproduce past and present behavior, it can be used also to predict what will happen in the future. In other cases, the phenomenon arises from the interaction between humans and the environment. Often when geological, civil, or mining engineers analyze a geo-system to be controlled or optimized, they use a physically based model. In analysis, engineers can build a descriptive model of the geo-system as a hypothesis of how the system could work, or try to estimate how an unforeseeable event could affect the system. Models are commonly used for managing natural resources with main applications to oil, and gas fields, or groundwater aquifers. In the field of geotechnics, they are commonly used for the design of geo-structures like tunnels, Earth retaining structures, cut slopes, embankments, and shallow and deep foundations.

The process of developing a geo-model can be explained by the flow chart in Fig. 1. First, the scenario must be characterized and the problem must be defined from data acquired through field observations and monitoring, including geological, geodetic, and geophysical observations. The definition of the problem produces a geological model where all the data: geometry, materials, load, and boundary conditions are defined describing the system in a given time. The involved information can be further characterized using laboratory and *in-situ* tests. After the recognition of the more relevant physical phenomena and key process that take place, the numerical modelling computes the system response according to the given geological model. In short, "numerical modelling" aims to quantify the proposed geological model.

Numerical modelling comprises: (i) a mathematical model that describes the multi-physical couplings of the studied phenomenon, (ii) a constitutive model that describes the behavior of the involved items, and (iii) a numerical model that solves the partial derivative equations obtained from the mathematical and constitutive model. A crucial part of the



Modelling, Fig. 1 Modelling flowchart

modelling process is the evaluation of whether or not a given model describes a system accurately. Usually the easiest part of model validation is checking whether a model fits the experimental measurements. An accurate model closely matches the experimental measurements. An inaccurate model should be improved through a better calibration of the input parameters or the improvement of the model itself by refining the geological, mathematical, constitutive, or numerical models, if a key process or model strategy was not well selected.

During recent decades, mathematical, constitutive, and numerical models have been very much improved and today their use is widespread both in industry and in research.

Mathematical Modelling

The term "mathematical model" refers to a mathematical description of the more relevant physical phenomena which take place in the problem being analyzed. It is indeed a wide area and several classification criteria can be used for mathematical models according to their structure. A first classification criterion is to differentiate discrete and continuum models; a discrete model treats objects as disconnected materials, such as the particles of a granular material, whereas a continuum model represents the objects in a continuous manner. Due to scale analysis, continuum models prevail in engineering geology applications. Another important classification criterion, that is crucial to describe for instance fluid-motion (fluid dynamics), is the choice between Lagrangian and Eulerian formulations. In a Lagrangian approach, the material or particles of the body are tracked as they move and their properties (velocity, temperature, density, mass, concentration, etc.) are recorded at different locations as the material moves. In an Eulerian approach, the observation point location is fixed and changes in properties are recorded as different materials pass through that location. In soil and rock mechanics, the approach followed most often is mixed, Lagrangian for the solid skeleton and Eulerian for the relative movement of the pore fluids relative to the soil skeleton.

The materials comprising the Earth's crust, soils and rocks, are geomaterials with voids which can be filled with water, air, and other media. They are, therefore, multiphase materials, exhibiting a mechanical behavior governed by the coupling between all the phases. Pore pressures of fluids filling the voids play a paramount role in the behavior of a geo-structure, and indeed, their variations can induce failure. If we consider soil as a mixture, we will have equations describing: (i) mass balance for all phases, that is, solid skeleton, water, and air, in the case of nonsaturated soils, (ii) balance of linear momentum for pore fluids and for the mixture, and (iii) constitutive equations. The first mathematical model describing the coupling between solid and fluid phases (HM models) for linear elastic materials was proposed by Karl von Terzaghi through the definition of the effective stress acting on a soil (Terzaghi 1925) and the one-dimensional consolidation theory (Terzaghi and Fröhlich 1936); a theory which was extended to three dimensions by Maurice Anthony Biot in 1941 (Biot 1941). These works were followed by further developments to extend the theory to nonlinear materials and large deformation problems, see for instance Zienkiewicz et al. (1999). The geotechnical community has incorporated coupled formulations to describe the behavior of foundations and geo-structures. Indeed, analyses of Earth dams, slope failures, and landslide triggering mechanisms have been carried out using such techniques during the last few decades.

In some cases, for instance, in energy production and storage applications such as nonconventional hydrocarbons extraction, deep geothermal energy production, very low-energy geo-structures, compressed air energy storage, heat storage in salt caverns, carbon dioxide geological storage or nuclear waste storage, temperature is a key variable in the analyzed process that might affect fluid transfers and the mechanical behavior of the geomaterials at play (soils, reservoir rocks, caprock, etc.). To tackle these complex problems that involve coupled thermo-hydro-mechanical phenomena (THM models), energy conservation equations should be used. In other cases, chemical reactions are of particular interest and must also be described as in the case of chemical dissolution of carbonate materials.

Summarizing, the mathematical model, represented by a set of partial derivative equations, aims to describe the chemo-thermo-hydro-mechanical coupling (ChTHM) occurring in the analyzed geo-system.

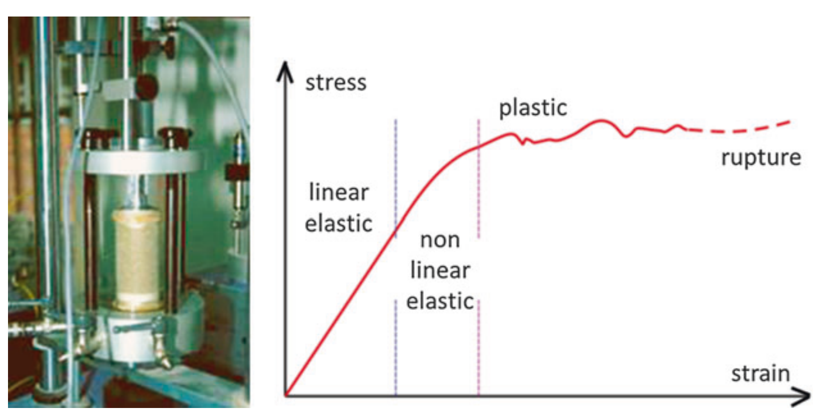
Constitutive Modelling

Mathematical models described in the preceding section have to be completed using constitutive or rheological models. These models describe the behavior of the involved materials. For the purely mechanical process, they relate stress and strain variables, and they explain how the body system deforms according to a given stress increment, or how the body stress changes according to a defined strain increment. These mechanical behaviors are different in the case of rock versus cohesive or granular materials. For coupled chemo-thermo-hydro-mechanical problems, they depend also on the presence of interstitial fluids, temperature, or chemical reactions.

Great effort has been devoted in the past few decades to develop accurate constitutive models that account for the more important aspects of soil and rock behavior, and today there is a wide choice between many elastoplastic, visco-plastic, hypoplastic, non-linear incremental, generalized plasticity, or damage models, to name a few. Interested readers are referred to "Constitutive Modelling of Geomaterials" (Cambou and Di Prisco 2000), published within the Alliance of Laboratories in Europe for Research and Technology (ALERT) "Geomaterials," created in 1989 as a pioneering effort to develop an European school of thinking in the field of the Mechanics of Geomaterials.

Figure 2 shows a typical stress-strain relationship obtained from a triaxial test apparatus. In this device two kinds of experiments can be performed on the selected material specimen, first the stress is prescribed and the strain is measured with a "load controlled" test and next the strain is prescribed and the stress is measured by a "displacement controlled" test. Both tests help to define the constitutive model and its constitutive parameters. For instance, in the case of the simpler elastic linear models, only the Young modulus and Poisson ratio must be defined. For more complicated behavior and advanced constitutive relations, the number of required constitutive parameters increases making the incremental stress-strain relationship strongly nonlinear.

The purpose of many engineering geology applications, as in the case of the analysis of slope stability problems, is to describe and estimate a final failure mechanism. Therefore laboratory tests are performed many times until the rupture of the specimen is achieved. The chosen constitutive relationship and calibration of the material parameters must reproduce the observed failure mechanisms. In some cases, this could be localized along a shear surface, in other cases this can be diffuse as in the case of liquefaction phenomena. Moreover, some materials can exhibit brittle failure behavior whereas other materials can exhibit ductile behavior.



Modelling, Fig. 2 Triaxial test device and stress-strain behavior

For fluid like materials, rheology, that relates stress and rate of deformation, must be defined through simple Newtonian behavior using a viscosity parameter or more complex non-Newtonian laws such as Bingham fluid models or cohesive frictional viscoplastic models.

Constitutive modelling is a key ingredient of the modelling strategy, the choice of an appropriate constitutive or rheological law that reproduces the behavior of the considered materials must be done very carefully.

Numerical Modelling

Mathematical and constitutive modelling produces a set of partial differential equations (PDEs), which are space and time dependent; furthermore, in most cases they are highly nonlinear. For the simpler models, analytical solutions can be obtained; for those which are more complicated, the resolution requires the use of numerical approximations. The numerical modelling refers to the numerical analysis performed to obtain the solution of these partial differential equations.

PDEs are solved by first discretizing the equations, namely, replacing continuum problems by a discrete problem whose solution is known to approximate the continuous problem; values are calculated at discrete places on a meshed geometry. The three most widely used numerical methods to do it are: (i) the finite element method (FEM), (ii) finite volume methods (FVM), and (iii) finite difference methods (FDM). Other kinds of methods called Meshfree methods were made to solve problems where the previously mentioned methods are limited, such as problems where large deformations of the material can occur. These methods eliminate all the spatial derivatives from the PDE, thus approximating the PDE locally with a set of algebraic equations for steady state problems or a set of ordinary differential equations (ODEs) for transient problems. Further numerical algorithms are needed to completely solve the numerical problem ranging from methods for solving systems of linear equations, methods for solving the ODEs equations, or methods for evaluating integrals.

The FEM has been extensively and successfully used in the last few decades to solve many engineering geology applications. Interested readers are referred to Zienckiewic's books (Zienkiewicz and Taylor 2013; Zienkiewicz et al. 2013a, b) where a complete description of the method for solid and fluid mechanics applications can be found. For the case of more specific soil or rock mechanics and geotechnical applications where the hydro-mechanical coupling is important, Zienkiewicz et al. (1999) and Potts and Zdravkovic (2001) are good references.

Finally, the chosen numerical algorithms must be implemented in a computer program using a programming language. Since the late twentieth century, most algorithms are implemented in a variety of programming languages mostly in Fortran and C focusing on speed performance. Speed computation and hardware memory are still important restrictions for the analysis of large problems as in the case of largescale 3D computational meshes. High performance computing using parallel processing on computer clusters is currently a key issue for these numerical drawbacks.

The mathematical, constitutive, and numerical models described so far have been implemented in an innumerable quantity of commercial and research codes around the word. To make a list is an impossible task and suffice that there is a program for each of the mentioned techniques. The choice of the appropriate program depends strongly on the intrinsic characteristics of the studied phenomenon or engineering geology application.

Examples of Modelling in Engineering Geology

In the following section, examples illustrating the modelling of some engineering geology applications are presented. For the sake of clarity and simplicity, only mass movement examples associated to different kinds of landslides and subsidence phenomena are showed.

Landslides and slope stability problems are one of the engineering geology applications causing important losses of human lives and damage to property. There are a wide variety of types of landslides depending on the materials involved and the triggering mechanism; therefore, several modelling strategies are known.

Landslides are commonly classified as: (i) slides, (ii) flows, (iii) falls, (iv) topples, (v) lateral spreads, and (vi) complex movements. Slides are characterized by a mass movement over a defined failure surface. From a mechanical point of view, it can be said that failure mechanism consists of localization of shear deformation on a thin surface. On the other hand, flows can be described as fluid like movements where individual particles travel separately. Regarding the triggering mechanisms, landslides and other slope failures are caused by changes in the effective stresses, variation of material properties, or changes in the geometry. Changes of effective stresses can be induced either directly, as consequence of variation of the external forces (earthquakes, human action) or indirectly through pore pressures (rainfall effects). Variations in material properties can be caused by processes of degradation (weathering and chemical attack). Finally, geometry can change because of natural causes (erosion) or human action (excavation, construction, reshaping).

In the following, three kinds of landslide modelling are illustrated: (i) a slide, (ii) a flow slide and (iii) a rock fall.

Slope Stability and Landslides Modelling

Physically based models have been mostly used in practical cases to estimate landslide occurrence and stability conditions for a given scenario through a stability factor, balance of shear stress and shear strength. Analytical solutions based on limit equilibrium analysis can only be obtained for very simple geometries. Complex numerical models such as the finite element method can provide a better understanding of the mechanism of failure because they can reproduce the fully coupled hydrogeological and mechanical behavior. Moreover, advanced constitutive laws, complex 3D geometries, and spatial variations in soil properties can be used.

Figure 3 depicts the modelling strategy that has been followed in the case of an active paleo-landslide that was reactivated by the construction of a parking area at the toe of a slope (Fernández-Merodo et al. 2014; Bru et al. 2017). Field studies and localized cracks indicate that the ground is moving (Fig. 3a). *In-situ* monitoring confirms this observation; inclinometer monitoring shows a deep rupture surface below the surface and ground base SAR monitoring indicates the surficial extension of the landslide (Fig. 3b). 2D and 3D geological models are shown in Fig. 3c, whereas 2D and 3D finite element meshes are presented in Fig. 3d. Finally some results of the finite element modelling showing the failure mechanism are plotted in Fig. 3e. In this case, a viscoplastic behavior of the altered slate has been considered for the constitutive model and a Lagrangian finite element method for the numerical model. This model reproduces the observed continuous creep deformation and acceleration of the movement during rainy periods (Fernández-Merodo et al. 2014; Bru et al. 2017).

Flow Slides Modelling

As examples of flows, it is worth mentioning that flow slides occur in poorly compacted deposits which experience a sudden collapse with important build-up of pore pressures and liquefaction in some cases. The failure mechanism can be described as diffuse, and the moving mass does not behave like a rigid block. Rock avalanches for instance are a case where rock blocks disaggregate into much smaller particles, and the final behavior is that of a frictional fluid. Of course, both types are idealizations of reality. It is possible to find slides which evolve into flows, especially in cases where there exists sufficient water which mixes with the soil. In the following example, a flow slide of coalmine waste dumps is simulated (Pastor et al. 2002). The mobilized volume of debris was approximately 200,000 m^3, which slid off a 100 m high dump, travelling a distance of 700 m before coming to rest. This dump constructed by end-tipping failed as a consequence of redistribution of the effective stresses caused by changes of the pore pressure and the liquefaction under quasi-undrained conditions of the finer sandy gravel layers of low permeability deposited parallel to the dump face. The topography of the initial and final situation is shown in Fig. 4a, computed space and temporal evolution of the flow slides are represented in Fig. 4b. In this case, a frictional fluid behavior has been considered for the rheological law and an Eulerian finite element method for the numerical model. The model reproduces the observed trajectory, velocity, and final state of the flow slide.

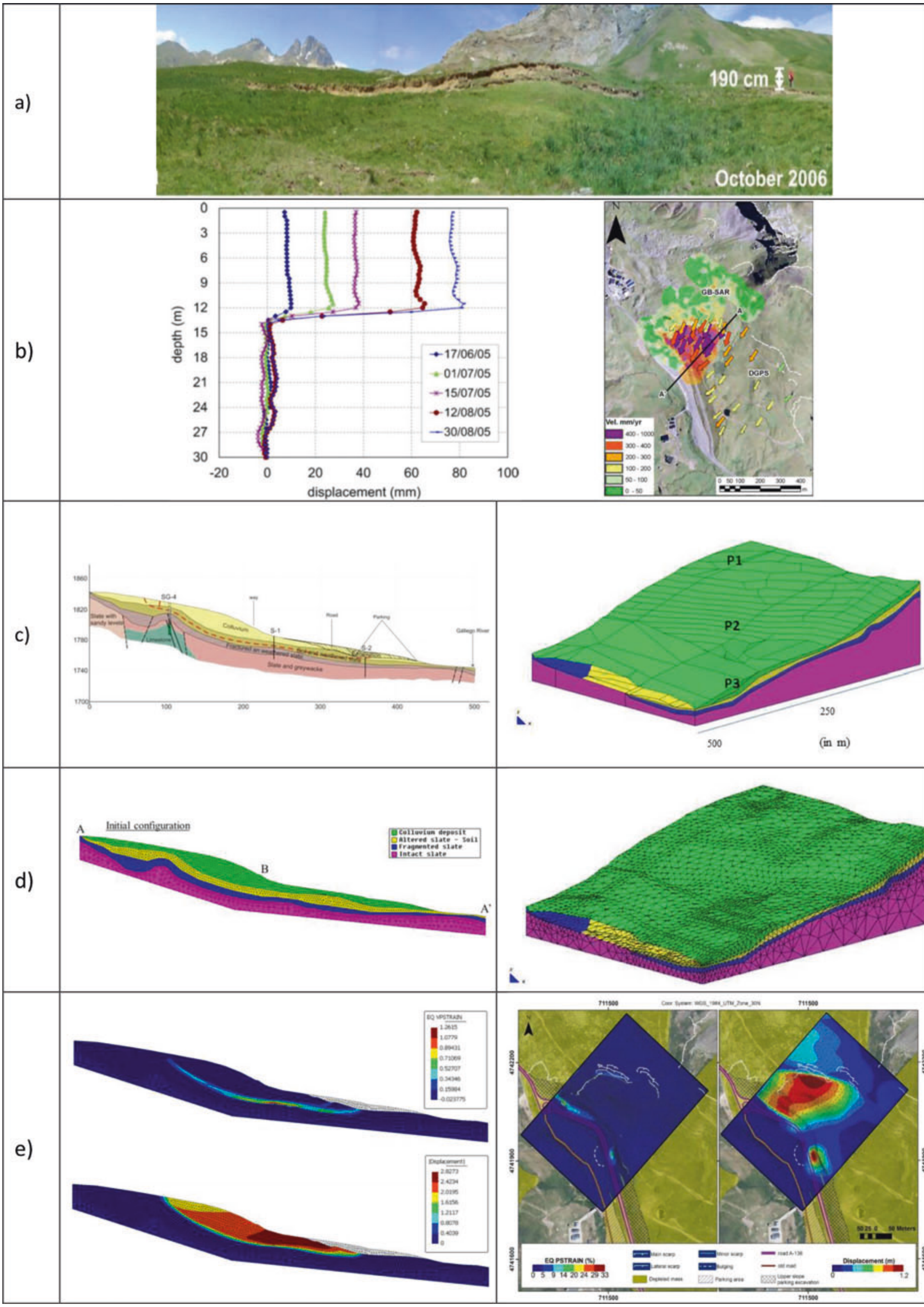
Rock Falls Modelling

In this case, modelling is a very powerful tool for the design and the dimensioning of protection works as the modelling computes trajectory, velocity, and stopping points of the detached blocks. Figure 5 shows an example of rock fall modelling where the 3D model of the topography, the blocks starting zones, and the characterization of materials have been defined. In Fig. 5c, computed trajectories and kinetic energy are shown. Here a frictional rolling and frictional sliding model are used together with a discrete numerical approach. The model computes expected trajectories according to small statistical variation of the input parameters.

Subsidence Modelling

Ground subsidence may be due to several causes including deep material dissolution, excavation of ground tunnels or mining galleries, deep erosion, lateral soil creep, compaction of soil materials due to fluid extraction (water, petroleum or gas), or tectonic activity. All of these causes are manifested at the ground surface as vertical deformations that can vary from a few millimeters to several meters during periods that vary from minutes to years.

The following example concerns a subsidence model in the case of a complex 3D geophysical system composed of the interaction of different boundary, load conditions, and mechanical processes due to: (i) the presence of two excavated tunnels, (ii) the presence of two buildings that produce an overload of the ground, and (iii) the progressive dissolution of carbonated materials (Ciantia et al. 2018). Figure 6a depicts a sketch of the geometry; Fig. 6b shows the 3D finite element mesh used, and Fig. 6c presents the subsidence modelling results as contours of displacements. Here an advanced chemo-mechanical behavior of the material based on elastoplasticity theory has been considered for the constitutive model and a Lagrangian finite element method for the numerical model.



Modelling, Fig. 3 Landslide modelling. (**a**) Field observations: main scarps, (**b**) monitoring: inclinometer and ground base SAR data, (**c**) geological model in 2D and 3D, (**d**) numerical model: finite element meshes in 2D and 3D, (**e**) modelling results: computed displacement and viscoplastic deformation contours in 2D and 3D (modified after Fernández-Merodo et al. 2014; Bru et al. 2017)

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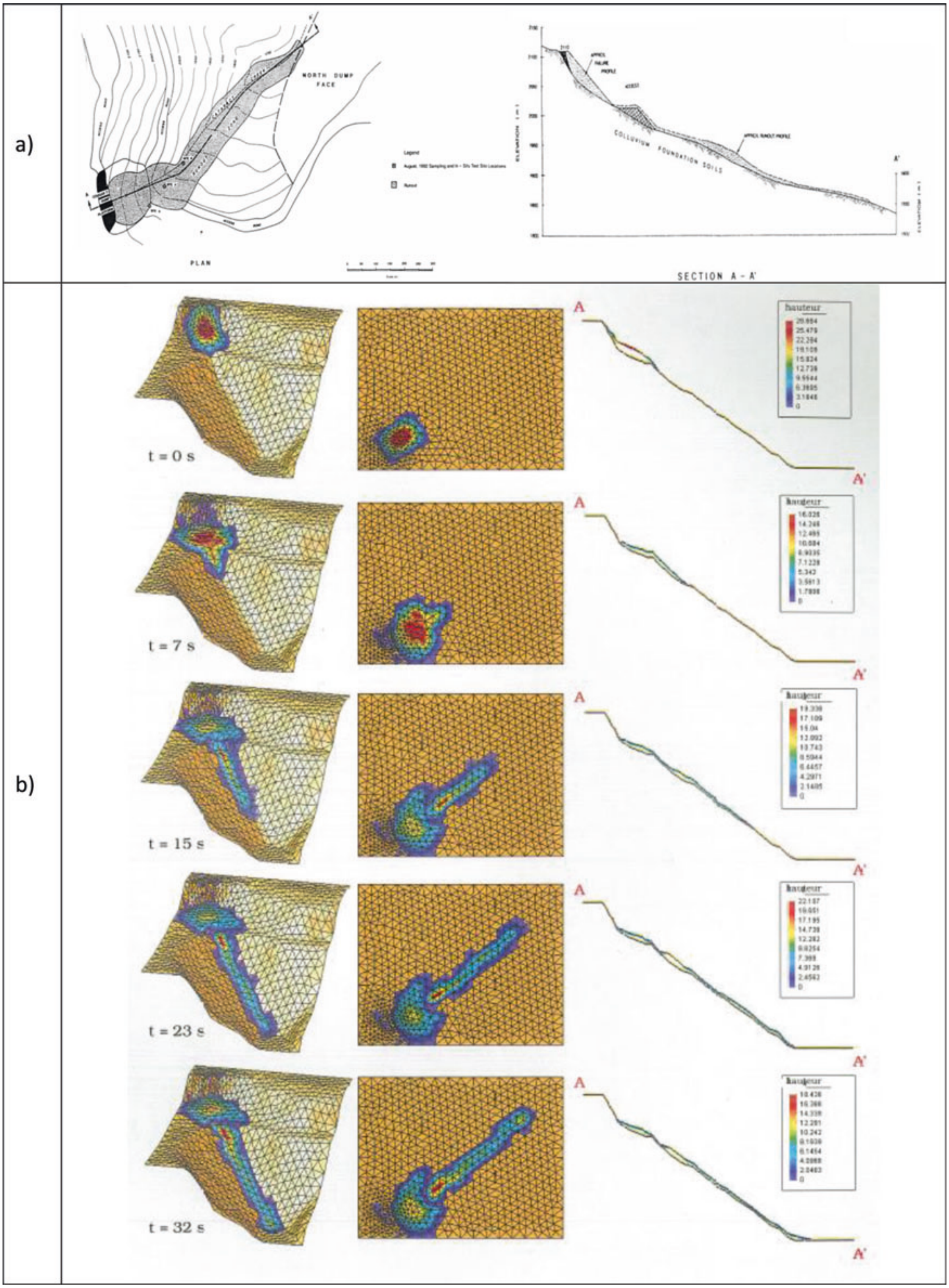
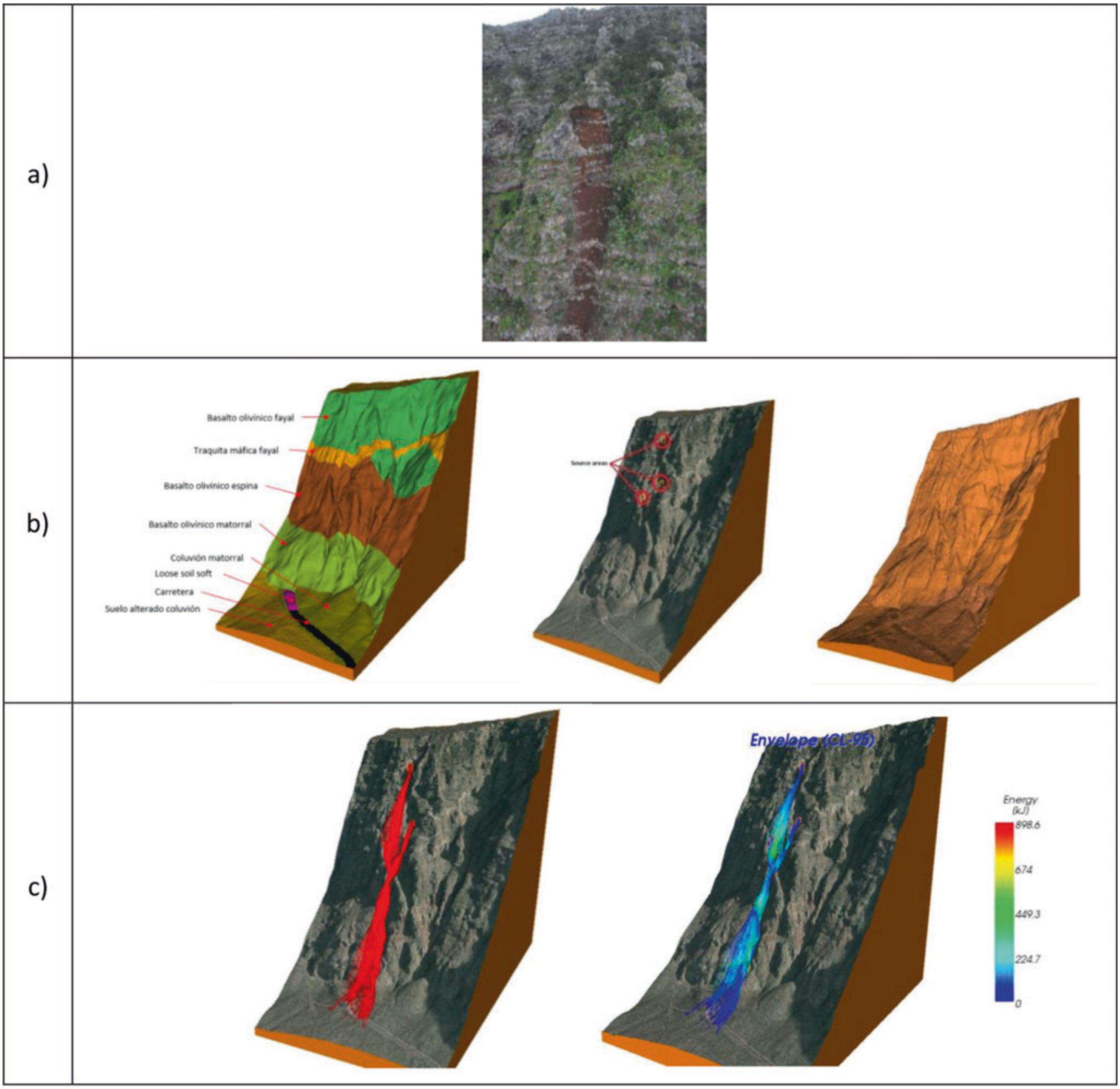


Modelling, Fig. 4 Flow slide modelling. (**a**) Geological model: plan view and section view, (**b**) modelling results: computed space and temporal evolution of the flow slide (modified after Fernández-Merodo et al. 2014; Bru et al. 2017)



Modelling, Fig. 5 Rock falls modelling. (**a**) Field observations: block detachment, (**b**) geological model: materials, source areas, and digital terrain model (DTM), (**c**) modelling results: computed trajectories and kinetic energy

Conclusions

Modelling in engineering geology is a reliable indicator of the degree of knowledge of the studied phenomenon and its triggering mechanisms. It is not possible to simulate or reproduce numerically a phenomenon whose mechanisms are not described by the proposed model.

A wide range of application can be covered by modelling provided that the problem could be reasonably simplified, key processes identified, and a numerical strategy optimized. During the last few decades, coinciding with the exponential development of computers, mathematical, constitutive, and numerical models have been much improved and today their use is widespread both in industry and in research. The choice

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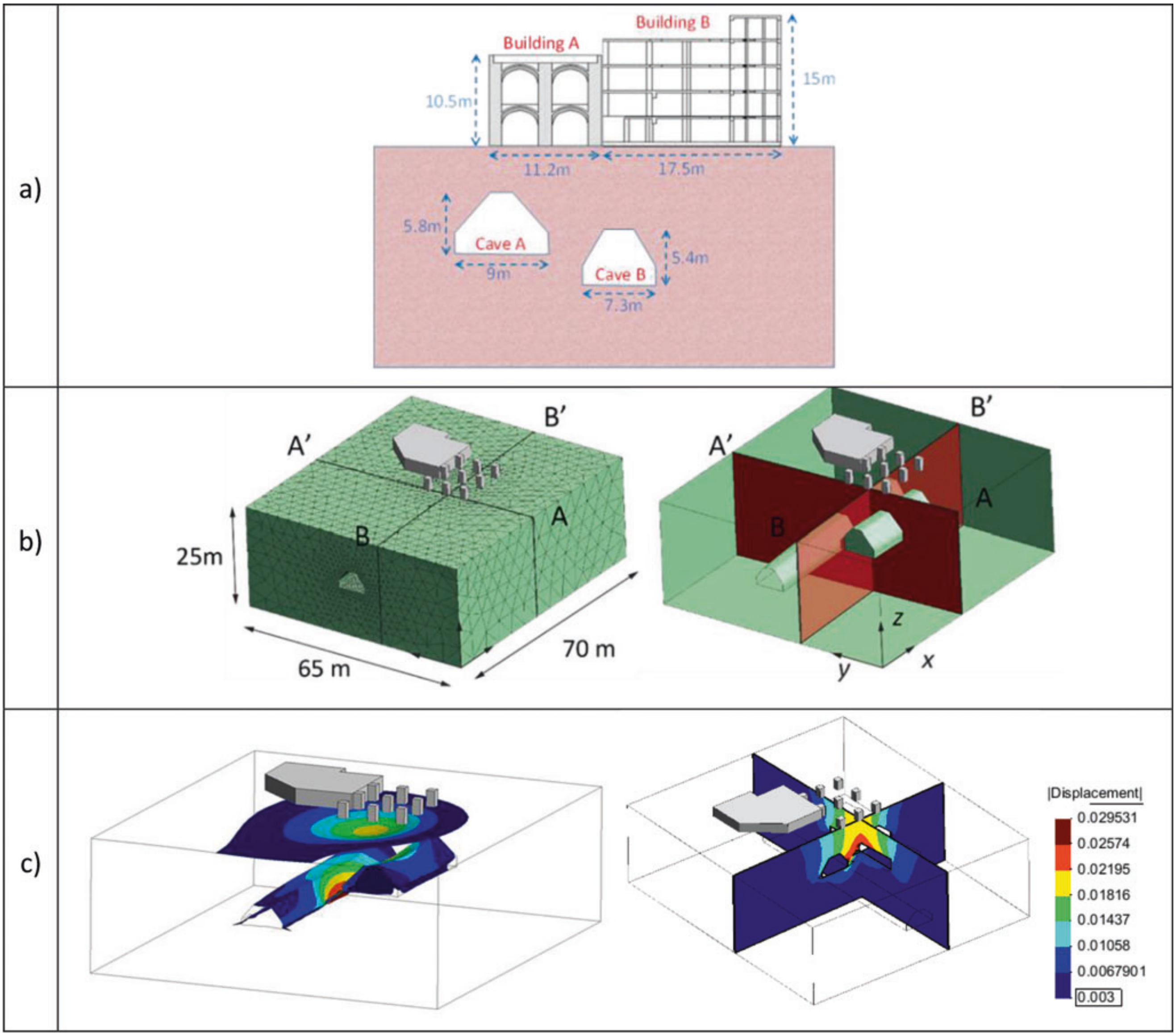

Modelling, Fig. 6 Subsidence modelling. (**a**) Geometry sketch, (**b**) 3D finite element mesh, (**c**) modelling results: computed displacement contours (Modified after Ciantia et al. 2018)

of the appropriate model depends strongly on the intrinsic geophysical characteristics of the engineering geology application.

Besides the model, quantification of the studied phenomenon cannot be achieved without reliable input parameters for the proposed model, namely, a complete characterization of the scenario through field observations and advanced monitoring. This requirement is often a technical and economical constraint due to the large scale of space and heterogeneity involved in engineering geological applications. Therefore, a geo-modeller always has to be aware of the uncertainties associated with the proposed model.

Consequently, quantification of the studied phenomenon relies on two fundamental pillars: (i) monitoring (or data collection) and (ii) modelling. In many engineering geology applications, models able to simulate past and present events can now be applied to make predictions about the behavior of geo-structures following some defined scenarios. The quantification of the response and the characterization of the expected consequences and effects, both in space and time, as they are dynamic models over time (4D), results in a very useful tool for decision-makers who have to manage, control, optimize, or reduce the risk associate to the studied phenomenon or geo-system.

Cross-References

- ▶ Instrumentation
- ▶ Landslide
- ▶ Mechanical Properties
- ▶ Rock Laboratory Tests
- ▶ Shear Strength
- ▶ Shear Stress
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Strain
- ▶ Strength

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Modulus of Deformation

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Synonyms

Young's modulus

Definition

The deformation modulus is Young's modulus (E) for a rock mass (E_{rm}) or a soil mass; it is the ratio of principal stress in one direction (σ_x) to corresponding strain in the elastic range in the same direction (ε_x). It is not easily measured, so estimation methods based on geophysical measurements or rock mass classification schemes tend to be used. Estimates of a dynamic Poisson's ratio (v_d) can be calculated from surface or downhole geophysical measurements of compression-wave (Vp) and shear-wave (Vs) velocities, and a dynamic shear modulus (G_d) can be calculated using geophysical measurement of shear-wave velocity (Vs) and estimates of mass density (ρ) for the volume of earth materials. A dynamic rock mass deformation modulus (E_{rmd}) can be estimated as

$$\begin{aligned} E_{rmd} &= 2\rho(V_s)^2\left[1+\left(\frac{1}{2}\frac{\left(\frac{Vp^2}{Vs^2}\right)-2}{\left(\frac{Vp^2}{Vs^2}\right)-1}\right)\right] \\ &= 2G_d(1+v_d) \end{aligned} \tag{1}$$

where ρ is in mass units (kg/m^3) and Vs and Vp are in velocity units (m/s), which gives G_d in MPa or GPa.

The method for estimated deformation modulus (E_{rm} in MPa) developed by Hoek and Diederichs (2006) is based on Young's modulus of the intact rock material (E_i), the Geological Strength Index (GSI) rock mass classification scheme (Hoek et al. 2013), and the rock mass disturbance factor (D) applied to the uppermost 0.5–3 m of the rock mass depending on excavation methods and care.

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Modulus of Deformation, Table 1 Examples of modulus ratio for selected rock types and textures (RocScience 2016)

Rock type	Texture	Modulus ratio (dimensionless)
Claystone	Very fine	250 ± 50
Shale	Very fine	200 ± 50
Siltstone	Fine	375 ± 25
Sandstone	Medium	375 ± 75
Conglomerate	Coarse	350 ± 50
Basalt	Fine	350 ± 100
Granodiorite	Medium	425 ± 25
Granite	Coarse	425 ± 125
Slate	Very fine	500 ± 100
Quartzite	Fine	375 ± 75
Schist	Medium	675 ± 425
Gneiss	Coarse	525 ± 225
Marble	Coarse	850 ± 150

$$E_{rm} = E_i \left(0.02 + \frac{1 - \frac{D}{2}}{1 + e^{\left(\frac{(60+15D-GSI)}{11}\right)}} \right) \tag{2}$$

where D ranges from 0 to 1, depending on extent of disturbance caused by rock blasting, and GSI is a rock-mass-strength score ranging from 1 to 100 (Hoek et al. 2013). If an intact rock modulus values has not been measured in the laboratory, it may be estimated based on unconfined compressive strength in MPa and a modulus ratio value that is indexed by rock type and texture to E_i (RocScience 2016; Table 1).

Cross-References

- ► Bulk Modulus
- ► Modulus of Elasticity
- ► Poisson's Ratio
- ► Rock Properties
- ► Shear Modulus
- ► Strain
- ► Stress
- ► Young's Modulus

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Modulus of Elasticity

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Definition

Modulus of elasticity (E) is the name given to the slope of the stress-strain (σ_x–ε_x) curve for a material subject to axial principal stress in the elastic range of the material. It is synonymous with Young's modulus, which also is represented by the symbol E.

Cross-References

- ► Young's Modulus

Mohr Circle

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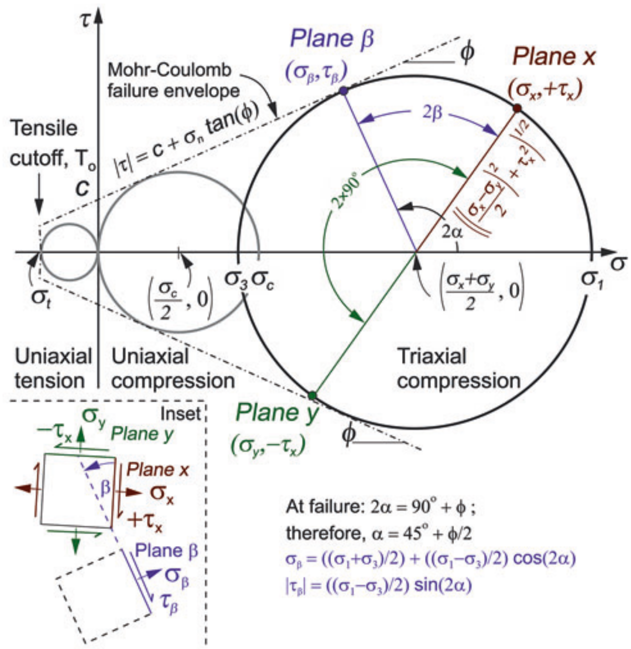
Definition

A graphical representation of normal stresses and shear stresses was developed by Professor Otto Mohr (1835–1918) who recognized that the equations could be rearranged into the form describing a circle:

$$\left[\sigma_\beta - \left(\frac{\sigma_x + \sigma_y}{2}\right)\right]^2 + \left[\tau_\beta - 0\right]^2 = \left[\sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_x^2}\right]^2 \tag{1}$$

where the second part of the right-hand term is the x coordinate of the circle center, the second part of the middle term is the y coordinate of the circle center, and the term inside the right-hand bracket is the circle radius. The Mohr circle graph contains three circles in a plane stress example: $\sigma_2 = 0$. A similar graph showing three-dimensional information would have two additional circles inset into each of the three circles shown that intersect the normal stress x axis at the value of σ_2 with diameters equal to ($\sigma_1 - \sigma_3$) (shown),

$(\sigma_1 - \sigma_2)$, and $(\sigma_2 - \sigma_3)$. The smaller circles that intersect the origin are uniaxial compression and uniaxial tension. The dot-dash lines that are tangent to the circles denote the Mohr-Coulomb failure envelope; ϕ is the angle of internal friction. The inset graph displays positions of Plane x, Plane y, and Plane β in the largest Mohr stress circle. A similar construction can be made to show the Mohr strain circle.



Mohr-Coulomb Failure Envelope

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Synonyms

Mohr-Coulomb Failure Criterion

Definitions

A constitutive model that describes the shear strength of ground.

Exceeding the shear strength results in failure of the ground which can be described by the Mohr-Coulomb Failure Envelope. This is also used, sometimes, as a constitutive model for the shear strength along surfaces of, for example, discontinuities.

Ground materials are diverse and may be gases, fluids, solids (i.e., minerals, grains, and aggregates of grains or minerals), and any mix. Also, man-made ground, such as fills and waste dump materials, may often be modeled with the Mohr-Coulomb failure envelope. Ground is commonly differentiated between soil and rock, soil being an aggregate of loose or weakly bonded particles, and rock consisting of particles cemented or locked together, giving rock a tensile strength. Soil and rock can be differentiated based on a compressive strength difference with soil being weaker than 1 MPa and rock being stronger. A differentiation is made between "intact" and "discontinuous" ground, that is, ground without versus with distinct planes of mechanical weakness such as faults, joints, bedding planes, fractures, schistosity, etc. A groundmass consists of (blocks of) intact ground with discontinuities if present. The stresses in the ground should be in terms of total and effective stresses.

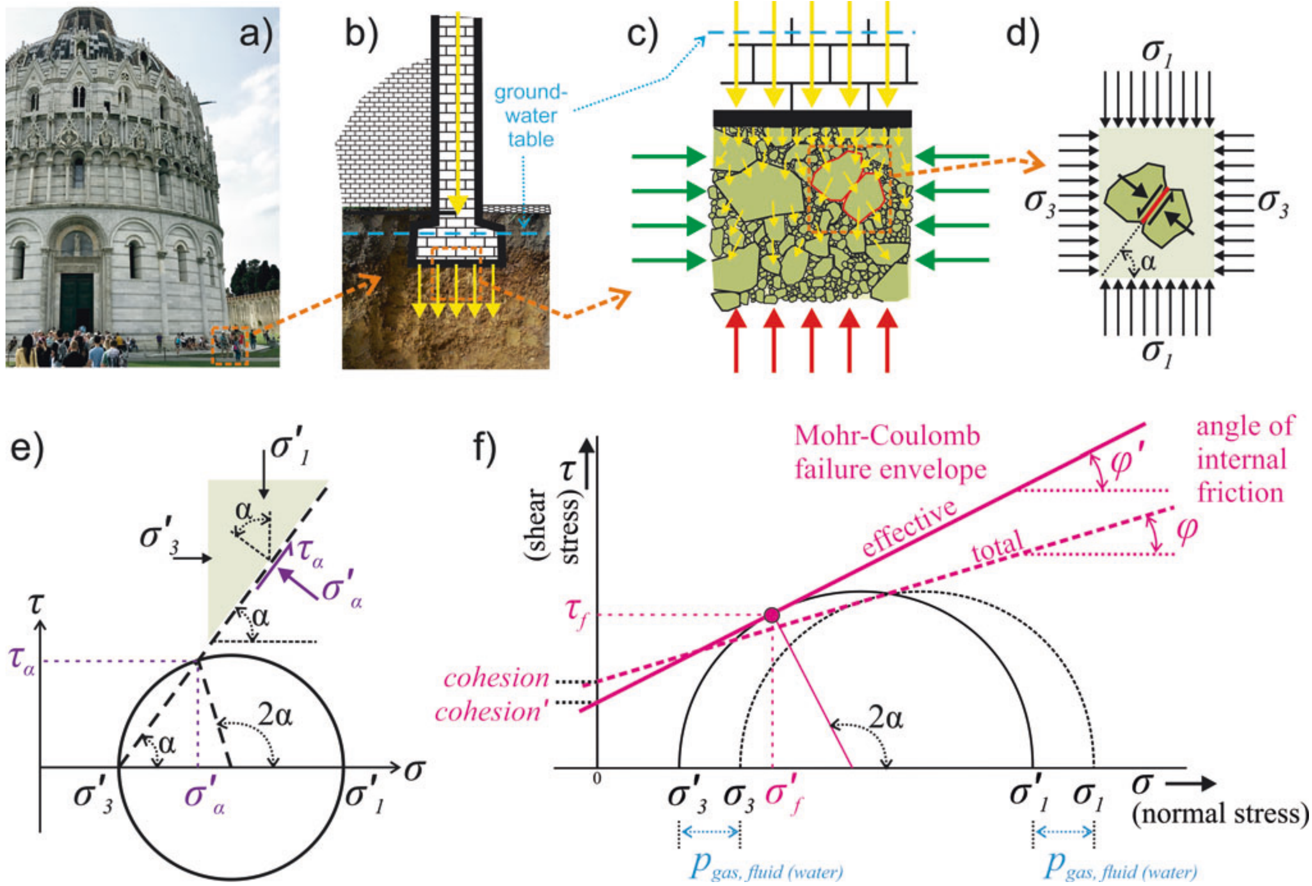
Mohr-Coulomb Failure Envelope

Figure 1a shows the foundation of a surface object, the reactions in the ground (Fig. 1b, c), and the stress configuration between two particles (Fig. 1d). The stress configuration on the contact between the two ground particles is shown in Fig. 1e in the Mohr-circle diagram. The maximum shear stress sustainable between the ground particles can be formulated with a Mohr-Coulomb failure envelope (Coulomb 1776). The envelope gives the boundary condition of the shear and normal effective stress configuration at which the shear stress equals the shear strength and is formulated in Eq. 1. The stresses are effective stresses and, for uniformity, also the parameters are identified to indicate this. Figure 1f shows the stress configuration when the stress circle touches the Mohr-Coulomb failure envelope (in point σ'_f, τ_f).

$$\tau = c' + \sigma' \tan \varphi' \qquad (1)$$

c' = (effective) *cohesion* (in rock mechanics also denoted *Si*)
φ' = (effective) *angle of internal friction* (for material strength) or (effective) *angle of friction* (for a surface)
τ = shear strength
σ' = effective normal stress on shear plane

The parameter "*cohesion*" (Eq. 1) is not the same as the "tensile" strength of ground. Ground with tensile strength will also have cohesion, but not necessarily the same value, and ground without tensile strength may or may not have cohesion. The Mohr-Coulomb failure envelope is a constitutive



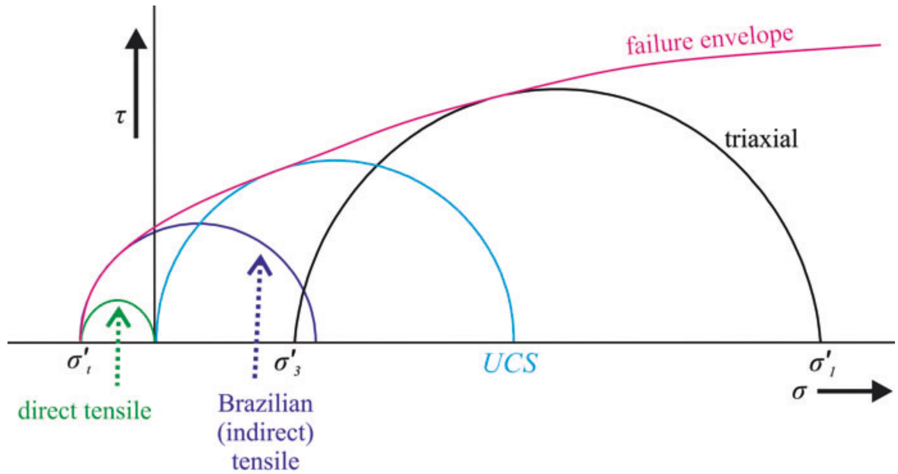
Mohr-Coulomb Failure Envelope, Fig. 1 (**a**) Baptisterium, Pisa, Italy; (**b**) schematized foundation of the wall, the arrows indicate the load of the foundation; (**c**) foundation load (yellow); confining pressure (green) in part due to sideway expansion of the ground under the foundation load, and in red the reaction stress of the subsoil due to the foundation load; (**d**) enlargement showing the configuration of normal and shear stresses between two ground particles; (**e**) the geometry, stresses, and position in the Mohr-circle diagram; (**f**) Mohr-circle diagram with Mohr-Coulomb failure envelope (Photo courtesy Arnoldus 2017)

model suitable for describing the strength of many soils, intact rock, and rock masses. Values of *cohesion'* and *φ'* for different grounds including the range of confining pressure for which these apply are given in the chapter on Mechanical Properties. The Mohr-Coulomb failure envelop can also be formulated in terms of total stresses, that is, effective stresses plus pore gas and fluid pressure (Fig. 1f). This is normally only applicable to situations where the pore gas and fluid pressure cannot dissipate quickly or at all, for example, for fast loading of a low-permeable clay (undrained situation). Although the Mohr-Coulomb failure envelope is often suitable it is not always valid over the entire range of confining pressures. For many types of ground the envelope is not a perfectly straight line, but is curved (Fig. 2). The parameters of the Mohr-Coulomb envelope are then only applicable for the range of confining pressure where the curved envelope can be approximated by a straight line.

The Mohr-Coulomb failure envelope may also be applied to the shear strength along a plane (i.e., a discontinuity). The mathematical formulation is similar to Eq. 1; however, more sophisticated constitutive models are mostly used for discontinuities that include, for example, roughness and strength of asperities (Hencher 2015).

Curved Failure Envelope

Many types of ground and, in particular, groundmasses do not fit the Mohr-Coulomb failure envelope very well so other empirical relations between failure and stress configuration have been proposed (e.g., Hoek and Brown 1980; Bieniawski 1974; Hoek



Mohr-Coulomb Failure Envelope, Fig. 2 Empirical failure envelope following Eq. 2

et al. 2002; Hack et al. 2003). Figure 2 shows the empirical relation of Bieniawski (1974) which is formulated as:

$$\frac{\sigma_1'}{UCS} = 1 + N\left[\frac{\sigma_3'}{UCS}\right]^M \qquad (2)$$

σ_1', σ_3' = major, minor effective principal stress
N, M = material constants
UCS = Unconfined Compressive Strength

Cross-References

▶ Mechanical Properties
▶ Shear Strength

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Monitoring

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Definition

Systematic use of quantitative approaches and/or measuring devices to observe changes which may occur over time in the state of a system.

Introduction

Observation is the mainstay of the Galilean method (Fisher 1993). To determine the range of variability of a specific phenomenon under investigation, the observation has to be repeated over time, in a systematic fashion, and by considering straightforward quantitative approaches. This process is usually referred to as monitoring.

In the past the unique sources of observation were human eyes; however, monitoring today is mostly undertaken using devices able to record quantitative measurements of one or more physical parameters. In general, when

Monitoring, Fig. 1 Temperature survey on an active volcanic field, Lastarria volcano, Andes, Chilean-Argentinian border (Photo: A. Manconi)

measurements are aimed at performing provisional analyses, the data acquired with monitoring instruments are jointly investigated and/or properly combined with significant information of the same target area. Regarding engineering geology applications, this approach is achieved by compiling quantitative information obtained from monitoring instruments on thematic maps, using, for example, geographical information system (GIS) platforms or by interpreting the data through conceptual or mathematical models. When only a static snapshot of the situation at the time of measurements is retrieved, the analysis is referred to as "surveying" or "site investigation." A survey evolves into proper monitoring only when multitemporal data is acquired with a predefined strategy, in order to reconstruct the spatial and/or temporal evolution of one or more physical parameters of interest.

For example, thermometers (or thermistors) can be used for spot measurements on rocks (or soils) at a given time to understand local effects of temperature (see Fig. 1). However, only by installing permanently specific devices (e.g., thermocouples) and connecting them to readout units can the evolution of temperatures be monitored over time. This allows analysis and understanding of how the characteristics (or behavior) of materials are affected by temperature variations (e.g., cyclic expansion/contraction). Moreover, multitemporal data acquisition performed with infrared cameras may also allow retrieval of information on the spatial distribution of temperatures (Bell 2004).

Similar examples, considering different monitoring instruments, can be found among other parameters of frequent interest in engineering geology applications, such as groundwater pore pressure, stress, and strain.

The Monitoring Pillars

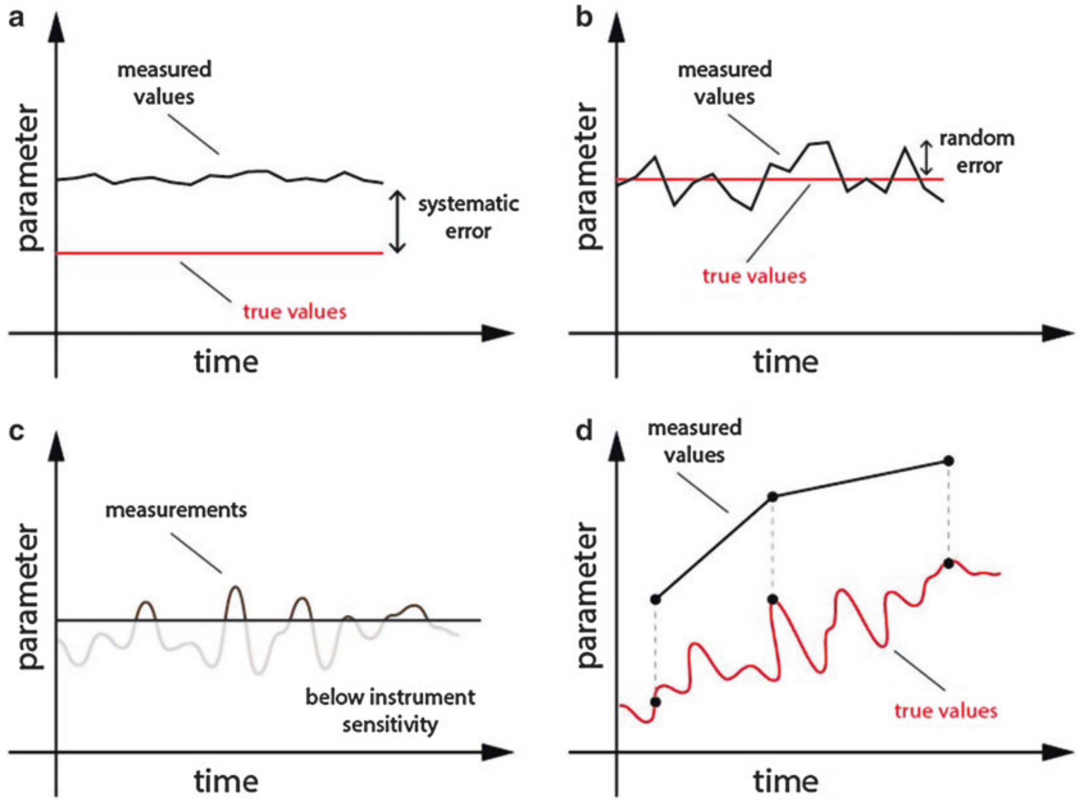
When introducing monitoring, a number of background concepts have to be carefully taken into account (see Fig. 2), including:

(a) *Precision*: is a description of the consistency between repeated measurements of a given physical quantity. Precision is usually referred to as the standard deviation of the values resulting from multiple measurements performed in similar conditions. Precision is often referred to as "repeatability." The concept of precision is associated with "random" errors, that is, variations in the measurements that occur without a predictable pattern.

(b) *Accuracy/Uncertainty*: is the degree of closeness between the measurement of a quantity and its true value. However, since all measurements are intrinsically affected by errors, the true value is never achievable (Bell 2001). Thus, the accuracy concept blurs into the definition of "uncertainty," which is typically expressed as the range of values in which the true one lies within a given statistical confidence interval. Accuracy involves both a "random" error component (see point (a) above) and a "systematic" error component, that is, a predictable constant or proportional bias in the measurement. Systematic errors are usually caused by imperfect calibration of instruments.

(c) *Sensitivity*: is an absolute quantity defining the smallest variation in input signal to which an instrument can respond. The sensitivity of a monitoring system thus describes its capability to detect a change in the observed quantity with a single measurement. Sensitivity can be also referred to as "resolution" and is closely related to the instrument precision.

(d) *Sampling*: refers to the reduction of a continuous quantity to a discrete observation. Indeed, measurements



Monitoring, Fig. 2 The monitoring pillars. (**a**) Example of high-precision/low-accuracy measurements. (**b**) Example of high-accuracy/low-precision measurements. (**c**) When the monitored parameter is below the sensitivity threshold of the instrument, no data can be retrieved. (**d**) In the example, sampling points (*black dots*) are able to catch only a portion of the true temporal evolution of the parameter of interest. A wrong sampling strategy might lead to inaccurate or even misleading interpretation

are generally performed at specific locations and predefined times, that is, only a portion of the signal associated with the observed phenomenon is retrieved. The definition of sampling strategies, both in space and time, is the basis for achieving representative monitoring data.

Monitoring Scenarios in Engineering Geology

The data acquired from monitoring systems are often the key for a comprehensive understanding and interpretation of a phenomenon under investigation. Generally speaking, monitoring in engineering geology applications is aimed at evaluating possible changes occurring in the engineering geology matrix (Price and De Freitas 2009), or the combination of the material properties (shear strength, deformability, etc.), the mass fabric (bedding, discontinuities, faults, etc.), and the environmental factors (meteo-climatic variables, tectonic stress, time, anthropic interference). In this context, the ultimate goal of monitoring activities is usually the evaluation of stability of the ground mass (soils, rocks, as well as fluids/gases contained into them) and/or of engineering works. The technological advances achieved in the past few decades now allow retrieval of information in unprecedented detail. A wide spectrum of instruments and techniques are available, ranging from *in situ* instrumentation to remote sensing approaches. Examples of monitoring activities in the following two typical scenarios associated with engineering geology applications are presented.

Subsidence

The progressive lowering of the ground is defined as “subsidence.” Natural subsidence may occur in environments characterized by soluble rocks and/or compressive soils, as well as due to active tectonic and/or volcanic environments (see Fig. 3). Moreover, subsidence can be triggered by engineering activities, for example, mining, reservoir or aquifer exploitation, and tunnel construction (see Fig. 4). Excessive

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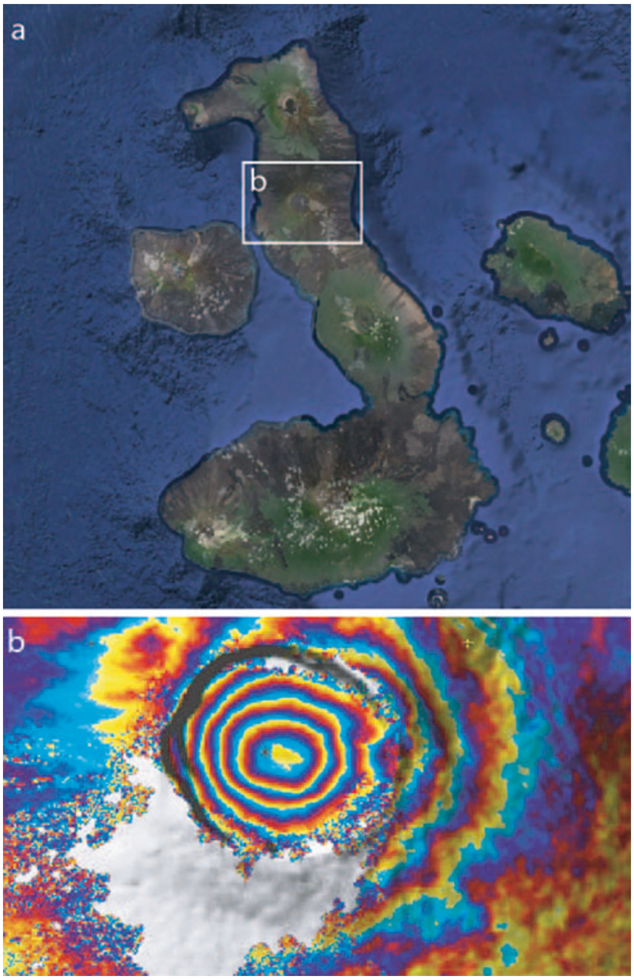


Monitoring, Fig. 3 Natural subsidence in an active volcanic field. Subsidence and uplift phases in active volcanoes are caused by the contraction and expansion of the magmatic system. (**a**) Galapagos Islands, satellite view from Google Earth. (**b**) Surface displacements from 1992 to 1998 at Darwin volcano, retrieved via DInSAR analysis. Each *color* fringe (from *blue* to *red*) shows 5.6 cm of subsidence, reaching a maximum value of about 25 cm in the inner caldera. The *star* shows the estimated position of the magma chamber (see Manconi et al. 2007)

subsidence can lead to local failure and/or potential damages to infrastructures.

The deformation caused by ground settlement is typically monitored by measuring the change of elevation at specific points (benchmarks) located on the surface by applying geodetic methods. In specific cases, measurements can be performed also in the subsurface, for example, by using special extensometers installed in boreholes.

Moreover, remote sensing approaches such as photogrammetry (Kajzar et al. 2011) and LiDAR (Yu et al. 2011) can also be used to retrieve information on subsidence. In recent years, subsidence at different locations has been identified and monitored by means of InSAR (Przyłucka et al. 2015). This technique is particularly convenient because it allows retrieving both the spatial extent and the temporal evolution of the ground displacements.

Slope Instability

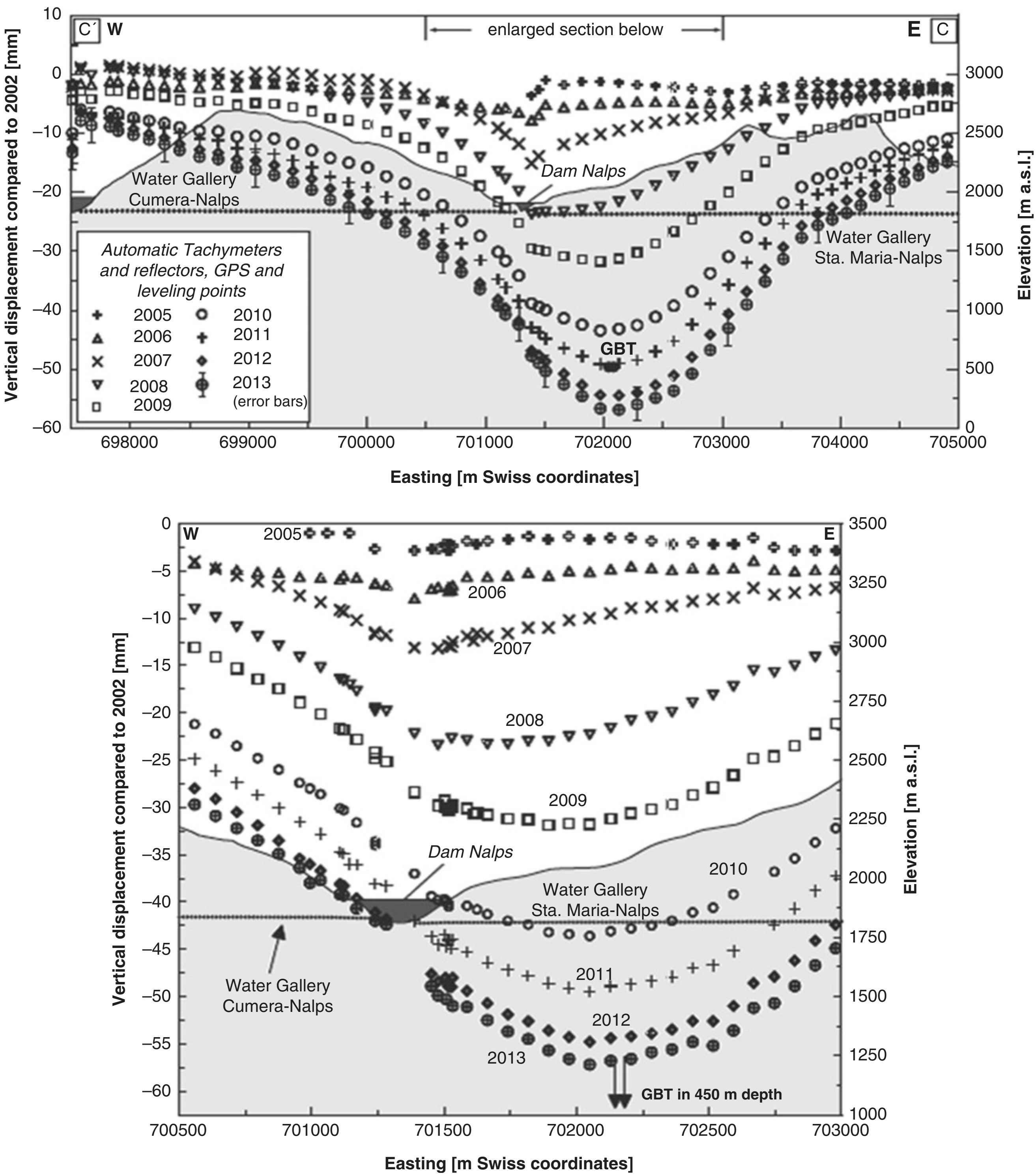
Instability occurs due to the natural evolution of the landscape, particularly in mountain environments, or due to engineering constructions causing oversteepening of slopes. Due to specific geological and geomorphological predisposing factors, as well as external triggers (intense and abundant rainfall, rapid snowmelt, and earthquakes), the shear strength of the materials can be exceeded, and landslides may occur. Several methods can be used to monitor slope stability. Field instrumentation to monitor surface deformation at unstable slopes includes permanent GPS/GNSS receivers (Fig. 5), geodetic levels, robotized total stations (Fig. 6), tiltmeters, and extensometers (Fig. 7). In addition, the geometry and the temporal evolution of the sliding surface at depth are monitored through inclinometers. Because hydrogeology deeply influences the mechanical behavior of landslides, additional parameters are also monitored, such as rainfall and/or water discharge evolution at surface (Fig. 8) and/or groundwater level and pore pressure in the subsurface (piezometers). As for subsidence, remote sensing techniques as LiDAR and InSAR are increasingly being used to detect, map, and monitor the evolution of surface displacements due to mass movements (Giordan et al. 2013; Manconi et al. 2014; Wasowski and Bovenga 2014).

The selection of the most appropriate technique, or combination of different techniques, depends on multiple factors, such as the extent of the study area, the size and type of the investigated phenomenon, and the main scope of monitoring (Wieczorek and Snyder 2009). The integration of several instruments and monitoring approaches is increasingly used to provide a more complete understanding of complex processes (Fig. 9).

Conclusions

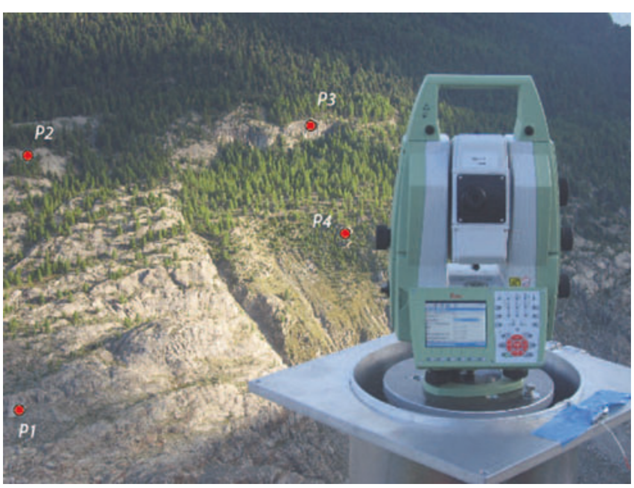
Research and technological development continue to make available more and more advanced monitoring devices of great importance in many fields of engineering geology. However, monitoring should not merely be considered in terms of installation and maintenance of instruments but extends to all the processes from the definition of the target of interest to the interpretation of the acquired data (Dunnicliff 1988).

Monitoring is crucial to assess environmental impacts, in particular due to large engineering works such as dams,

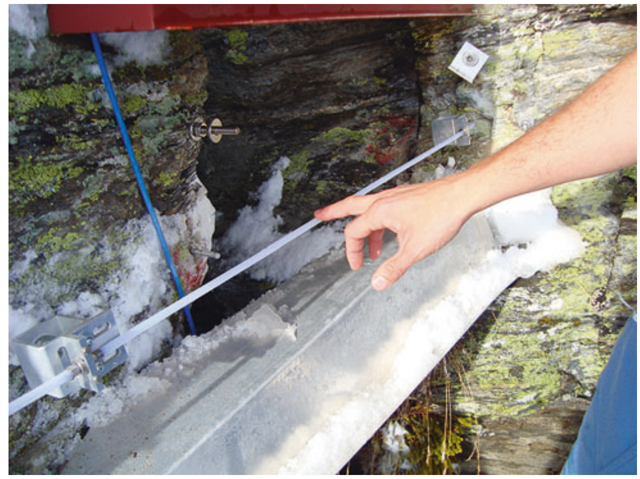


Monitoring, Fig. 4 Subsidence due to tunneling. Settlements 2005–2013 measured in a specific section through hydropower drifts and Nalps Dam oriented at high angle to the Gotthard Base Tunnel (GBT) axis. (**a**) Overview. (**b**) Detailed view of Nalps Dam location (Loew et al. 2015)

Monitoring, Fig. 5 GNSS station at Kilchenstock, Switzerland, monitoring long-term displacements of the slope in order to characterize movement behavior of a large creeping mass. The system was installed by ETH, Institute of Geodesy and Photogrammetry, and is in operation since 2014 (Photo A. Wolter)



Monitoring, Fig. 6 Leica Nova™ Robotic Total Station (RTS) installed in the Great Aletsch region, Switzerland. The RTS is a modern electro-optical monitoring system capable of measuring changes in the position of optical prisms installed in the area of interest (*red points*). In this specific case, the RTS was installed to monitor the evolution of a large unstable slope (Photo. F. Glüer)



Monitoring, Fig. 7 Detail of extensometers installed at Randa landslide, Switzerland (Moore et al. 2010) (Photo V. Gischig)

Monitoring, Fig. 8 Rainfall gauge and discharge measurement at the Cerentino landslide in Switzerland. The discharge is automatically recorded multiple times a day at the outlet of a surface collection pond installed to prevent meteoric water infiltration into the landslide body. The hydrogeological conditions at Cerentino are significant to its mechanical behavior. The system was installed by Canton Ticino and is in operation since 2009 (Photo A. Wolter)

tunnels, landfills, and deep geological repositories. Repeated measurements of a set of parameters selected as "sentinels" are usually performed both on the infrastructure and over an estimated area of influence. The frequency of measurements and the overall duration of monitoring activities vary depending on the criticality of the project and hazard potential. In particular situations, monitoring can be aimed at protecting vulnerable elements (Lollino et al. 2015). Systems of this kind are based on the intensive acquisition of measurements on limited zones and consist of precise and automated instruments capable of making

Monitoring, Fig. 9 Multiparametric monitoring station installed along the Montegrande ravine, Volcan de Colima, Mexico. At this location, the main task of the monitoring station is to provide timely information on the activity of the volcano. This is achieved by combining the analysis of optical images acquired at high rate (with a high-resolution video camera) and monitoring ground vibrations with geophones (Capra et al. 2016). (**a**) Overview of the multiparametric station. (**b**) Detail of the view framed by the video camera. (**c**) Detail of a geophone installed at ground (Photos: V. Coviello)

measurements at very high sampling rates (Intrieri et al. 2013). Automatic and near-real-time monitoring is fundamental to provide relevant information to ensure the safety of people and infrastructures.

Cross-References

- ▶ Aquifer
- ▶ Boreholes
- ▶ Deformation
- ▶ Earthquake
- ▶ Extensometer
- ▶ Failure Criteria
- ▶ GIS
- ▶ Hazard
- ▶ Hydrogeology
- ▶ Inclinometer
- ▶ Infrastructure
- ▶ InSAR
- ▶ Instrumentation
- ▶ Landslide
- ▶ LiDAR
- ▶ Mass Movement
- ▶ Mining
- ▶ Mountain Environments
- ▶ Photogrammetry
- ▶ Piezometer
- ▶ Pore pressure
- ▶ Remote Sensing
- ▶ Reservoirs
- ▶ Site Investigation
- ▶ Strain
- ▶ Stress
- ▶ Subsidence
- ▶ Surveying
- ▶ Thermistor
- ▶ Tiltmeter

► Tunnels
► Volcanic Environments
► Water

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Mountain Environments

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Definition

Large landforms or groups of landforms that are elevated in relation to surrounding areas and which are characterized by processes and ecosystems that reflect corresponding altitudes.

Introduction

A mountain is a landform defined by both relative and absolute characteristics. Elevation and morphology are often used to define mountains; however, it is their relative prominence over adjacent landforms and landscapes that truly set them apart. The high degree of spatial heterogeneity in topoclimatic variables makes mountains different from other terrestrial environments. Mountains commonly have a tectonic or volcanic origin though erosional mountains also exist. The associated environments show a combination of biotic and abiotic populations and processes for regions of similar characteristics.

Mountain environments exist at the interface between uplift and erosional processes. In geological terms, most represent relatively young and dynamic landscapes. They account for roughly 25% of the Earth's land surface (Fig. 1) and generally have profound implications on adjacent lowland regions by influencing their climate, hydrology, and sediment supply (Kapos et al. 2000). Geohazards in mountain environments include earthquakes, volcanoes, landslides, snow avalanches, and floods. Other surface processes such as those related to the presence of permafrost (e.g., solifluction) or deep-seated gravitational slope deformation are also considerations for hazard assessment in mountain environments.

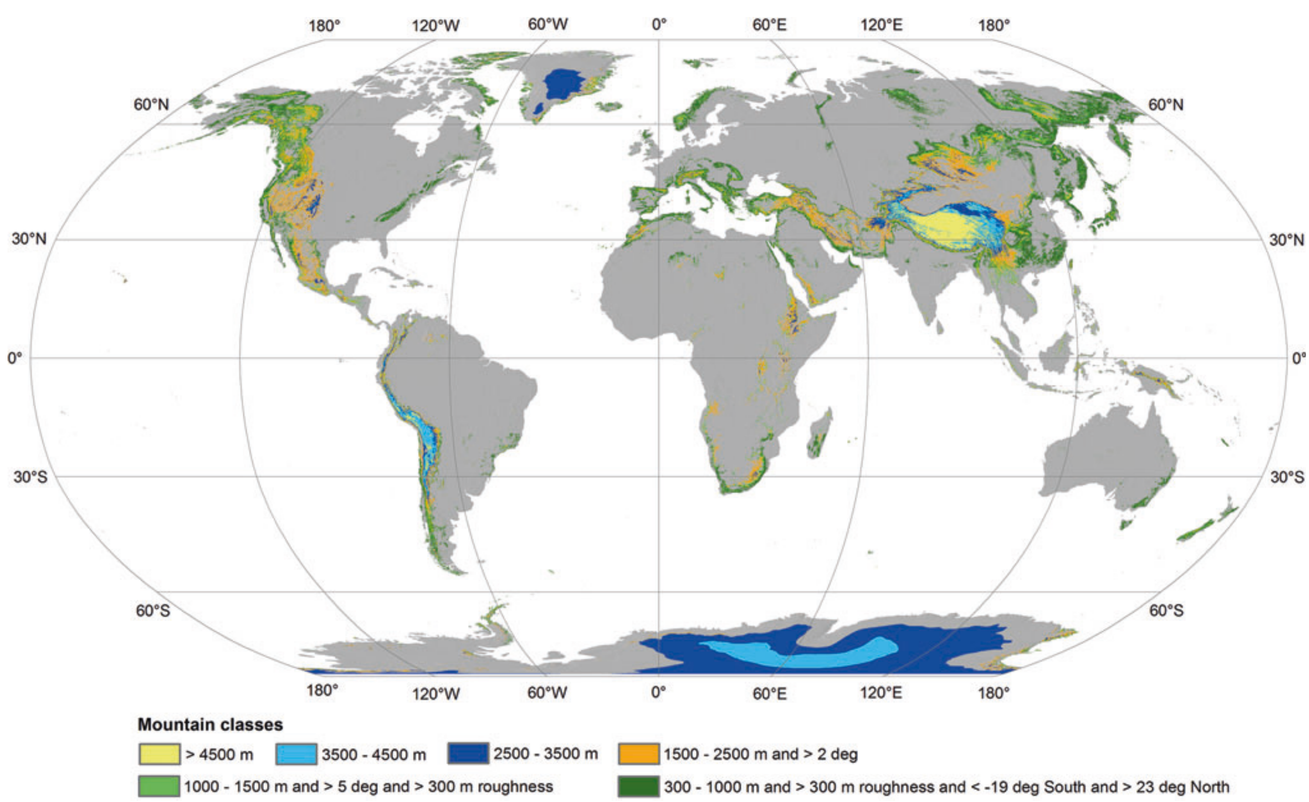
Defining mountains is typically done with relative rather than absolute criteria. Latitudinal influences on climate affect the topoclimatic characteristics of mountain ecotypes (hill, montane, alpine, and nival) and as such a comprehensive definition based on absolute criteria has yet to be achieved. Additional criteria such as elevation, prominence, summit morphology, geological structure, and orogeny are used to define and categorize mountains. The United Nations adopted a method of defining mountains globally by combining elevation, slope, relief, and latitude (Fig. 1; Kapos et al. 2000).

The Origin of Mountains

The present-day appearance of a mountain is the result of the complex interactions between uplift (tectonic, volcanic, or isostatic) and denudation/erosional (landslides, glaciers, streams, and rivers) processes across multiple spatial and temporal scales.

Tectonics

The mountain building process typically involves tectonically driven orogenic processes responsible for the folding and fracturing of bedrock into elevated, often steep, and unstable landforms. Mountains may form by the collision of tectonic plates, where the plate with the lower density will generally rise above the other. Mountains along tectonic plate boundaries are commonly associated with seismic activity and volcanism. The geology of a mountain can be sedimentary, igneous, or metamorphic, and it is common for mountain chains to have multiple geological boundaries, owing to uplift and tilting of geological structures along with the metamorphism of parent material.



Mountain Environments, Fig. 1 Mountain classes of the world (Adapted from Kapos et al. 2000)

Volcanism

Mountain landforms can also be volcanic in origin. Lava and pyroclastic materials accumulate over time forming igneous extrusive rock. Lower density magma rises through the lithosphere and may cause bulging or regional uplift forming igneous intrusive batholiths. Volcanic mountains can be found in chains or groups that reflect their tectonic setting. For example, volcanism at the divergence of the Atlantic oceanic plates formed the longest mountain chain on Earth. Strato-volcanoes are the most common volcanic mountains and result from explosive subduction zones. In contrast, shield-volcanoes (e.g., Hawaii) found over tectonic hot spots are built from lower viscosity mafic lava and have quiescent effusive eruptions.

Erosion and Weathering

Mountains can also be found at passive tectonic margins (Owens and Slaymaker 2004). In such settings, weathering and erosion of weaker geological layers can result in residual mesas, cuestas, and tablelands (plateau mountains). Other zones of weaknesses that can be preferentially eroded include faults and discontinuities. The orientation of layers, especially for sedimentary or meta sedimentary layers, greatly influences their resistance to erosion and the shape of mountains.

Mountain Climate and Weather

Mountain climate and weather exhibit a wide range of temporal and spatial variability and strongly influence landscape processes. The geographical controls on climate include latitude, continentality, altitude, and topography (Barry 2008). Other local meteorological phenomena such as convective heating can also add to the complexity of mountain climates.

Latitude

Latitude affects the amount of solar radiation received by controlling the amount of daylight hours. Due to the Earth's tilt, the amount of solar radiation decreases from equatorial regions to the poles. Solar radiation directly influences air and ground temperatures (values and range) which in turn control the fauna and flora present, type of precipitation, and presence or absence of permafrost, snow, and ice cover (Barry 2008). Topographic effects such as shading can be enhanced with latitude. Additionally, precipitation amounts vary with latitude. These latitudinal variations in temperature and precipitation are modified by the continentality, altitude, and topography.

Continentality

Continentality is a measure by which we assess the proximity of a location to the oceans. Oceans effectively moderate air temperature such that continental locations have greater annual temperature ranges than maritime locations. Continentality also affects cloud cover and precipitation. Maritime regions are wetter and cloudier than inland areas. Continentality affects not only the amount, type, and timing of the precipitation but also the character of snow accumulation. Snow pack density in maritime mountain climate tends to be higher than in continental mountain climates.

Topography

Altitude influences air temperature, pressure, density, and moisture, and consequently wind and the amount of radiation the ground surface receives. Local topography can influence the horizontal and vertical wind speeds which in turn influence the location, amount, and type of precipitation. Orographic effects are especially important in understanding mountain environments, whereby moist air masses are cooled as they are forced up a mountainside which results in precipitation. Mountains have a wet windward side and the comparatively dry leeward side.

Landscape Processes

Mountains are very active geomorphic environments where weathering, fluvial, hillslope, glacial, and periglacial processes may all be active. The previous section discussed the temporally and spatially variability of climate patterns in mountainous environments which in turn influence the intensity and timing of most landscape processes. Climate, elevation, and slope/stream gradient provide the primary control on the intensity of the fluvial and hillslope processes. The local and regional geological and glacial history of mountains plays an important role in terms of topography, sediment availability, and slope stability. Permafrost and frost-related processes contribute in many ways to the catastrophic nature of alpine hazards. The planning and development of infrastructure in mountainous environment requires detailed investigation and consideration of the variety and variability of geomorphic processes present.

Weathering

Weathering refers to the mechanical and chemical processes by which rock at or near the ground surface are broken down. Temperature and the presence of water influence the rate of weathering. Freeze-thaw and frost wedging are also important weathering processes, and often result in the accumulation of talus. Mountains in tropical climates, which are warm and wet, have higher weathering rates than mountains in desert climates.

Fluvial Processes

The source of water for mountain streams and rivers is rainfall, snowmelt, and glacier melt. Mountains are barriers to climate systems and force precipitation via orographic uplift. Because mountains have comparatively thin or in some cases no soil, rainfall water is rapidly transferred to streams and rivers resulting in flash floods. Snowmelt in late-spring and early-summer can be rapid or more gradual due to the air temperature over a week to month time-scale. Maximum yearly discharge often occurs in association with snowmelt. In mountainous regions that do not support large winter snow packs, seasonal precipitation period may control the timing of the freshet (e.g., monsoon). Glacial meltwater is slowly released throughout the summer and as such many mountain streams can be ephemeral.

Hillslope Processes

Hillslope processes refer to erosion and landslides. Erosion refers to the gradual removal of Earth, debris, and rock by water, glaciers, wind, and frost action. Erosion rates are affected by soil texture, rock mass characteristics, and slope gradient along with natural and anthropogenic changes in vegetation cover. Soil can be removed by rain splash, sheet, rill, and gully erosion. Landslides are movement of Earth, debris, and rock due to gravity and the topographic relief generated by tectonic uplift and fluvial incision. In mountain environments, the occurrence of landslides is influenced by the slope gradient, presence of vegetation, weathering rate, climate and weather, surface, groundwater regime, earthquakes, and anthropogenic landscape modification.

Glacial Processes

Pleistocene glaciation (from ~2.6 Ma to 11,700 years BP) oscillated between glacial (cold) and interglacial (warm) stages. Many mountain environments experienced significant and accelerated geomorphic changes during the Pleistocene. The manifestation of a glacial cycle includes the growth of ice sheets, ice caps, and ice fields as well as piedmont, valley, and hanging glaciers. In mountain environments, glaciers expand as small accumulations of glacier ice derived from snow, and grow to flow downstream, constrained by the valley morphology. In conditions where the glacier surface continues to rise, glacier flow can become no longer constrained by topography, and ice fields and caps are formed. Glaciers are major geomorphic erosional and depositional agents in many mountain regions. Paraglaciation refers to sediment production, transport, and deposition as directly conditioned by glaciers. Large-scale deep-seated gravitational slope deformations and a variety of related landslides are often associated with glacial retreat and paraglaciation.

Periglacial Processes

Periglacial processes refer to nonglacial freeze thaw processes that significantly contribute to local geomorphology (French 2007). The periglacial environment generally refers to areas with air temperature frequently fluctuating near or below 0 °C, limited or dwarf vegetative cover, seasonal or perennial snow, along with presence of permafrost (although not necessarily). Permafrost is ground material that remains at or below 0 °C for two or more consecutive years. Periglacial processes include frost action, permafrost-related slope failures (e.g., active layer detachment slides), melting ground ice (thermokarst), and nivation. In mountain environments, the presence, extent, and intensity of periglacial processes are controlled by mean air temperature, slope, aspect, snow cover, soil texture, hydrology, and rock mass characteristics.

Mountain Hazards

Flood and Sediment Transport

Floods are a common mountain hazard caused by fluvial processes. Their frequency and intensity are sensitive to changes in air temperature and precipitation regimes. For example, the snow depth and the characteristics of the spring freshet will influence how rapidly water is brought through the hydrological cycle. Saturated or hydrophobic soils will impede water filtration and cause overland flow, which rapidly transfers that water to the streams and rivers. The flashy character of mountain streams can cause floods and landslides. There is a continuous spectrum between fluvial to hillslope processes including, floods, debris floods, and debris flows (Jakob and Hungr 2005). Geology, topography, climate, and individual weather events control these hazardous sediment transport processes in mountain environments.

Landslides

Landslides are the gravity driven movement of Earth, debris, or rock materials. A wide range of landslide types exist with different characteristics, including: volume, velocity, activity, frequency, materials, morphologies, and triggers (Hungr et al. 2014). Mountain environments can experience many types of landslides because of their heterogeneity in geology, structure, topography, and climate. Rainfall and earthquakes are principal triggers of landslides (Davies 2014). Ground shaking associated with earthquakes will collapse soil structures and reduce shear strength of the soils. Shallow landslides can be triggered by intense (or prolonged) rainfall events, whereas deep-seated landslides often have a more complex relationship with their hydrological and hydrogeological conditions.

Landslide-dammed lakes are a secondary mountain hazard associated with landslides. They are temporary dams caused by the deposition of landslide debris in the valley bottom that impedes water flow. Landslide dams are unstable landforms with approximately 80% of them being breached within

1 year of their formation (Evans et al. 2011). Landslide dam breaches can be progressive or result in outburst floods with downstream impacts. Displacement waves may be generated when landslides enter water bodies. These potentially destructive waves can extend the footprint of damage by many kilometers.

Earthquakes

Earthquakes are frequent along tectonic plate boundaries in mountainous areas. Ground shaking in response to earthquakes is complex. Seismic waves interact with the topography which can result in local amplification of the shaking (e.g., Ashford et al. 1997). In many parts of the world, national and regional building codes have been developed to minimize the impact of ground shaking on infrastructure. Secondary hazards, in addition to or as a result of ground shaking, can be triggered by earthquakes. For example, the 2015 Gorkha Earthquake in Nepal triggered over 4,000 co-seismic and post-seismic landslides. Some of these landslides temporarily dammed rivers and created landslide-dammed lake outburst flood hazards.

Glacier Lake Outburst Floods

Glacial lake outburst floods occur when lakes drain suddenly. Drainage can happen due to the failure of soil (moraine) or ice (glacier) dams. Glacial lakes subject to catastrophic drainage may be subglacial, proglacial, englacial, or supraglacial (Owens and Slaymaker 2004). Drainage may happen only once, as with breach of moraine dams, but may also be cyclical for many ice dammed lakes. The floods can be triggered by precipitation events, glacier retreat, and subglacial eruption (e.g., 2010 Eyjafjallajökull Eruption, Iceland) along with seepage erosion or overtopping in the case of moraine-dammed lakes.

Snow and Snow Avalanches

The timing, character, and duration of the mountain snowpacks influence snow avalanche, landslide, and flood hazard potential. Snow avalanche hazard varies locally because of the influence of topography, wind, and vegetation on snow distribution, and regionally due to mountain climates (e.g., wet maritime vs. dry interior; Schaerer and McClung 2006), and the hazard changes over time due to the evolution of the snowpack structure. Because snow avalanches have a higher frequency than other mountain hazards (with the possible potential exception of rock fall), snow avalanche specialists issue daily avalanche hazard forecasts to help manage the risk to recreational users and mountain infrastructure. Landslide and flood hazard can be increased by delayed and rapid snowmelt in the spring season, or by rain-on-snow (ROS) events in the fall. These ROS events contribute large amounts of water to the mountain hydrological system which, when severe, can present an important hazard. Our understanding of these ROS events is increasing but these and the associated hazards are still difficult to anticipate.

Climate Change

Climate change is predicted to have profound impacts on mountain environments, including changes in ecology, hydrology, and geomorphology. Warming air and ground temperatures, more extreme precipitation patterns, pronounced freeze-thaw cycles, and permafrost degradation are expected to increase the frequency and magnitude of landslides and floods. Positive feedback loops acting on mass movement processes may affect landscape process chains that can evolve independently from climate change. Threshold behavior and tipping points in mountain environments may be reached. Changes, and associated lag-time effects, in the storage of sediment and ice can also be expected. Increased precipitation and air temperature can also impact mountainous periglacial processes and their interrelationships with the other landscape processes and hazards (Huggel et al. 2015).

Conclusions

Mountain environments are spatially and temporally complex systems where multiple landscape processes are active. Topography, climate, and weather events are key parameters controlling landscape processes and mountain hazards. Understanding these processes requires the contributions of numerous specialists (e.g., meteorologist, geologists, hydrologist, snow avalanche, and landslide experts) and their inputs are crucial to providing the information necessary for the safe and sustainable development in mountain environments.

Cross-References

- ▶ Avalanche
- ▶ Climate Change
- ▶ Coastal Environments
- ▶ Desert Environments
- ▶ Erosion
- ▶ Floods
- ▶ Fluvial Environments
- ▶ Geohazards
- ▶ Geology
- ▶ Glacier Environments
- ▶ Hazard
- ▶ Landforms

► Landslide
► Marine Environments
► Mass Movement
► Permafrost
► Tropical Environments
► Volcanic Environments

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Nearshore Structures

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Definition

"Nearshore structures" in the context of this entry encompass architecture suited to nearshore environments, with an emphasis on engineering geology aspects of nearshore environments. The description of structures as nearshore engineering solutions does not necessarily preclude the technology deployed beyond the nearshore, although in some cases limitations apply.

Types and Functions of Nearshore Structures

A variety of nearshore structures facilitate offshore energy development worldwide. For over 50 years, projects have exploited fossil fuels – oil and gas – and more recently an offshore renewable energy industry has emerged – predominantly wind, but also tidal and to a lesser extent wave energy. Examples of nearshore structures for oil and gas and renewable energies are illustrated schematically in Fig. 1.

Structures may be "fixed" or "floating," in the latter case enabling limited movement in response to the environmental actions acting on the structure from wind, waves, currents, tides and in places sea ice. Fixed structures may be "subsea" or "surface piercing." Subsea structures rise from the seafloor to some height within the water column and surface-piercing structures extend above the water line. Floating structures comprise a buoyant facility and mooring system.

Nearshore structures for oil and gas production are typically fixed structures rather than floating since a minimum depth, approximately 200 m, is required to provide sufficient flexibility in the riser casings that transport hydrocarbons from the seafloor to the platform, in order to tolerate the lateral movement of a floating system (Randolph and Gourvenec 2011).

Nearshore structures for renewable energy can be fixed or floating – and in the case of wave energy buoys, buoyancy provides the mechanism of power generation. Floating concepts for wind and tidal energy structures have evolved from established deep water oil and gas technologies but due to the absence of risers they can be deployed in any water depth. Wind turbines are fixed to the seabed via a range of substructures but beyond about 50 m water depth a floating wind turbine solution becomes competitive, and the first floating wind farm commenced production in late 2017. For tidal energy facilities and wave buoys, the foundation or anchoring system, is required to hold the device at the required height in the water column.

Export pipelines and cables bringing offshore energy to shore require structures at their termination and junction points, and are sometimes also provided with stabilization structures such as gravity or embedment anchors and concrete mattresses.

Scale of Nearshore Structures

Fixed platforms for oil and gas structures typically range from a few tens of meters to a few hundreds of meters in height and are limited practically by water depth as the footprint becomes too large and the structure too heavy to build and float out. The largest gravity base platform in the world, the *Troll A* condeep platform, has a total height of 472 m including the topside, and is located in the Norwegian North Sea in 305 m depth of water. The concrete substructure is 370 m tall with a foundation base 160 m in diameter (Andenaes et al. 1996). The largest jacket platform in the world, *Bullwinkle,* is located in the Gulf of Mexico in 412 m of water and stands 529 m tall. At distances from shore of 80 km and 260 km for the *Troll A* and *Bullwinkle* platforms, respectively, in conjunction with

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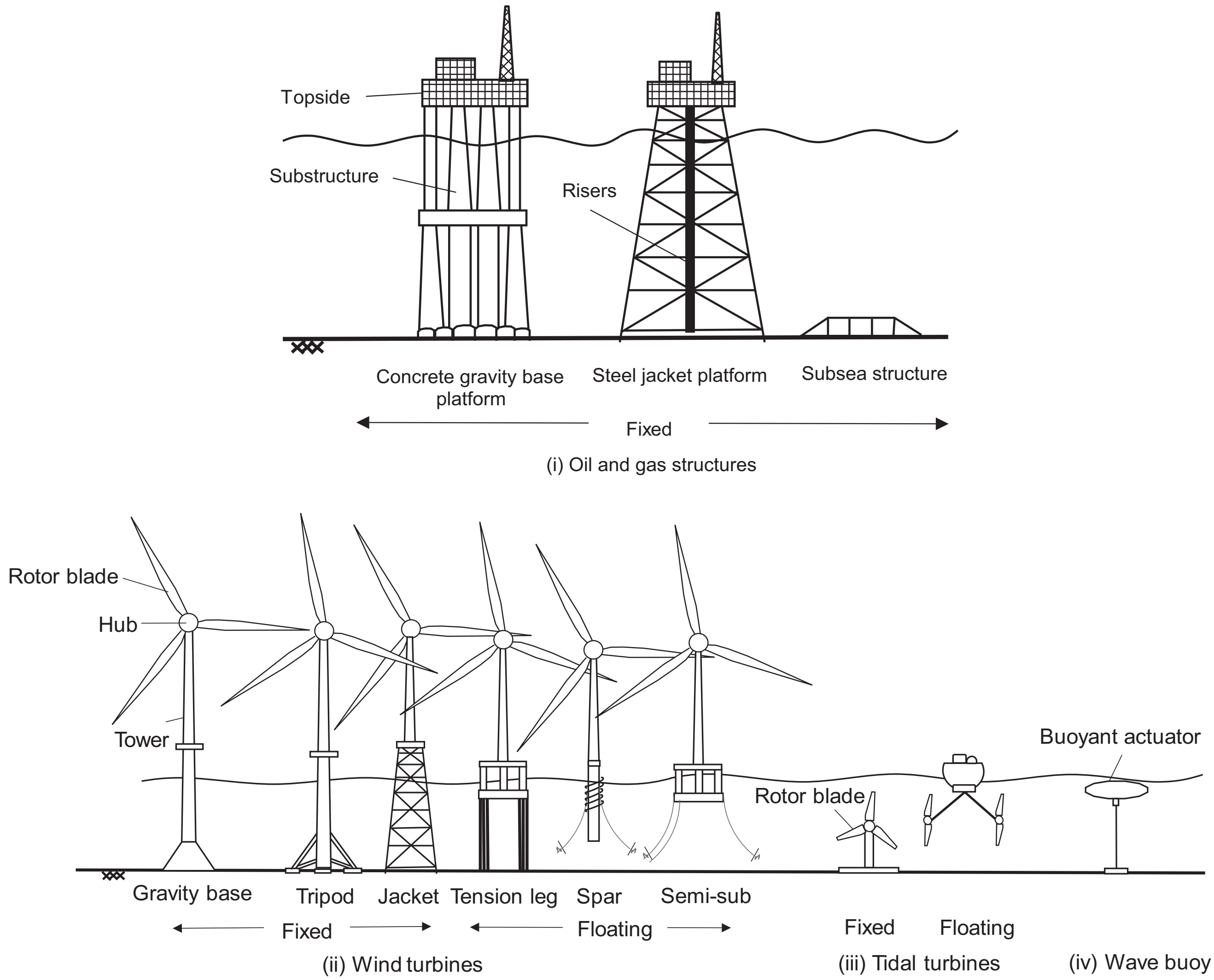
https://doi.org/10.1007/978-3-319-73568-9



Nearshore Structures, Fig. 1 Nearshore energy structures

the water depths in which they are sited, these are arguably not "nearshore" structures, but merely conform to a nearshore architecture. That these structures were installed 20 years ago and are still record holders speaks to the evolution of oil and gas architecture towards floating and subsea solutions.

Subsea structures perform a variety of functions that dictate their scale and may range from a meter or so edge length to tens of meters.

Offshore wind turbines have evolved from structures with hub height and rotor diameter of less than 20 m to over 100 m to facilitate the increase in yield from tens of kWs to several MWs (World Energy Council 2016). At the time of writing, the largest wind turbines had a maximum capacity of 9 MW and rotor diameters up to 180 m (Wind Europe 2017). With improvements in blade technology and controllability of offshore wind turbines, the continued increase in wind turbine size to facilitate increase in power output is not inevitable. Fixed offshore wind turbines have been deployed in water depths of 50 m and located as far as 100 km from shore. Fixed offshore wind structures are practically limited to water depths less than 50 m because of allowable bending deflection of the structure up to the transition piece. Tripod and jacket founded turbines can overcome this limitation to some extent but are not commonly adopted technologies. The world's first and only floating windfarm (Hywind) is installed in 100 m water depth, 25 km from shore, using technology that has the potential for deployment in any water depth, with an appropriate mooring, at any distance from shore.

Tidal turbines and wave energy buoys are too limited in number to quote typical dimensions but those in existence or development have a rotor or buoy diameters up to 20 m and are sited nearshore in relatively shallow water.

Irrespective of size and function, all offshore structures require fixing in place on the seabed with a system that is sufficiently robust to resist the design loading of the structure and ensure the function of the structure over the design life.

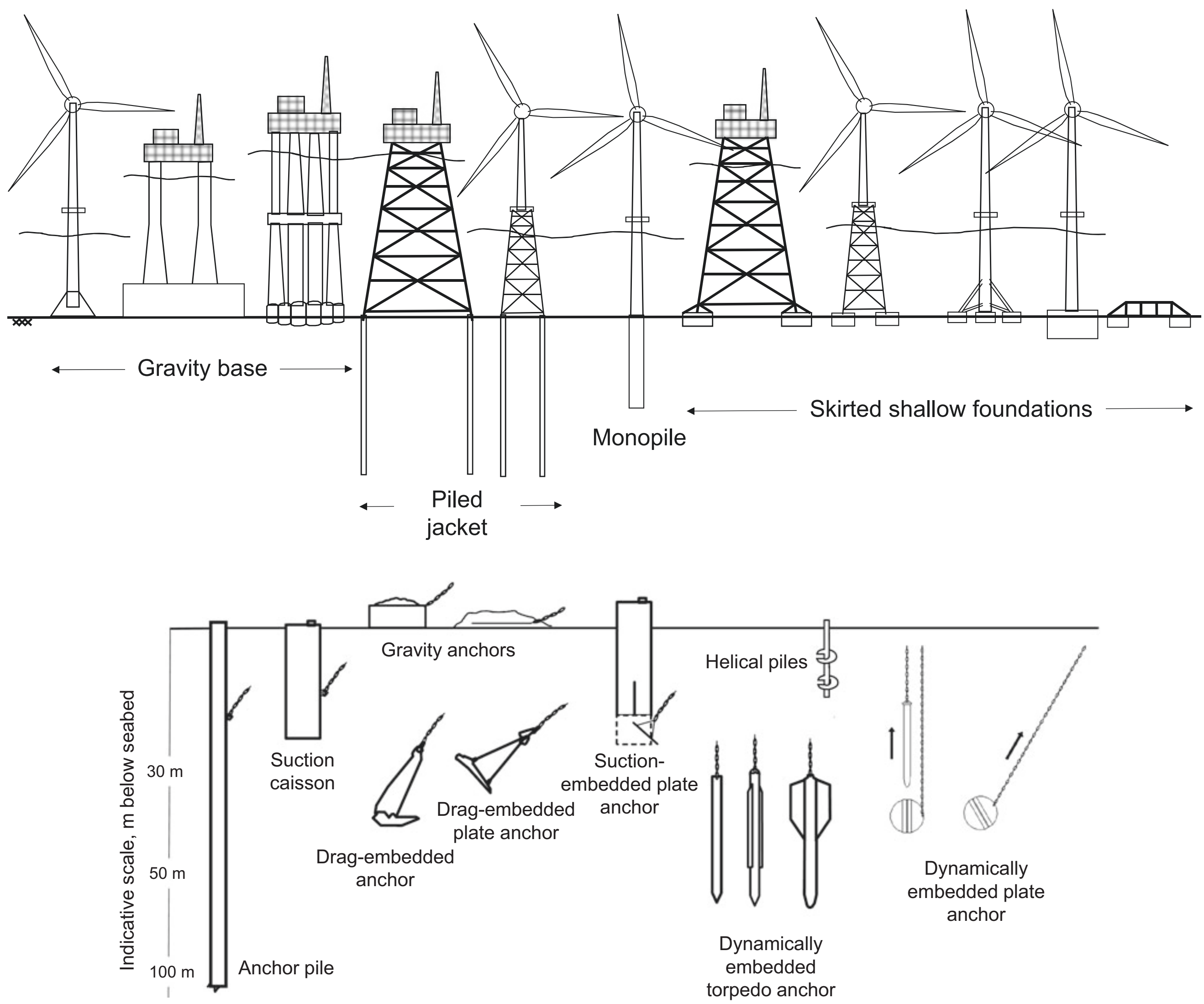
Foundations and Anchors for Nearshore Structures

Foundations support fixed structures whereas the term anchor is used to describe a foundation for a floating facility. The foundation elements used to support a fixed or floating structure can be essentially identical with the different label simply defining a different application (for example, pile and anchor pile). A range of foundation and anchor solutions for offshore energy structures are shown in Fig. 2.

Gravity foundations or anchors, irrespective of what structure is being supported or tethered, rely on their submerged self-weight to maintain stability on the seabed and resist the loads transmitted by the structure to the foundation or anchor. If self-weight and gravity are insufficient to provide the required resistance, an embedded foundation or anchoring system is required. An embedded system mobilizes the resistance of the seabed to enhance the capacity of a foundation or anchor. All foundations and anchors, whether surface gravity systems or embedded systems, derive capacity from bearing and shear resistance of the seabed that they are installed on or in. The magnitude of the available bearing and shear resistance for design is intrinsically linked to the engineering properties of the seabed.

Embedment of foundations and anchors for offshore structures can be achieved by driving, drilling and grouting, jacking or dragging, or using suction or torque, or by dynamic free-fall. The ease or practicality of installation of different foundation or anchor types is intrinsically dependent on the engineering properties of the seabed.

Seabed conditions, and the influence in respect of installation and in-service performance, are thus a key driver in selection of a foundation or anchor type for an offshore structure.



Nearshore Structures, Fig. 2 Foundations and anchoring systems for nearshore structures

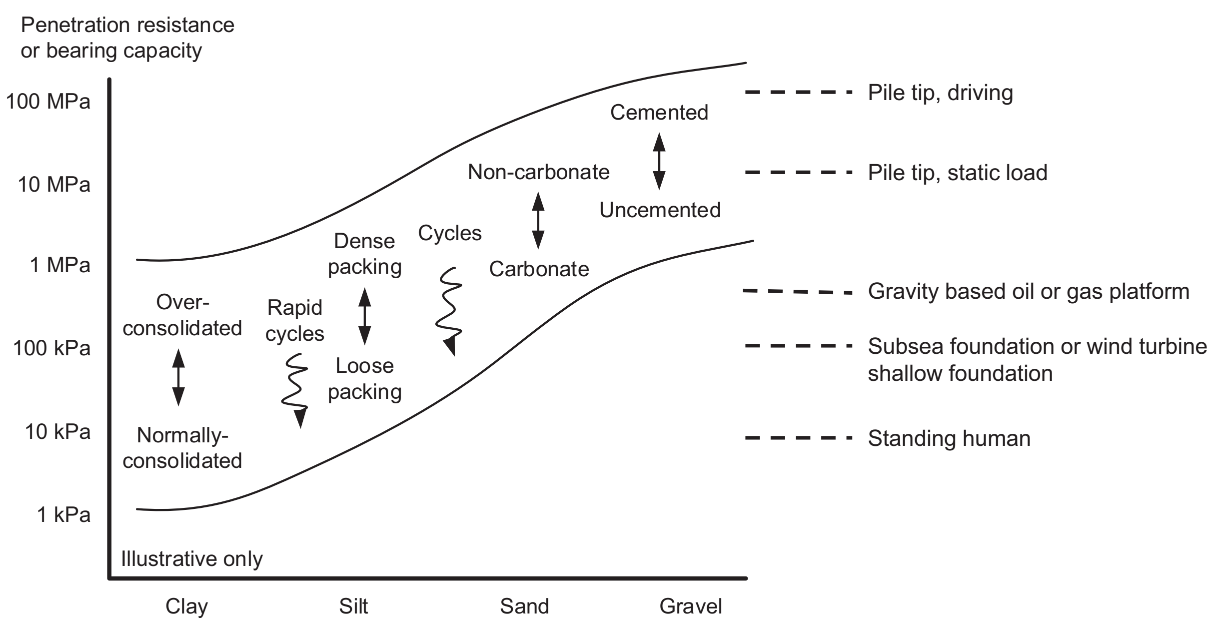
Nearshore Seabed Conditions

Nearshore seabeds may comprise almost any geomaterial, ranging from clay to rock. Substrates may range from competent granular materials to crushable carbonate sand, or clay and muds that may have strengths varying over several orders of magnitude depending on depositional history. Nearshore seabeds can also present extremely complex features and processes, including relics of historic buried shorelines or glaciation, geohazards, and scour. The range of offshore geomaterials in an engineering context is shown in Fig. 3. Different types of geomaterial are organized by grain size along the abscissa, small to large plotted left to right, and the strength of the deposit, expressed as a penetration resistance or capacity in stress units, is plotted on the ordinate. In an engineering geology context, clay is defined by a particle size below 0.002 mm; silt 0.002–0.063 mm, sand 0.063–2.0 mm, and gravel 2.0–63 mm (ISO 2016). The labels to the right show indicative values of stress required to install or support different foundation types. The stress exerted by an "average" person is also shown to give context to the scale, noting that a person standing on a soft normally consolidated seabed of a drained ocean would sink – until the increase of the strength of the soil with depth became sufficient to support their weight. Similarly, pile driving may not be needed in clay soils, since the tip load generated by the submerged self-weight of the pile may be sufficient to push the pile down through the clay.

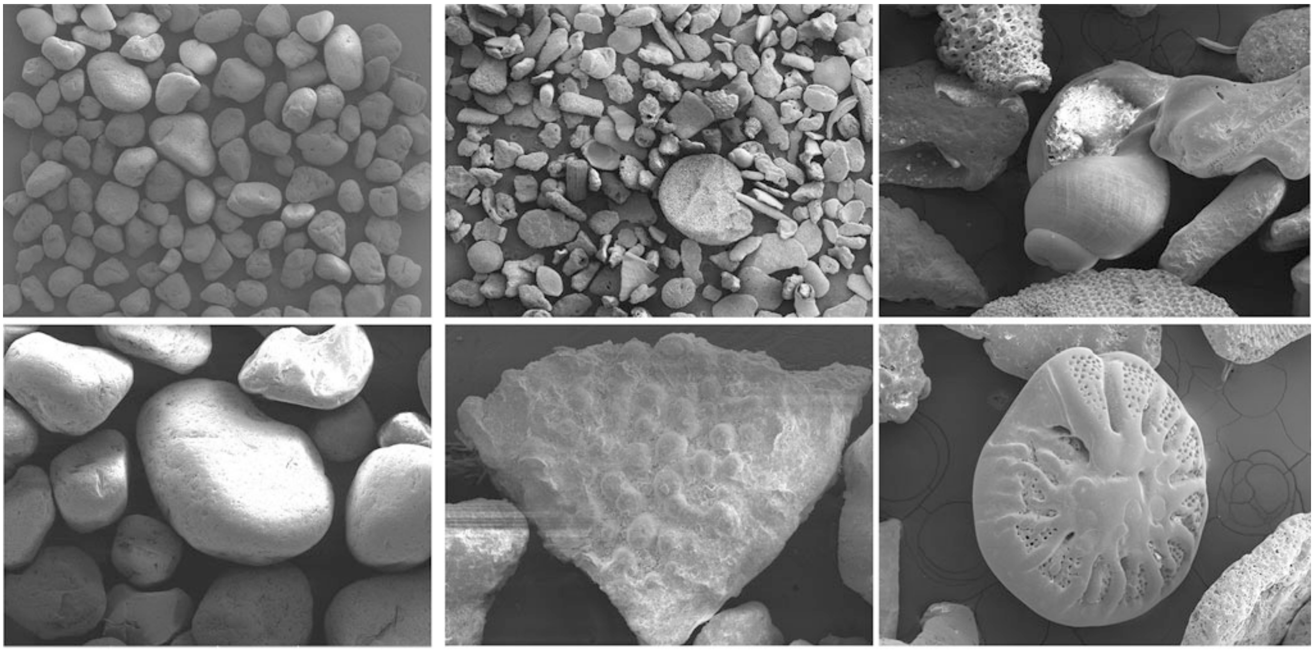
The strength of each seabed material can vary by orders of magnitude within each soil type depending on state in terms of grain mineralogy and microstructure, depositional history, and postdepositional physio-biological-chemical changes and grain packing. For example, for sand of similar particle size, silica-based granular materials are hard due to the quartz mineral and are generally a competent foundation material with a reliable geotechnical response; in contrast, carbonate deposits, which are in the most part comprised of the skeletal remains of marine plant and animal life, are constituted of soft and fragile calcium carbonate. The fragility of the calcium carbonate coupled with the highly variable grain shapes and sizes can lead to apparently high but easily degraded frictional strength and high compressibility. These characteristics can adversely affect, for example, driven pile capacity or shaft capacity of dynamically embedded anchors. They may also lead to large, potentially differential, settlements under shallow foundations, and significant strength degradation under cyclic loading. Calcium carbonate is also prone to postdepositional biochemical processes that can lead to cementation, causing spatially variable cemented lenses of material that can lead to pile toe buckling or pile refusal.

Figure 4 shows images from scanning electron microscopy that illustrate the differences in the microstructure of a silica and carbonate sand and the variability of grain shape and size in the carbonate sand.

Nearshore seabeds, with water depth less than ~150 m, were subaerial land surfaces during previous sea-level lowstands, meaning they were exposed and potentially subjected to glaciation in many parts of the northern hemisphere. A relic of the unloading from past glaciation is relatively homogenous, highly over consolidated stiff and strong clay that presents a reasonably desirable foundation material for



Nearshore Structures, Fig. 3 Variation in installation resistance or foundation capacity in different seabed geomaterials



Nearshore Structures, Fig. 4 Microstructure of silica and carbonate nearshore sand deposits from scanning electron microscopy

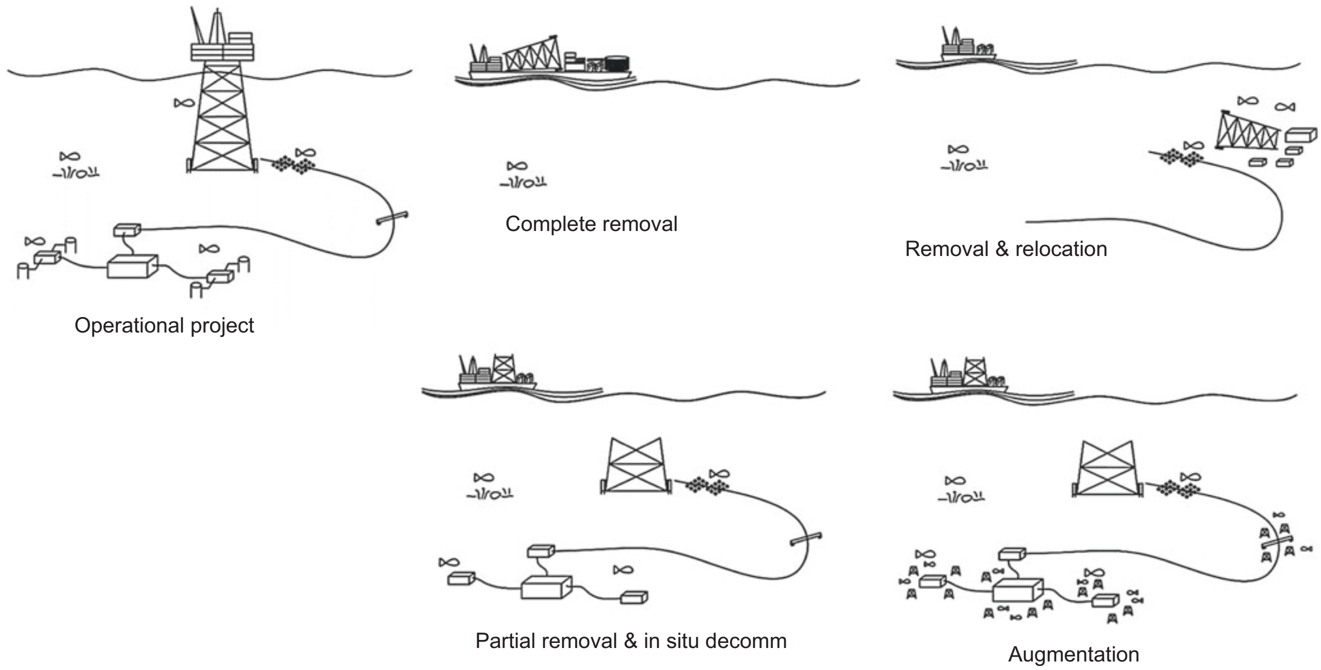
offshore development. However, postglacial till deposits are unsorted sediments derived from the erosion, entrainment and deposition of material by a moving ice mass. The high lateral and vertical variability in the nature and strength of tills, and related submerged landforms, is a key engineering challenge. Seabed conditions can vary significantly with depth and two locations a kilometer apart may have completely different stratigraphy presenting clear challenges for site characterization and design. Historical shorelines resulting from sea level changes have particularly high variability over short distances, for example, with competent rock and soft normally consolidated clay nearby. Seabed variability is particularly challenging to renewable energy developments as these typically cover tens of square kilometers of seabed with multiple structures that are required to be placed in a fixed array. Effective and efficient methods of site characterization over large areas are required to ensure a sufficiently detailed picture of a site can be practically constructed from site investigation data.

Ongoing seabed processes also pose a challenge to offshore engineering. For example, seabed scour that causes the net removal of material from adjacent to a seabed structure occurs due to the presence of the structure disturbing the flow, causing preferential transport of sediment close to the structure. The extent of scour is dependent on a number of factors, including the near bed flow conditions (the existence of currents, waves, or both), the local seabed sediment properties, and the shape of the structure. For deeply embedded structures, scour will usually reach some maximum (or equilibrium) erosion depth around the structure in a given flow condition. For shallowly embedded structures (including shallow foundations and pipelines), scour can cause undermining of the structure leading to tilt and settlement. Scour potential needs assessment as part of offshore foundation design and scour protection around the foundation may be needed, particularly nearshore, in shallow to medium water depths, where hydrodynamic energy tends to be high.

Future Challenges for Nearshore Structures

Globally, offshore infrastructure amounts to thousands of platforms, a range of seabed structures, many thousands of kilometers of pipeline and tens of thousands of wells, much of it located nearshore. Offshore oil and gas construction booms in the 1960–1980s have resulted in a significant asset base approaching or reaching the end of their production life and present a significant decommissioning challenge. Offshore wind turbines, although less mature, are already being decommissioned having reached the end of a shorter design life and to make way for larger, more powerful turbines.

Considering the potential scale of the challenge, the Gulf of Mexico hosts almost 3500 facilities; in excess of 1700 offshore installations are sited in South East Asia, nearly half of which are older than 20 years and due to be retired; over 600 fields are expected to cease production in the Asia-Pacific in the next 10 years; in Australia, there are 110 offshore



Nearshore Structures, Fig. 5 Potential decommissioning options for nearshore structures

oil and gas platforms and subsea structures many approaching the end of production life and only a small number of early projects have already been decommissioned; more than 550 platforms and subsea structures and more than 2500 wind turbines are currently installed in the North Sea (Gourvenec and White 2017).

Offshore decommissioning costs for just the oil and gas infrastructure in the North Sea are forecast to 47 bn GBP (US$66 bn) to 2050 – with an uncertainty of ±40% (Oil and Gas Authority 2016) and total global offshore decommissioning expenditures expected to amount to US$210 bn over the period 2010–2040 (Foxwell 2016). In the North Sea, only 12% of the infrastructure has been decommissioned to date, and 100 platforms are expected to be decommissioned on the UK and Norwegian continental shelves over next 10 years – along with 1800 wells and 7500 km of pipeline (Oil and Gas UK 2016).

The scale of the offshore decommissioning challenge is increasingly well understood – what is less understood is the life cycle effect of decommissioning alternatives – and the evidence base and decision tools to determine which alternative realizes the optimal outcome.

A range of potential decommissioning alternatives for offshore infrastructure are illustrated schematically in Fig. 5. These span the ubiquitous base case of complete removal; removal and relocation – best known through the US rigs to reef program; and *in situ* decommissioning, potentially with augmentation of purpose built artificial reef modules to stabilize structures left *in situ* and maximize the benefit to marine ecology or fisheries.

Determination of the most appropriate outcome on a project-by-project basis requires a multicriteria, multisector, transdisciplinary decision framework to inform all decommissioning outcomes and for all infrastructure types. The engine of the framework requires a bank of weighted evidence to assess whether infrastructure can be removed, relocated, or left *in situ* and determine the impact of the spectrum of options, reflecting multiple disciplines and a diversity of opinions (Gourvenec 2017).

An opportunity exists, with the right evidence base, to transform decommissioning of offshore infrastructure from the current base case of complete removal, borne out of guidelines to prevent sea dumping, to a broader portfolio of options including *in situ* decommissioning to ensure the minimum environmental impact of the decommissioning challenge ahead and the maximum positive outcome for other ocean users.

Summary

This chapter has presented a selection of types and functions of nearshore structures that facilitate offshore energy development worldwide. Structures for harnessing fossil fuels and renewable energies are presented in the context of design drivers and constraints, highlighting technology transfer from the established offshore fossil fuel industry to the emerging offshore renewables industry. The interaction of the structures with the seabed, through the foundation and anchoring systems that keep the structures in position, and meeting the basis of design is discussed. The extreme variability of seabeds (at all scales) as a result of origin and postdepositional processes and the challenges these pose to offshore engineering are highlighted.

Cross-References

- ► Engineering Geology
- ► Foundations
- ► Geology
- ► Geotechnical Engineering
- ► Infrastructure
- ► Marine Environments
- ► Sediments
- ► Site Investigation
- ► Soil Mechanics

Acknowledgment The SEM images presented in Fig. 4 were created with the facilities and technical support of the Australian Microscopy & Microanalysis Research Facility at the Centre for Microscopy at the University of Western Australia.

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Noncohesive Soils

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Synonyms

Cohesionless soils; Granular soils

Definition

Noncohesive soils are mineral soils that exhibit granular characteristics in which the grains remain separate from each other and do not form clods or hold together in aggregates of particles. Noncohesive soils also may be called cohesionless soils or granular soils. They tend to transmit water readily (relatively high hydraulic conductivity or permeability) and exhibit shear strength that has only a friction component with zero cohesion intercept. In the soil classification system used by soil scientists, noncohesive soils include sand, loamy sand, and possibly sandy loam if the silt-sized particles are nonplastic or nonsticky. In the soil classification system used by engineers (ASTM 2011), soils are classified initially by the amount of the soil mass that passes through a standard #200 sieve (200 openings per inch or 200 openings per 25.4 mm) with 0.075-mm openings (ASTM 2009); particles that are retained on a #200 sieve are classified as coarse-grained (sand, gravel, cobbles, boulders), whereas particles that pass through a #200 sieve are classified as fine-grained (silt, clay), and typically referred to as "fines."

Coarse-grained soils have more than 50% by weight larger than the #200 sieve. By this classification, all noncohesive soils are coarse-grained soils. However, coarse-grained soils are further classified by amount of fines that are included. "Clean" sandy or gravely soils are those that contain less than 5% fines (well-graded or poorly graded sand or gravel); sandy or gravely soils "with fines" contain more than 12% fines (silty or clayey sand or gravel). Coarse-grained soils containing between 5% and 12% fines are given dual classifications. Clean sand and gravel are noncohesive soils. Sand and gravel with silt may be noncohesive if the silt is

Noncohesive Soils, Fig. 1 Sand dunes in Death Valley National Park, California, USA, exhibiting angle-of-repose slopes associated with aeolian deposition of sand (Photo by Jeff Keaton, 22 Dec 2013)

nonplastic, which requires the determination of the Atterberg limits (ASTM 2010). Sand and gravel with clay or plastic silt would exhibit cohesive behavior.

The Atterberg limits consists of two measured properties (liquid limit and plastic limit) and one calculated parameter (plasticity index). Determination of noncohesive behavior of soils would require only the plastic limit test. All Atterberg limits are determined on samples of soil that pass the #40 sieve, which has 0.42-mm openings (medium sand size and smaller, including fines that may be part of the soil material). The plastic limit is the water content at which the soil-water paste changes from a semisolid to a plastic consistency as it is rolled into a 3.175-mm (1/8 in.) diameter thread in a standard test. Nonplastic soils are noncohesive; the soil-water paste breaks apart regardless of its water content as attempts are made to roll it into a standard thread. Noncohesive soils can have temporary cohesive-like behavior caused by the surface-tension effects moisture, which explains why a sand castle can be built with moist beach sand, but not with dry dune sand (Fig. 1), and why the sand castle collapses as the sand dries.

Cross-References

- ▶ Atterberg Limits
- ▶ Classification of Soils
- ▶ Clay
- ▶ Cohesive Soils
- ▶ Liquid Limit
- ▶ Plastic Limit
- ▶ Plasticity Index
- ▶ Sand
- ▶ Silt
- ▶ Soil Laboratory Tests
- ▶ Soil Properties

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Normal Stress

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Definition

Normal stress is stress acting perpendicular or normal to a plane of interest or a reference plane.

The stress tensor in a Cartesian coordinate system is represented by three normal stress components and six shear stress components. If a reference cube with orthogonal faces is rotated to a specific orientation in a stress field, the magnitudes of all the shear stress components become zero, and all the normal stress components become principal stresses.

In direct shear tests of soil or rock samples, a vertical load is applied to the sample while part of the sample is shifted horizontally relative to part of the sample which is held in position. The vertical load divided by the area of the sample is taken to be the normal stress of the test. Typically, the vertical load bears on a steel plate that has been manufactured to fit in the testing device to ensure that the load is applied uniformly over the surface of the sample.

Cross-References

- ▶ Stress

O

Organic Soils and Peats

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Definition

Organic soils are those containing a significant amount of organic material recently derived from plant remains. Technically any material that contains carbon is called "organic." However, engineers and geologist use a narrower definition when applying the term to soils. This implies it needs to be "fresh" and still in the process of decomposition, and thus retain a distinctive texture, color, and odor. Some soils contain carbon that is not recently derived from plants and thus are not considered organic in this context. For example, some sand contains calcium carbonate (calcite), which was chemically precipitated. Peats are organic soils with more than 75% organic matter. The identification of organic soils and peats is very important because they are much weaker and more incompressible than inorganic (mineral) soils.

Introduction

In agricultural use, raised bogs and fens have to be drained to adjust the water and air in the soil to meet up with the conditions of cultivated or pasture plants. The loss of water from organic soils and peats by drainage, followed by oxidation, leads to compaction and subsidence of the surface. The drainage of these soils increases the release of CO_2 and N_2O but reduces the release of CH_4. Release rates depend on the temperature of organic soils and peat, level of groundwater, and burning CO_2, but CH_4 and N_2O are also released. In the process of organic soils and peat extraction, the greenhouse gas (GHG) sink function of the land is lost. Emissions also occur in the preparation of the surface for removing vegetation and ditching; extraction of peat and its storage; and transportation, combustion, and after-treatment of the cut area. Combustion accounts for more than 90% of the greenhouse gas emissions.

As mentioned above, the organic contents are essential remains of plants for which the rate of accumulation is faster than the rate of decay. The precise definition of peat however varies between that of soil science and engineering. Soil scientists define peat as organic soil with organic content of greater than 35%. To a geotechnical engineer, however, all0 soils with an organic content of greater than 20% are known as organic soil, while "peat" is an organic soil with organic content of more than 75%. The engineering definition is essentially based on the mechanical properties of the soil. It is generally recognized that when a soil possess organic content of greater than 20%, the mechanical criteria of the conventional mineral soil (silt and clay) can no longer be generally applied.

Sometimes, the plant fibers are visible but, in the advanced stages of decomposition, they may not be evident. Peat will turn into lignite or coal over geologic periods of time under appropriate conditions. Also, the fresher the peat, the more fibrous material it contains, and as far as engineering is concerned, the more fibrous the peat, the higher is the shear strength, void ratio, and water content. In fact, the property of the peat is greatly dependent on the mode of formation of peat deposits. This means that peat at different locations will have different properties.

Commonly, the classification of peat is developed based on fiber content, organic content, and ash content following combustion. Decomposition is the breakdown process of the plant remains by the soil microflora, bacteria, and fungi during aerobic decay. In this procedure, as mentioned earlier, there is disappearance of the peat structure and change in the primary chemical composition of peat. At the final stage, carbon dioxide and water are the products of the

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decomposition process. The degree of decomposition varies throughout peat since some plants or some parts of the plants are more resistant than others. Also, the degree of decomposition of peat depends on the combination of conditions, such as the chemistry of the water supply, the temperature of the region, aeration, and the biochemical stability of the peat-forming plant.

In tropical countries like Malaysia and Indonesia, peat is generally termed as basin, and valley peat. Basin peat is usually found on the inward edge of the mangrove swamps along a coastal plain. The individual peat bodies may range from a few to 100,000 hectares and they generally have a dome-shaped surface. The peat is generally classified as the ombrogenous or rain-fed peat, and is poor in nutrients. Due to coastal and alluvial geomorphology, these are often elongated and irregular, rather than having the ideal round bog shape. The depth of the peat is generally shallower near the coast and increases land inward, locally exceeding more than 20 m. Water plays a fundamental role in the development and maintenance of tropical peat. A balance of rainfall and evapotranspiration is critical to sustainability of the system. Rainfall and surface topography regulates the overall hydrological characteristics of the peat land. Peat land is also generally known as wetland or peat swamp because of its water table, which is close to or above the peat surface throughout the year and fluctuates with the intensity and frequency of rainfall.

Peat or organic soils may also occur as deposits buried beneath, and covered by, inorganic alluvial soils. These often are difficult to detect, and can be a source of large differential settlements. Organic deposits also may be mixed with inorganic soils, such as silt and clay, producing soils less problematic than as peat but more problematic than inorganic deposits.

Classification of Organic Soils and Peats

In organic soils and peats, the degree of decomposition is usually assessed by using von Post's (1922) scale. This scale is based on factors such as botanical composition, degree of humification, and the color of peat water after squeezing. In this classification system, there are 10 degrees of decomposition ranging from H_1 (very fibrous) to H_{10} (very few fibers), which represent the state of decomposition/decay of the organic plant remains. The higher the number in von Post's scale, the greater is the degree of decomposition. Ulusay et al. (2010) suggest that peats near the surface fall into the H_3 and H_4 categories, but with increasing depth, would be classified as H_5–H_7.

According to the American Society for Testing and Materials standard (ASTM 1990), peat classification has been narrowed to only three classes based on fiber content, ash content, and acidity of soil. In terms of fiber content, peat is divided in to three groups: (i) fibric (fibrous; least decomposed with more than 67% fiber content), (ii) hemic (semifibrous; intermediate decomposed), and (iii) sapric (amorphous; most decomposed with less than 33% fiber content).

According to the classifications of von Post (1922) and ASTM (1990) from the fiber content view point, the groups H_1–H_4 containing more than 66% fibers are called fibrous peat. Hemic peat includes groups H_5–H_7 and consists of 33–66% fibers. The other three groups, i.e., H_8–H_{10} having less than 33% of fibers is sapric peat.

Physical Characteristics of Organic Soils and Peats

Since the main component is organic matter, peat is very spongy, highly compressible, and combustible. These characteristics give peat distinctive geotechnical properties compared with other inorganic soils consisting largely of clay and sand particles (Deboucha et al. 2008). Table 1 shows the variation in physical and chemical properties of peat.

Problems with Organic soils and Peat from Geotechnical Engineering Viewpoint

Organic soils and peats are considered as problematic in relation to design parameters used by geotechnical engineers because their engineering characteristics are inferior to other soft soils making these unsuitable for construction in its natural state (den Haan and Kruse 2006). For example, further humification of the organic constituents alter the soil mechanical properties such as compressibility, shear strength, and hydraulic conductivity (Huat et al. 2013).

Therefore, buildings on peat are usually supported on piles. However, the surrounding ground may still settle. Also, sometimes a construction line such as road embankment is not only subject to localized sinking, slip failure, and massive primary and long-term settlement but also variable behavior due to linear variations in peat material properties (Kazemian et al. 2011b).

The bearing capacity is apparently influenced by the water table and the presence of subsurface woody debris. Lowering of ground water may cause shrinking and oxidation of peat leading to humification with consequent increase in permeability and compressibility. Even if failure can be avoided, it is also inevitable that soft water-logged soil and peat takes a long time to settle when loaded by an embankment or soil fill. Under these conditions, the embankment will settle progressively into the ground below, even if the soils do not fail by displacement.

Organic Soils and Peats, Table 1 Physical and chemical properties of organic soils and peats (Kazemian et al. 2011a)

Peat type	Natural water content (w, %)	Bulk density (Mg/m^3)	Specific gravity (G_s)	Acidity (pH)	Ash content (%)	Reference
Fibrous-woody	484–909	–	–	–	17	Colley (1950)
Fibrous	850	0.95–1.03	1.1–1.8	–	–	Hanrahan (1954)
Peat	520	–	–	–	–	Lewis (1956)
Amorphous and fibrous	500–1500	0.88–1.22	1.5–1.6	–	–	Lea and Brawner (1963)
	200–600	–	1.62	4.8–6.3	12.2–22.5	Adams (1965)
	355–425	–	1.73	6.7	15.9	
Amorphous to fibrous	850	–	1.5	–	14	Keene and Zawodniak (1968)
Fibrous	605–1290	0.87–1.04	1.41–1.7	–	4.6–15.8	Samson and La Rochelle (1972)
Coarse fibrous	613–886	1.04	1.5	4.1	9.4	Berry and Vickers (1975)
Fibrous sedge	350	–	–	4.3	4.8	Levesqe et al. (1980)
Fibrous sphagnum	778	–	–	3.3	1	
Coarse fibrous	202–1159	1.05	1.5	4.17	14.3	Berry (1983)
Fine fibrous	660	1.05	1.58	6.9	23.9	Ng and Eischen (1983)
Fine fibrous	418	1.05	1.73	6.9	9.4	
Amorphous granular	336	1.05	1.72	7.3	19.5	
Peat Portage	600	0.96	1.72	7.3	19.5	Edil and Mochtar (1984)
Peat Waupaca	460	0.96	1.68	6.2	15	
Fibrous peat Middleton	510	0.91	1.41	7	12	
Fibrous peat Noblesville	173–757	0.84	1.56	6.4	6.9–8.4	
Fibrous	660–1590	–	1.53–1.68	–	0.1–32.0	Lefebvre et al. (1984)
Fibrous peat	660–890	0.94–1.15	–	–	–	Olson and Mesri (1970)
Amorphous poeat	200–875	1.04–1.23	–	–	–	
Peat	125–375	0	1.55–1.63	5–7	22–45	Yamaguchi et al. (1987)
Peat	419	1	1.61	–	22–45	Jones et al. (1986)
Peat	490–1250	–	1.45	–	20–33	Yamaguchi et al. (1987)
Peat	630–1200	–	1.58–1.71	–	22–35	Nakayama et al. (1990)
Peat	400–1100	0.99–1.1	1.47	4.2	5–15	Yamaguchi 1990
Fibrous	700–800	~1.00	–	–	–	Hansbo (1991)
Peat (Netherlands)	669	0.97	1.52	–	20.8	Termaat and Topolnicki (1994)
Fibrous (Middleton)	510–850	0.99–1.1	1.47–1.64	4.2	5–7	Aljouni (2000)
Fibrous (James Bay)	1000–1340	0.85–1.02	1.37–1.55	5.3	4.1	

Soil Improvement for Construction on Organic Soils and Peats

Soil improvement plays a vital role in geotechnical engineering because it is the only way to stabilize and enhance the properties of soils. Most of the time, the improvement is focused on modifying and stabilizing the soil. Therefore, the ground improvement technique aims to increase the density and shear strength of peats to aid stability, the reduction of compressibility, influencing permeability to reduce and control ground water flow or to increase the rate of consolidation, or to improve homogeneity.

There are various methods for ground improvement such as excavation, displacement, replacement, or reinforcement to enhance soil strength and stiffness like preloading and stage construction, stone columns, piles, thermal precompression and preload piers; or by reducing driving forces by using light-weight fill; and chemical admixture such as with cement and lime (Edil 2003; Kazemian and Huat 2009b). The chemical admixtures can be applied either by a deep in situ mixing method (lime-cement columns) or as surface stabilizer (Kazemian et al. 2010). Compared to other methods, the chemical admixtures are a more economical and time-saving option. Ahnberg et al. (1995), Ding (2000), Hebib and Farell (2003), Hashim and Islam (2008), Kazemian and Huat (2009a), and Kazemian et al. (2009) conducted a study to determine the various engineering properties of peat and observed the effect on strength after stabilization by soil-cement column technique. Ordinary Portland cement, bentonite, and well-graded sand are used as binders.

Summary and Conclusion

In organic soils and peats, the organic contents are remains of plants the rate of accumulation of which is faster than the rate of decay. Since the main component is organic matter, most organic soils and peats are very spongy, highly compressible, and combustible. These characteristics relate to distinctive geotechnical properties compared with inorganic soils. Physical properties of organic soils and peats are mainly dependent on porosity and pore-size distribution. These in turn are related to particle-size distribution. Peat has very high natural water content due to its natural water-holding capacity. The high natural water-holding capacity is due to organic coarse particles (fibers) which can hold a considerable amount of water since the fibers are very loose and hollow. It has been emphasized that the water content of peat may range from 200 to 2000% which is quite different from that for clay and silt deposits which rarely exceed 200%. The bulk density of peat is both low and variable compared to mineral soils. The average bulk density of fibrous peat is around the unit weight of water (9.81 kN/m3).

For organic soils with an organic content of 75% and greater, the specific gravity is in the range from 1.3 to 1.8 with an average of 1.5. The natural void ratio commonly ranges from 5 to 15 and it is around 25 for fibrous peat. Permeability plays a vital role in the properties of these soils because it controls the rate of consolidation and increase in the shear strength of the soil. Generally, organic soils and peats are in an acidic condition and the pH value often lies between 4 and 7. The most common exchangeable cations in peats are Ca2+, Mg3+, Al3+, K+, Na+, (NH4)+. The cation exchange capacity CEC will increase with an increase in pH value and the exchangeable cation concentration. Among the peats, the CEC for fibrous peat is larger than others.

There are various methods for ground improvement such as excavation, displacement, or replacement, ground improvement or reinforcement to enhance soil strength and stiffness like preloading and stage construction, stone columns, piles, thermal precompression and preload piers; or by reducing driving forces by light-weight fill; and chemical admixture such as cement and lime. The chemical admixtures can be applied either by a deep in situ mixing method (lime-cement columns) or as surface stabilizers. Compared to other methods, the chemical admixtures are a more economical and time-saving option.

Cross-References

► Classification of Soils
► Ground Preparation

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P

Peels

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Definition

A method for directly sampling materials in the face of a trial pit or trench, using layers of resin and a backing net, for future use and reference.

The sedimentary structure of an anthropogenic or natural stratum can be precisely sampled by making a peel of the face of a trial pit or trench. This method can be used to examine depositional structures, structures associated with the liquefaction and fluidization of a stratum and, through testing, the presence of contaminants (Nirei et al. 2016). The basic principle of relief peeling is that liquid resin is impregnated into an exposed stratum, a backing medium is placed on the face, and the hardened layer of the stratum is peeled off like a film. The thickness of the layer to be peeled off depends on the permeability of the stratum, which makes it easy to sample fine sedimentary structures. The stages in the preparation of a peel are:

1. **Shape the exposure**: The face of the stratum to be sampled is ground until it is flat and smooth. If the prepared surface is dry it should be moistened by spraying water from an atomizer. The prepared face should be photographed

Peels, Fig. 1 Using a brush, apply a high-strength resin, such as epoxy resin, to the hardened surface of the isocyanate resin

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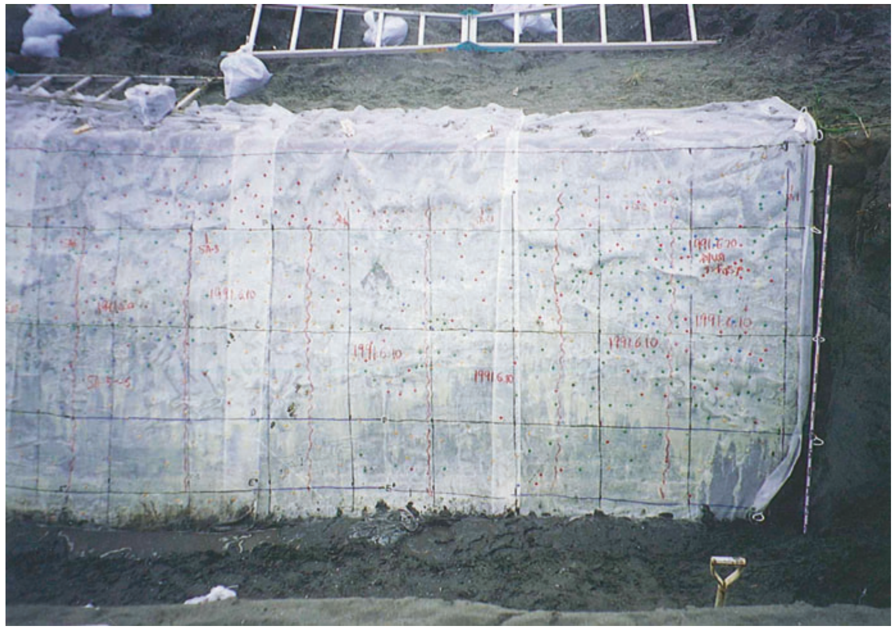
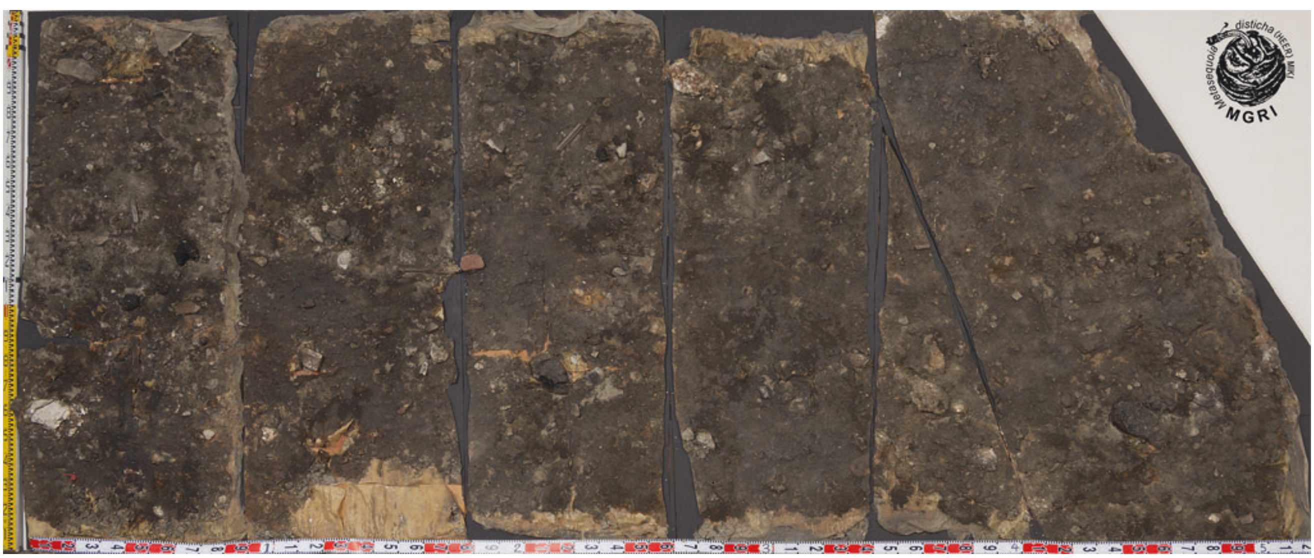


Peels, Fig. 2 Immediately place a net with a mesh size of a few mm to a few cm on the surface. Apply the resin over the net and to cut the layer to a size suitable for carrying, and mark cutting lines with an oil-based marker



Peels, Fig. 3 Large peel with large slag fragments mainly containing Cr^{+6} and Fe exhibited by the Medical Geology Research Institute (MGRI) on the wall of Marker's Lecture Centre, Katori City, Japan

2. **Apply isocyanate resin**: Using an atomizer, sufficiently impregnate isocyanate resin diluted with acetone into the area of the face that is being peeled off. If the layer is sand, a resin is prepared by adding four to five times as much acetone as resin. If the layer is mud, the resin mixture is prepared by adding eight to ten times as much acetone as resin. The resin hardens in 30–60 min. As soon as it hardens, the next step is undertaken.
3. **Apply backing resin**: A high-strength resin, such as epoxy resin, is applied to the hardened surface of the isocyanate resin using a brush then a net with a mesh size of a few mm to a few cm is immediately placed over the surface and more resin is applied over the net (Fig. 1). The net should be placed so that it protrudes a few dozen cm above the layer to make it possible to peel the layer while lifting it. The resin hardens in a few hours.
4. **Preparing to cut the preparation**: The peel must be cut to a size suitable for transportation. Lines for cutting are marked using an oil-based marker. The markings are photographed. The position of the cut lines is measured in three dimensions (Fig. 2).
5. **Cutting and lifting the preparation**: Using a knife, the preparation is cut along the cut line and the layer is peeled

off, little by little, by lifting it. The peeled layer should be photographed.

6. **Transporting the peel**: The peel supporting the sample of the face should be wrapped with a cushioning material for transportation.
7. **Washing the surface of the peeled layer**: Excess material is washed off using flowing water. A brush must not be used. The peel is then allowed to dry without heating.
8. **Fixing the surface of the peel**: An ethyl acetate-based resin, such as SANKOL, is then used to fix the stratum particles onto the surface of the peeled layer (Fig. 3) for long-term preservation, examination, and sampling.

Cross-References

- ▶ Artificial Ground
- ▶ Contamination
- ▶ Liquefaction
- ▶ Site Investigation

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Percolation

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Synonyms

Infiltration

Definition

Percolation can be defined as the flow of fluids through a porous media (filter). Infiltration rate may be defined as the meters per unit time of the entry of water into the soil surface regardless of the types or values of forces or gradients. Water entry into the soil is caused by matric and gravitational forces. Infiltration normally refers to the downward movement (Kirkham 2004).

Context

The rate of infiltration is influenced by the physical characteristics of the soil such as soil hydraulic conductivity ($K(\theta)$, LT^{-1}), initial water content (θ_i, $m^3\ m^{-3}$), residual water content (θ_r, $m^3\ m^{-3}$), saturated water content (θ_s, $m^3\ m^{-3}$), soil cover (i.e., plants), soil temperature, and rainfall intensity (Kirkham 2004; Essig et al. 2009). Numerical results have elucidated the role of gravity, capillary forces, and slope angle on infiltration over various periods.

The degree of infiltration very much depends on the soil type and thickness. Sandy soils allow water to easily infiltrate, whereas silt and clay soils have slow infiltration rates and more prone to move water as surface runoff. Similarly, moist soils produce more runoff than dry soils. In fact, high infiltration rates that occur in dry soils show slowing as the soil becomes wet.

Infiltration rates are known to change over time. Studies during water infiltration events (entry from above) have shown high infiltration rates at the beginning of the event, followed by a relatively rapid decline that transitions toward near-constant values.

Many models for water entering into the soil have been developed such as the Lewis, Horton, Green-Ampt equation, and Philip infiltration model. They all deal with one-dimensional downward infiltration of water into the soil. Richard's Equation and Green-Ampt model are theoretical methods that estimate this phenomenon (Green and Ampt 1911; Mein and Larson 1973).

Water infiltration can be described by Darcy's law. The flow rate ($m\ s^{-1}$) across a unit cross section (m^{-2}) of soil is:

$$q_x = -kh\left(\frac{dz}{dx} + \frac{d\left(\frac{p}{\gamma_w}\right)}{dx}\right) \quad (1)$$

$$q_z = -kh\left(1 + \frac{d\left(\frac{p}{\gamma_w}\right)}{dz}\right) \quad (2)$$

where q_x and q_z are the flow rates ($m.s^{-1}$) in the horizontal (x) and vertical (z) directions, respectively, $-Kh$ is the hydraulic conductivity ($m.s^{-1}$), p is the soil water pressure ($F.m^{-2}$, where F is force), and γ_w is the weight density of liquid water ($F.m^{-3}$).

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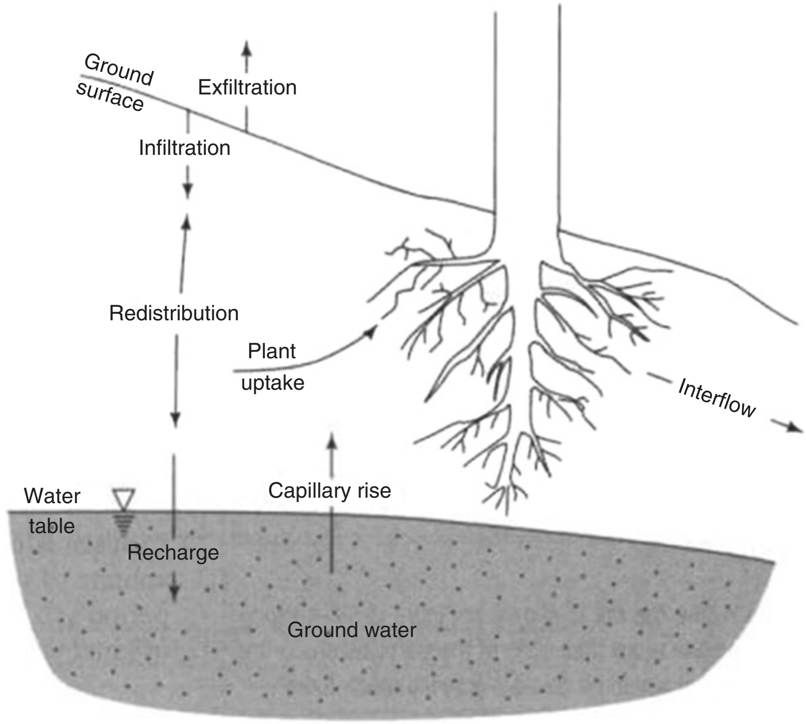
Percolation, Fig. 1 The cycle of infiltrated water in the unsaturated zone



Richards equation combines Darcy's Law for vertical unsaturated flow with the conservation of mass. It is widely used as a basis for numerical modeling of soil water flow by specifying appropriate boundary conditions, dividing the soil profile into very thin layers, and applying the equation to each later sequentially over small increments of time. In this Equation (Kirkham 2004):

$$\frac{\partial \theta}{\partial z} = -\frac{\partial kh(\theta)}{\partial z} + \frac{\partial}{\partial z}\left[kh(\theta).\frac{\partial \psi(\theta)}{\partial z}\right] \tag{3}$$

expressed verbally, the time rate of change in volumetric soil moisture for a given thin layer of soil depends on the vertical rate of change of the hydraulic conductivity (itself a function of θ) and the vertical rate of change of the product of (a) the hydraulic conductivity, and (b) the vertical rate of change of the pressure head ψ (the matric suction gradient), the pressure head also being a function of θ. In this expression, z is taken to increase *downwards*.

The term "redistribution" refers to the continued movement of water thorough a soil profile after irrigation or rainfall has stopped at the soil surface. Redistribution occurs after infiltration and is a complex process because the lower part of the profile ahead of the wet front will increase its water content and the upper part of the profile near the surface will decrease its water content, after infiltration ceases.

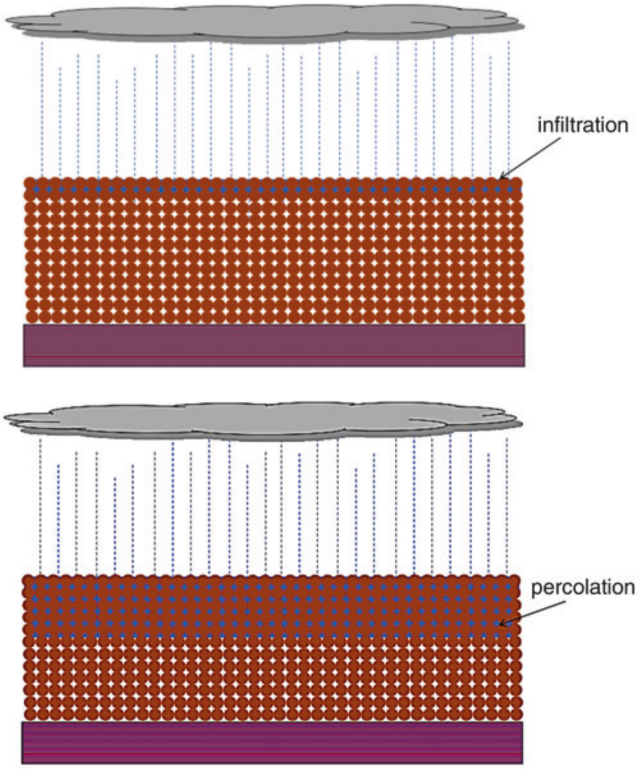
In other words, redistribution is the subsequent movement of infiltrated water in the unsaturated zone of a soil. This can involve exfiltration (evaporation from the upper layer of the soil), capillary rise (movement upward from the saturated zone to the unsaturated zone due to surface tension), recharge (movement of water from the unsaturated zone to the saturated zone), and inter flow (flow that moves down slope). The components of this cycle are shown in Fig. 1.

Thus, percolation is a general term for the downward flow of water in the unsaturated zone of the soil profile. The terms infiltration and percolation are often used interchangeably (see Fig. 2). Also, K(θ), θ_i, θ_r, θ_s, soil cover, soil temperature, soil texture, and rainfall rate are some of the factors that govern percolation rate. Percolation plays an effective role in the hydrologic cycle by partitioning rainfall into surface and subsurface streams (Essig et al. 2009).

Percolation is one phenomenon that can be readily measured and evaluated. Related terms include percolation theory that suggests the existence of a structural phase transition that can be established by measuring the components in structural transition. Another is percolation process that is the transition of liquids by filtering. The process involves the extraction solvent material from a powder by filtering.

Summary

Percolation is an important concept in engineering geology. Percolation is linked to soil and rock properties and is a significant parameter of consideration in the spatial variability and vertical fluctuation of water tables.



Percolation, Fig. 2 Illustration showing the relationship between the infiltration and percolation rate

Cross-References

- ▶ Aquifer
- ▶ Aquitard
- ▶ Cohesive Soils
- ▶ Darcy's Law
- ▶ Filtration
- ▶ Fluid Withdrawal
- ▶ Hydraulic Action
- ▶ Hydrocompaction
- ▶ Hydrogeology
- ▶ Hydrology
- ▶ Infiltration
- ▶ Pore Pressure
- ▶ Run-off
- ▶ Soil Mechanics
- ▶ Soil Properties

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Permafrost

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Definition

Permafrost, or perennially frozen ground, is defined as soil or rock having temperatures below 0 °C over at least two consecutive winters and the intervening summer. Much of the permafrost has been frozen since the Pleistocene. Permafrost occurs in the Arctic, Antarctic, and high alpine regions. About one-fifth of the total land area of the world is underlain by permafrost (Burdick et al. 1978).

The top layer of the ground in which the temperature fluctuates above or below 0 °C during the year is defined as the active layer (Andersland and Ladanyi 1994). Other terms such as seasonally frozen ground, seasonal frost, and annually thawed layer are synonyms for the active layer. The thickness of this layer varies spatially and temporally.

The upper boundary of permafrost is defined as the permafrost table. In the discontinuous permafrost zone, taliks form between the active layer and the permafrost table. Taliks, or unfrozen ground, are layers of ground that remain unfrozen throughout the year (Andersland and Ladanyi 1994). In the continuous permafrost zone, taliks often occur underneath shallow thermokarst lakes or rivers, where the water below a certain depth may not freeze in winter and, thus, the soil underneath will not freeze either. Other terms, such as thaw lake or cave-in lake, have also been used for a thermokarst lake. Open talik is an area of unfrozen ground that is open to the ground surface but otherwise enclosed in permafrost. Through talik is unfrozen ground that is exposed to the ground surface and to a larger mass of unfrozen ground

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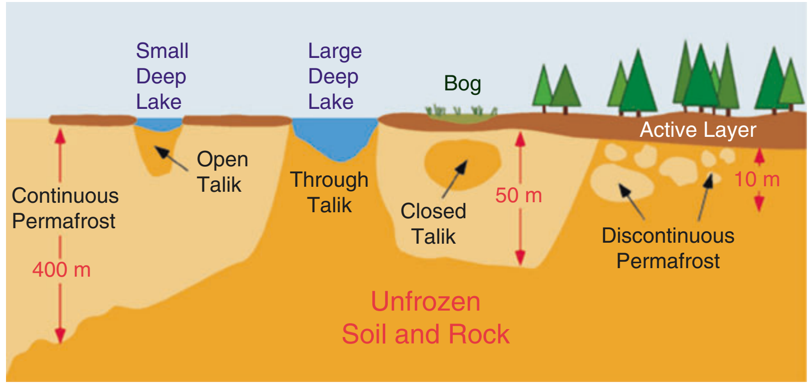


Permafrost, Fig. 1 Vertical cross section of the transition zone between continuous and discontinuous permafrost. The graphic also shows the various types of talik (Pidwirny 2008)

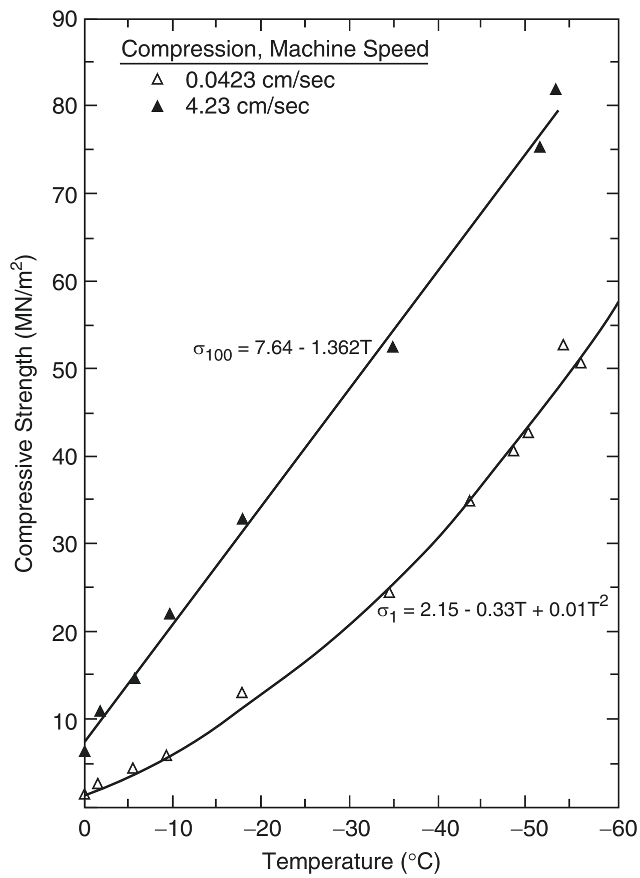
beneath. Unfrozen ground encased in permafrost is known as a closed talik (Fig. 1, Pidwirny 2008).

Introduction

Permafrost underlies approximately 20% of the world's land surface (Burdick et al. 1978). Engineering construction in these regions presents unique engineering challenges due to the alteration by human activities of the ground thermal regime. Investigations relating to frozen ground engineering have developed rapidly in the past several decades because of increasing civil constructions and mineral resource developments in cold regions. These developments have helped to stimulate research on many frozen soil problems. An impressive array of useful information has been presented in a variety of professional publications.

Research on the temperature effect on the strength of frozen soils has been conducted by several investigators (e.g., Sayles and Haines 1974; Haynes and Karalius 1977). Because of its direct influence on the strength of intergranular ice and on the unfrozen water content in a frozen soil, temperature plays a significant role in the mechanical behavior of frozen soils. Generally, a decrease in temperature results in an increase in strength of a frozen soil; however, it increases the brittleness of the frozen soil at the same time. Haynes and Karalius (1977) reported that at a given strain rate the compressive strength of a frozen silt increased more than an order of magnitude as the temperature decreased from −0.1 to −50 °C. The compressive strength of a frozen silt as a function of temperature is shown in Fig. 2. Efforts to express the theoretical strength variation of frozen soils with temperature have been less successful. Laboratory tests on frozen sand and frozen clay were conducted by several investigators. However, fewer attempts were made on frozen gravel due to technical difficulties.

Research on thermal properties and heat transfer in frozen soils has a long history. Berggren (1943) predicted the temperature distribution in frozen soil and indicated three obstacles to accurately predicting thermal condition in soils. First,



Permafrost, Fig. 2 Average strength versus temperature relationship for a frozen soil in uniaxial compression tests (Haynes and Karalius 1977)

the prediction is based on the observed or assumed initial distributions and subsequent surface conditions. Second, it is difficult to measure the thermal properties. Third, the migration of water during freezing is likely to invalidate the determination of initial soil properties. Brown and Johnson (1965) conducted one of the earliest field investigations on the active layer. Modeling efforts relating to the freezing and thawing process have been made by many investigators (e.g., Outcalt et al. 1975; Goodrich 1982). These efforts include, but are not limited to, (1) numerical modelling on simulating the snowmelt and soil thermal regime through the surface energy balance, (2) study of the effect of snow cover on ground temperature, and (3) two-dimensional numerical modelling on coupled heat and moisture transport in freezing soil. The last model was applied to investigate variations in permafrost thickness in response to changes in paleoclimate and to study the impacts of sea level and climate change on permafrost temperature and gas hydrates.

Research on thaw lake problems has been conducted by many investigators. More than half a century ago, Hopkins (1949) indicated that “thaw lakes,” resulting from surface collapse caused by thaw of ice-rich permafrost, are important and conspicuous features of Arctic and subarctic lowland landscapes. Rex (1961) conducted hydrodynamic analysis of circulation and orientation of lakes in northern Alaska. Sellmann et al. (1975) carried out a detailed investigation on the classification and geomorphic implications of thaw lakes on the Alaskan Arctic coastal plain. The technique relied on multispectral sequential Landsat 1 images and observations of the persistence of ice cover during the thaw season. Jefferies et al. (1996) reported a new method of determining lake depth and water availability on the North Slope of Alaska. This method combines spaceborne synthetic aperture radar (SAR) remote sensing technique with numerical modelling on ice growth to determine lake depth and water availability. Zhou and Huang (2004) used numerical modelling to study the impacts of thaw lakes on ground thermal regime. The numerical model included a multimedia system with transient heat transfer. The system includes a snow cover on top, a shallow lake in the middle, and unfrozen/frozen soils beneath the lake. The model is verified against field observations. The difference between the simulated and observed ice thickness in the lake is less than 3%.

Engineering Geology Considerations

Many distinct terrain features are associated with frozen ground. From an engineering geology point of view, the more important features include ice-rich permafrost, ice wedges, pingos, and thermokarst topography. These features may cause difficult and expensive construction problems. Engineering considerations require an understanding of the freezing/thawing process, the effects of thawing of the frozen ground, frost heave, and thaw settlement. Some aspects of the frozen ground can be utilized by engineers, for instance, using ground freezing techniques to enhance the stability of the underground excavations and to control the ground deformation due to disturbance to ground temperature.

Alternating freeze and thaw will occur in foundation soils during the cooling and warming cycles. Thawing of frozen soils will result in ice disappearance, void ratio change, and hence volume changes (settlement). Mitigation of these frost-and-thaw-related problems is one of the typical concerns of practicing engineers in cold regions.

The most important characteristic of frozen soil is that, under natural conditions, its matrix, composed mostly of ice and water, changes continuously with varying temperatures and applied stress. The mechanical behavior of frozen soil is closely related to the ground temperature. Ground temperatures are determined mainly by air (or ground surface) temperatures, seasonal snow cover, heat flow from the interior of the earth, heat from solar radiation, and soil thermal properties.

The response of the soil thermal regime to changes in the environment requires an understanding of their thermal properties, such as thermal conductivity, heat capacity, thermal diffusivity, latent heat, and thermal expansion. These thermal parameters vary with air temperature, soil type, water and ice contents, degree of saturation, and soil density.

Selected Case Studies

Many engineering problems in cold regions may be attributed to the changes of ground temperature, that is, the thermal disturbance of permafrost by human activities or nature processes. Engineering projects in cold regions, such as construction of buildings, roads, and pipelines, must be based on a good understanding of the ground thermal regime and its interaction with climate.

Goering (2003) presented results from a passively cooled railway embankments study. The basic idea is to control the ground thermal regime in permafrost areas. In cold regions, even moderate alteration of the thermal regime at the ground surface can induce permafrost thawing with consequent settlement and damage to roadway or railway embankments. Goering (2003) examined the heat transfer and thermal characteristics of railway embankments constructed of unconventional, highly porous materials. He revealed that convection enhances the upward transport of heat out of the embankment during winter, thus cooling the lower portions of the embankment and underlying foundation soil.

Frost heave is a common issue for pipeline design in cold regions. Kim et al. (2008) studied pipe displacement due to frost heave under the assumption that the amount of soil heave at the free-field area, an area free of influence of pipe restraint (Fig. 3), is negligible. A quasi two-dimensional explicit finite difference model was developed to predict the pipe displacement, and the segregation potential (SP) concept was applied to the model. The developed frost heave model was verified by a full-scale buried chilled gas pipeline experiment in Fairbanks, Alaska. The simulated thermal analysis results agreed with the observed results.

The Qinghai–Tibet Railway is now considered to be an engineering miracle. Constructing the Qinghai–Tibet Railroad

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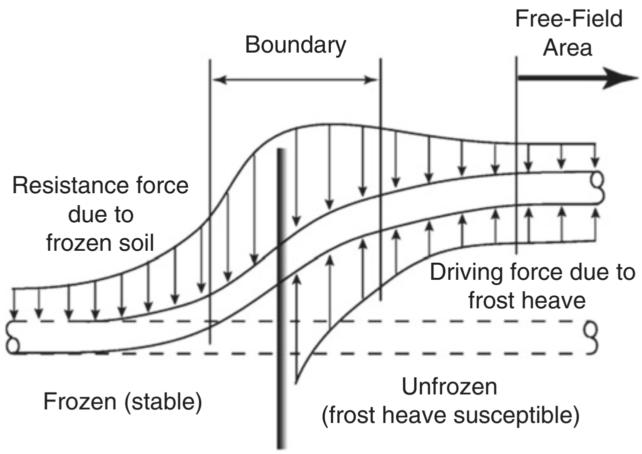

Permafrost, Fig. 3 Differential heaves along a pipeline (Kim et al. 2008)

was a serious challenge to permafrost roadbed engineering geologists. Cheng (2005) reviewed and summarized the engineering–geological studies carried out for various lifelines, such as roads, pipelines, and communication fiber-optic cable in this largest high-altitude permafrost area of the world and provided guidelines for constructing the Qinghai–Tibet Railroad. Cheng (2005) summarized advances made in engineering–geological site investigation, in studies of roadbed behavior in various permafrost areas of the Qinghai–Tibet Plateau and in creating provisions and regulations to guide engineering activities. Two parameters, that is, ground temperature and ice content, were studied intensively due to their significant impacts to the engineering–geological properties of permafrost. Based on the data collected from these studies, recommendations are made to enhance the stability of roadbeds. Under a climate warming scenario, a "roadbed cooling" approach is suggested for road constructions in "warm" permafrost.

Summary and Conclusions

Approximately 20% of the total land area of the world is underlain by permafrost. In the past several decades, under the pressure of population growth and expansion of urbanization, civil construction and mineral/enginery development have extended rapidly into the cold regions. These developments pose serious challenges to engineering geologists or geological engineers due to the disturbance to the ground thermal regime and alteration of ice contents. Advances have been made in many perspectives of permafrost engineering studies, such as permafrost site investigation, engineering behavior of permafrost, and mitigation techniques for controlling the thermal regime of frozen ground.

Cross-References

▶ Characterization of Soils
▶ Climate Change
▶ Field Testing
▶ Foundations
▶ Glacier Environments
▶ Karst
▶ Mountain Environments
▶ Pipes/Pipelines
▶ Thermistor
▶ Thermocouple

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Petrographic Analysis

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Definition

Petrography is the description and systematic classification of rocks, mainly by the microscopic examination of thin sections.

Introduction

Petrographic analysis identifies the origin, whether igneous, sedimentary, or metamorphic, and the mineral content for the classification of a rock.

Analysis usually comprises the description of the macroscopic aspects of the rock, such as fabric, color, grain size, and other relevant characteristics that may be visually observed in hand specimen or in outcrops, and chiefly the identification and description of microscopic characteristics of the studied material in thin sections such as mineral composition, texture, grain size, and evidence of alteration and/or deformation.

The results of petrographic analysis are an important tool for many branches of the geosciences by allowing the determination of the conditions of the formation of the rock, but, in engineering geology, the aim is to explain the mechanical behavior and anticipating the durability of the rock and its performance as a construction material, a foundation substrate, tunneling medium or ease of excavation.

Depending on the purpose of the analysis, certain microscopic features are emphasized. For engineering geology, some important issues are the presence of potentially deleterious minerals for the planned use, the type and degree of alteration of minerals, the intensity of deformation or microcracking, as well as other features that may influence the engineering properties of the rock. This information also supports the interpretation of laboratory test determinations of physical and/or mechanical properties, often clarifying the differences between superficially similar rocks. They are also essential for the diagnosis of the deterioration of rocks or rock products such as concrete.

Guidelines for petrographic examination and mineralogical identification appear in an extensive literature including standardization by the major normalizing institutions, such as those from CEN and ASTM: EN 932 (BSI 1997), EN 12407 (BSI 2007), ASTM C295 (ASTM 2012), and ASTM C1721 (ASTM 2015).

It should be noted that, in most of standards and recommendations, it is made clear that petrographic examinations must be carried by suitably qualified geologists (petrographers).

Macroscopic Description

Macroscopic description is the first step for the characterization of the studied rock. It aims to identify the most important features observed by the naked eye, such as color and fabric.

Color

In spite of the subjectivity inherent in color identification, it is an essential parameter to characterize the rock; even though it may be diverse for the same rock type.

Color is intimately associated with the mineralogical composition and, essentially, to impurities present in the minerals or in the rock.

In igneous rocks, such as granites, it is highly dependent on the type and variety of feldspars, which can exhibit a large color range: pink to dark red, light to dark green, gray, etc. The black color of rocks like diorite and basalt is due to the ferromagnesian rock-forming minerals. When the rock is weathered, the presence of iron hydroxides filling microcracks in minerals is responsible for shades of yellow or orange. Some uncommon colors are associated with specific minerals such as blue quartz in rhyolites and sodalite (an intense blue-colored feldspathoid) in syenites.

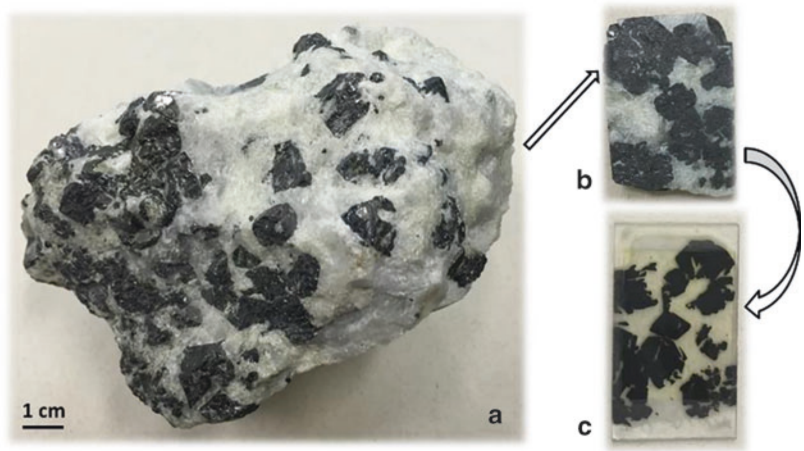
Sedimentary rocks, such as sandstone, generally show colors ranging from white to beige or pink to reddish; in this case, it is mainly due to a very thin film of iron oxides or hydroxides covering the grains. Shale or argillite may have dark gray to black and red colors, depending on the abundance of carbonaceous residues (buried organic matter) or iron hydroxides, respectively.

Metamorphic rocks, such as marble, commonly are white or gray, with greenish shades due to the presence of talc, amphibole (tremolite), pyroxenes (diopside), or olivine (forsterite). Quartzite is commonly white but can display a wide variety of colors due to minor mineral components, showing variations to red (by the presence of iron hydroxides) and even blue (dumortierite).

Structure

Structure is a broad term referring to the orientation and spatial position of rocky masses or rock unit in a given area, as well as the features resulting from geological metamorphic processes as faulting and folding; igneous intrusions, such as columnar structure; and even to the sedimentary deposition, such as bedding. It is generally best seen in the outcrop rather than in hand specimen or thin section.

Typically, igneous rocks are massive, and the minerals do not show preferred orientation, both in outcrop and hand specimen. This compact rock formation normally results in homogeneous physical and mechanical properties (isotropy). However, igneous rocks can also show fluid or fluxionary structure, in which the minerals may show orientation resulting from directional movement of ascending magma.



Petrographic Analysis, Fig. 1 Magnetite (*black crystals*) carbonatite: in hand specimen (**a**), after sawing in a small block (**b**) and the thin section (**c**)

Metamorphic processes that are controlled by physical conditions such as pressure and temperature may result in metamorphic rocks having either massive structure, like marble or quartzite, or, more characteristically, directional structures, like foliation, lineation, and layering, which may be observed in gneiss and schist, and which give anisotropy to these rocks, that is, different mechanical properties according to the different directions.

Foliation may be curved (folded) or distorted. Some examples of foliations (Bucher and Grapes 2011) are schistosity, cleavage, and gneissose structure (best observed at hand-specimen scale) produced by deformation and recrystallization and defined by irregular or poorly defined layering or by augen and/or lenticular aggregates of mineral grains (augen structure, flaser structure).

Layering or bedding are typical structures of sedimentary rocks and represent the arrangement of these rocks in distinct layers with thickness ranging from centimeters to few meters. The term lamination is used to refer to the layers with thickness less than 1 cm.

Microscopic Description

Microscopic description is understood by many as the main aspect of petrographic analysis. It highlights the features that control and affect practically all properties of the rocks: the mineralogical composition, grain size, textural arrangement, alterations, weathering, and microcracking.

A great difficulty in petrographic description is the approach needed to systematize two key aspects: weathering and degree of microcracking.

Mineralogical Composition

In order to determine the mineralogical composition of the rock, some important previous information is required regarding mineral identification and quantification.

Mineral Identification by Petrographic Microscope

In the microscopic study of rocks, the first step is the identification of the constituent minerals because the mineralogy reflects the chemical composition, the formation conditions, and any modifications after solidification or consolidation, which have decisive influences on the rock properties.

The minerals are usually determined by the observation of a thin section under a polarizing microscope (petrographic microscope), built specifically to permit the verification of the distinctive optical characteristics that make it possible to identify a large number of minerals.

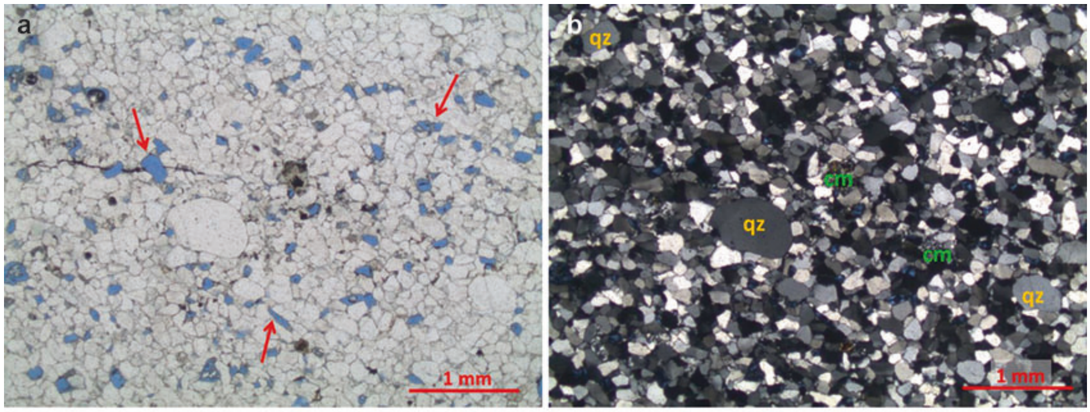
The thin section consists of a fragment or sawed piece of a selected area of the rock or mineral mounted on a glass slide (around 1.5 mm thickness) by an appropriate resin (usually epoxy), mechanically ground to a thickness of approximately 30 μm and finally covered by a very thin glass slide (circa 0.15 mm) usually fixed with balsam (Fig. 1).

Some important characteristics of the studied material for engineering geology purposes are the presence and distribution of microcracks in igneous and metamorphic rock, open pores in sedimentary rocks, or amygdales in igneous rocks. These may be enhanced by filling them with epoxy resin with special fluorescent or staining compounds (Fig. 2) introduced in the rock before the making of thin section or in the uncovered thin section (Hibbard 1995).

Mineral color and/or pleochroism, relief, cleavages, birefringence, extinction angles, elongation, and interference figures, the main properties observed through the microscope, are comprehensively described in the literature (MacKenzie and Guilford 1980; Klein and Dutrow 2008; Deer et al. 2013).

As a guideline, Moorhouse (1959) and Demange (2012) suggest the following observations:

- Form and crystallographic properties (usually viewed by orthoscopy or under parallel polarized light – PPL):
  – Crystal form



Petrographic Analysis, Fig. 2 Epoxy resin with blue colorant compound filling spaces resulting from detached grains (mainly quartz – *qz*) or matrix (clay minerals – *cm*) in sandstone (quartz arenite)

  – Shape of grain: fibrous, acicular, radiating, reticulate, tabular, platy
  – Relative index and relief
  – Transparent or translucent minerals: color and pleochroism
  – (*)Opaque minerals (minerals that do not transmit light): color under reflected light
- Optical properties (usually viewed by conoscopy or under cross polarized light – CPL)
  – Presence of cleavage, parting, or fracture: number of cleavages and angular relationships to one another, perfection of cleavage, characteristics of parting and fracture
  – Inclusions, intergrowths, association with another minerals
  – Twinning
  – Isotropic or anisotropic character. If anisotropic, interference colors and determination of birefringence
  – Extinction angle
  – Determination of length slow or length fast behavior
  – Determination of interference figures: uniaxial or biaxial, whether positive or negative

Note: (*) the identification of opaque minerals (usually Fe, Cu, Ni oxides, or sulfides) is done through the petrographic microscope by using reflected light devices instead of transmitted light.

Some Auxiliary Techniques

- **Staining tests**: performed on hand specimen or uncovered thin section in order to facilitate the identification and quantification of certain minerals. Hutchison (1974) described several staining methods, among them the use of sodium cobaltinitrite solution for distinguishing K-feldspar that stains yellow to plagioclase, red or white, and quartz (no stain) and the use of Alizarin red S® to distinguish calcite (stains red or pink) from dolomite (no stain).
- **X-ray diffraction**: useful to the identification of minerals that are very difficult to be determined by optical means as opaque and very fine-grained minerals, especially clay minerals. It is also used to differentiate minerals having very similar optical properties, such as pyrophyllite and talc.
- **Scanning electron microscopy** (SEM): increasingly used for the identification and characterization of minor mineral phases that are considered important to the purpose of analysis. It permits not only the imaging but the chemical composition determination of the selected mineral by X-ray microanalysis, using EDS (Energy Dispersive X-ray Spectroscopy) and WDS (Wavelength-Dispersive X-ray Spectroscopy) detectors (Klein and Dutrow 2008).
- **Image analysis**: the quantification of the mineral phases present in the rock as well as the shape and grain-size determination, by thin section examination, may be done manually or automated. The improvement of image analysis equipment coupled to petrographic microscopes has enabled the development of many techniques for quantification of selected features (Allard and Sotin 1988).

Mineral Groups

A mineral is generally defined as a solid, inorganic, and homogeneous natural substance that shows defined chemical composition and characteristic atomic structure. It is formed, in nature, mainly by the crystallization from magmatic fluids or thermal solutions, which takes place when atoms, ions, or ionic groups, in defined proportions, are attracted by electrostatic forces and neatly distributed in space.

The classification of minerals is based on the anion or anionic group dominant in its chemical formula, and may be summarized in two groups: silicates and nonsilicates.

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Petrographic Analysis, Table 1 Some examples of rock-forming silicate minerals (Klein and Dutrow 2008)

Class	Groups	Mineral	Chemical formula
Nesosilicates	Olivine	Forsterite	Mg_2SiO_4
	Garnet	Almandine	$Fe_3Al_2Si_3O_{12}$
	Zircon	Zircon	$ZrSiO_4$
	Al_2SiO_5	Sillimanite, kyanite, andalusite	Al_2SiO_5
Sorosilicates	Epidote	Clinozoisite	$Ca_2Al_3O(SiO_4)(Si_2O_7)(OH)$
Cyclosilicates	–	Cordierite	$(Mg,Fe)_2Al_4(Si_5O_{18})\bullet nH_2O$
		Tourmaline	$(Na,Ca)(Li,Mg,Al)_3(Al,Fe,Mn)_6(BO_3)_3(Si_6O_{18})(OH)_4$
Inosilicates	Pyroxene	Augite	$(Ca,Na)(Mg,Fe,Al)(Si,Al)_2O_6$
		Aegirine	$NaFe^{3+}Si_2O_6$
		Diopside	$MgCaSi_2O_6$
		Hypersthene	$(Mg,Fe)_2Si_2O_6$
	Amphibole	Hornblende	$(Na,Ca)_2(Mg,Fe)_5\ Si_7AlO_{22}(OH)_2$
		Riebeckite	$Na_2Fe^{2+}{}_3Fe^{3+}{}_2Si_8O_{22}(OH)_2$
Tectosilicates	Feldspar	K-Feldspar (microcline, orthoclase)	$KAlSi_3O_8$
		Plagioclase (albite-anorthite series)	$(Na,Ca)(Al,Si)AlSi_2O_8$
	SiO_2	Quartz	SiO_2
		Opal	$SiO_2\bullet nH_2O$
	Feldspathoid	Leucite	$KAlSi_2O_6$
		Nepheline	$KNa_3(AlSiO_4)_4$
		Sodalite	$Na_4Al_3Si_3O_{12}Cl$
	Zeolite	Natrolite	$Na_2Al_2Si_3O_{10}.2H_2O$
Phyllosilicates	Mica	Muscovite	$K_2Al_4Al_2Si_6O_{20}(OH)_4$
		Biotite	$K_2(Mg,Fe)_6Al_2Si_6O_{20}(OH)_4$
		Phlogopite	$K_2Mg_6Al_2Si_6O_{20}(OH)_4$

Petrographic Analysis, Table 2 Some examples of nonsilicate minerals (Klein and Dutrow 2008)

Class	Mineral	Chemical formula
Oxides	Hematite	Fe_2O_3
	Ilmenite	$FeTiO_3$
	Rutile	TiO_2
	Magnetite	Fe_3O_4
Sulfides	Pyrite	FeS_2
	Galena	PbS
Native elements	Gold	Au
	Graphite	C
Carbonates	Calcite	$CaCO_3$
	Dolomite	$CaMg(CO_3)_2$
Halides	Halite	NaCl
	Fluorite	CaF_2

Silicates are the main rock-forming minerals, and in accordance to the structural configuration of SiO_4 tetrahedra, they are subdivided in six subclasses (Table 1, after Klein and Dutrow 2008).

Nonsilicate minerals (Table 2, after Klein and Dutrow 2008) mainly consist of a large group of oxides, sulfides, hydroxides, carbonates, sulfates, halides, and native elements, usually occurring in minor or trace quantities but which may be concentrated in special geologic conditions forming ore deposits.

Minerals which are subjected to alteration produce secondary minerals, usually salts, hydrous aluminosilicates (as clay minerals), and iron and aluminum hydroxides (Table 3, after Klein and Dutrow 2008).

The term clay refers to a natural material composed primarily of very fine-grained minerals, usually plastic when the water content is suitable, which harden when dried or burned (Neuendorf et al. 2011). In terms of size, clay usually refers to particles <0.004 mm. By means of X-ray diffraction techniques, it has been shown that clays are also a group of crystalline substances known as clay minerals (chlorite, illite, kaolinite, and montmorillonite groups), which are essentially hydrated aluminum silicates, arranged in a layer structure, or are mixed-layered clay minerals consisting of these components.

The montmorillonite group, also called smectite group, is of special significance for engineering geology as they are characteristically expansive in the presence of water but contract on drying. This often causes the physical breakdown of the rock, and consequently damage to built constructions and structures. But, they can also be used in engineered barriers preventing movement of pollutants into groundwater.

For engineering purposes, some minerals are considered deleterious as they may be harmful after their application. Examples are sulfide minerals, such as pyrite which, when present in stone aggregate used for concrete or mortar, may

Petrographic Analysis, Table 3 Some examples of minerals that are usually secondary (Klein and Dutrow 2008)

Groups	Mineral	Chemical Formula
Serpentine	Antigorite, chrysotile, lizardite	$Mg_6Si_4O_{10}(OH)_8$
Chlorite	Chlorite	$(Mg,Fe,Al)_6(Al,Si)_4O_{10}(OH)_8$
–	Talc	$Mg_3Si_4O_{10}(OH)_2$
Clay Minerals	Kaolinite	$Al_4Si_4O_{10}(OH)_8$
	Montmorillonite or Smectite	$(Al,Mg)_2Si_4O_{10}(OH)_2{\bullet}4H_2O$
	Illite	$(K,H_3O)Al_2(Si_3Al)O_{10}(H_2O,OH)_2$
	Vermiculite	$Mg_{0.7}(Mg,Fe,Al)_6(Si,Al)_8O_{20}(OH)_4{\bullet}8H_2O$
Sulfates	Barite	$BaSO_4$
	Anhydrite	$CaSO_4$
	Gypsum	$CaSO_4{\bullet}2H_2O$
Hydroxides	Goethite	$\alpha FeO(OH)$
	Limonite	$FeO{\bullet}OH{\bullet}nH_2O$
	Gibbsite	$Al(OH)_3$

react to solutions coming from different sources resulting in sulfate compounds that are expansive and cause fissuring and disaggregation of the concrete or mortar.

Mineral Quantification

In petrographic analysis, some features have to be quantified in order to establish the appropriate rock classification: primarily the mineral content but also grain size and shape of grains depending on the nature of the rock.

The quantification of the mineral phases present in the rock, by thin section examination, may be done in several ways. Visual estimation is most frequently used as it is faster and easier for general purposes and gives the most important information for rock classification.

However, if a more detailed quantification is necessary, the point-counting technique (Hutchison 1974; Hibbard 1995), manual or automatic, is used. Automated image analysis is gradually being more used.

Rock-forming minerals usually required for the petrographic classification of igneous and metamorphic rocks are regarded as essential. Those that occur in smaller quantity, the presence of which is not decisive for classification, are called accessories (in general constituting less than 5% of the total).

Texture

Texture is the microscopic spatial arrangement of minerals, intimately related to the mineralogy and dominant formation physical conditions; specific textures are often exclusive to specific rock types.

Crystal shape and orientation are the main features for texture classification. Relative to shape, there are some special terms commonly used: (a) euhedral (or idiomorphic and automorphic), when a grain is bounded by well-formed crystal faces; (b) subhedral (or hypidiomorphic and hypautomorphic), when a grain is partly bounded by crystal faces; and (c) anhedral (or xenomorphic and allotriomorphic), when crystal faces are not developed (Neuendorf et al. 2011).

The most common textures of rocks are summarized in Table 4 (after Mackenzie et al. (1982), Williams et al. (1982), Yardley et al. (1990), Neuendorf et al. (2011)).

Petrographic Analysis, Table 4 Some common textures of rocks

Texture	Definition
Phaneritic	Minerals are large enough to be distinguished with the unaided eye, generally relating to plutonic rocks
Aphanitic	Minerals with very small dimensions, making them invisible to the naked eye, generally relating to volcanic rocks
Granular	Mineral grains showing nearly equal size. May be applied to sedimentary rocks, e.g., sandstones, or to igneous rocks, e.g., granites, pyroclastic rocks
Microcrystalline	Applied to rocks where individual crystals are visible only under the microscope
Cryptocrystalline	Crystals are too small to be recognized and separately distinguished even under the ordinary microscope (although crystallinity may be revealed by using the electron microscope)
Porphyritic	Larger crystals (called phenocrysts) dispersed in a groundmass (finer matrix), which can be crystalline or glassy, relating to volcanic and plutonic rocks
Granoblastic	Recrystallized minerals are essentially equidimensional, normally with straight boundaries. Relating to non-foliated, massive metamorphic rocks, e.g., quartzite and marble
Porphyroblastic	Larger crystals (called porphyroblasts) arranged in a granoblastic or lepidoblastic finer-grained matrix, relating to metamorphic rocks
Lepidoblastic	Parallel or subparallel alignment of sheet silicate minerals (micas, chlorite) in some metamorphic rocks, e.g., schists, phyllites
Nematoblastic	Preferred orientation of prismatic minerals (pyroxenes or amphiboles) in a metamorphic rocks, e.g., amphibolite

Grain Size

Grain size refers to the size of the grains or crystals that constitute the rock, determined by measuring the largest axis of the grain/crystal in a calibrated graduate scale located in the microscope ocular or by image analysis. It is one of the main criteria for classification of sedimentary rocks (e.g. mudstone, siltstone, sandstone, conglomerate).

Igneous and metamorphic rocks, in petrographic description, are referred to as very fine to very coarse in a graduation (Table 5) based on the size of individual crystals.

Petrographic Analysis, Table 5 Grain size designation usually adopted for igneous and metamorphic rocks

Grain size	Size (mm)
Very coarse grained (pegmatitic)	>30
Coarse grained	5–30
Medium grained	1–5
Fine grained	<1
Very fine grained	<0.1

Alteration

The type and intensity of alteration of rock-forming minerals is important petrographic information for engineering geology because it can affect rock strength.

Alteration may be understood as any chemical or physical modification of the original mineral components of the rock. There are a number of alteration processes that can affect the rock but for petrography they are usually referred to as hydrothermal, when involving the action of magmatic solutions, and weathering, when relating to the action of supergene fluids or solutions.

Hydrothermal alteration results from the reaction between pre-existing minerals and hot solutions rising from a cooling magma producing new minerals and sometimes ore deposits but generally does not result in notable modifications of the engineering properties of the rock, although there are some exceptions, for instance, kaolinization of granites.

Weathering takes place when the rock is exposed to atmospheric conditions, where the action of several agents such as rainfall, freezing and thawing, roots and microorganisms, and others results in the formation of secondary minerals that may lead to chemical disintegration (decomposition) or mechanical disaggregation of the rock, which significantly affect the engineering properties of the rock and rock mass.

Weathering is macroscopically indicated by changes in color and surface characteristics of the rock. Porosity, density, and mechanical strength are some engineering parameters influenced by this process.

Petrographically, weathering is indicated by features such as the presence of iron hydroxides in mineral microfissures or small lamellae of sericite or clay minerals in certain minerals, such as plagioclase (Fig. 3). There is no formal method to quantify the degree of weathering by microscopic examination. As a guideline, Frascá (2013) proposed four grades (Table 6) to rank weathering and microcracking in igneous and quartz-feldspar-rich metamorphic rocks.

Microstructures

Microstructures (microscopic scale) include microfractures, microcracks, microfaults, microfolds, etc. Their observation (see Table 6) is very important to the understanding of the mechanical behavior of rocks and, especially, to the evaluation of mechanical test results.

Microcracking in igneous and metamorphic rocks (Fig. 4) as well as pore size and configuration in sedimentary rocks play a significant role in water absorption and capillary uptake which may affect the stone durability or conservation when used, for instance, as construction materials.

Petrographic Classification

Petrographic Classification of any rock takes into account the mineralogical composition usually in combination with other aspects like grain size or structure. Nevertheless, there is no consensual classification for all rock types, each subject to different criteria according to different authors or institutions.

A statement of Bucher and Grapes (2011) for metamorphic rocks may be applied to all other rock types: “There is not one sole classification principle used for the description of metamorphic rocks, which consequently means that all metamorphic rocks may have a series of perfectly correct and acceptable names”.

Details of petrographic classifications are given in the igneous, sedimentary, and metamorphic chapters, but some of these are outlined below.

Igneous Rocks

Igneous rock classification is based on two main aspects: the modal mineralogy and grain size, also a criterion to separate volcanic from plutonic rocks. Exceptions are made for glassy or very fine-grained rocks (Shelley 1992), which may be classified on their chemical composition.

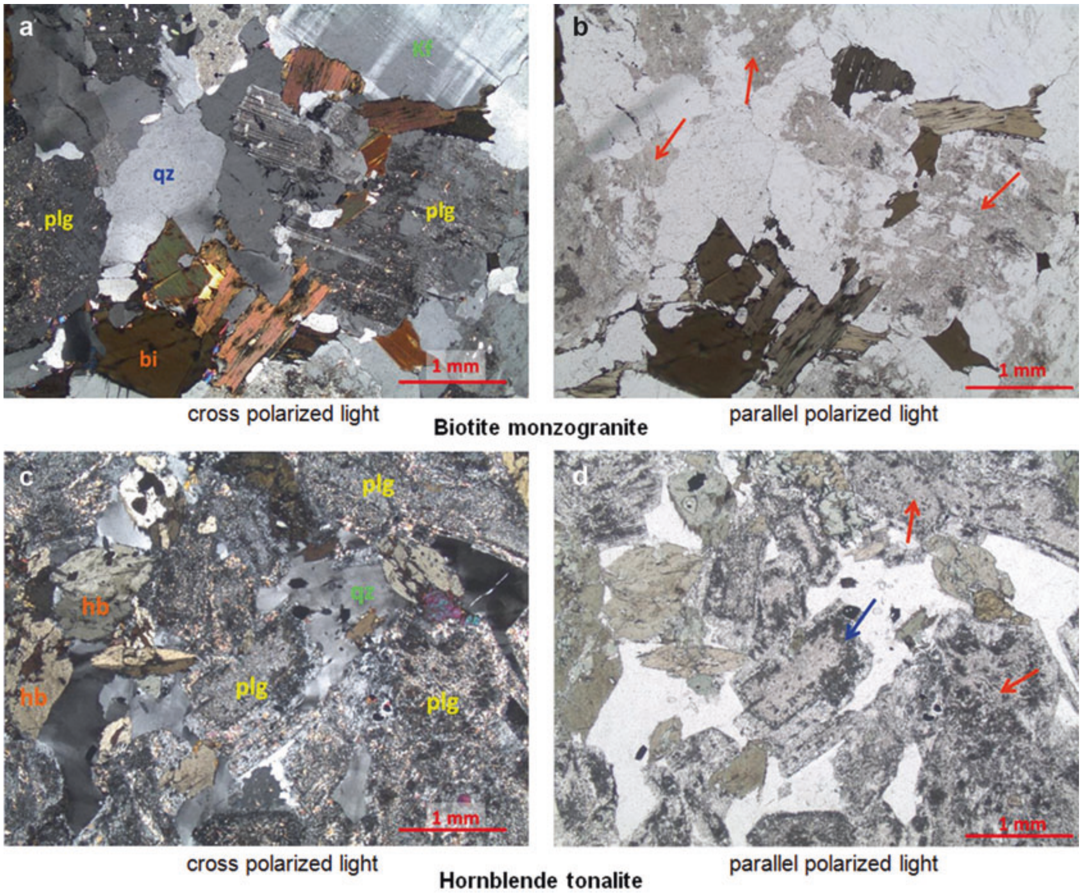
The IUGS classification for igneous rocks (Le Bas and Streckeisen 1991, Le Maitre 2003) is the most widely adopted. It is based on the relative proportions of the essential minerals, plotted in triangular diagrams constructed for each category of rocks, for example, plutonic, volcanic, and ultramafic, giving the root names such as granite, syenite, basalt, rhyolite, gabbro, dunite, charnockite, etc.

Sedimentary Rocks

The main systematic element for the classification of clastic sediments and clastic sedimentary rocks is particle size. But the petrographic classification of these rocks is much more complex, because it has to take account also the detrital components, normally rock fragments, quartz, feldspar, micas, clay minerals, heavy minerals (tourmaline, zircon, rutile, etc.); grain morphology, as shape, sphericity, and roundness (very angular, angular, subangular, subrounded, rounded, well rounded); cementation (carbonate, silica); sorting; and other aspects (see Carozzi 1993) that will reveal the depositional environment. The sandstone classifications proposed by Pettijohn (1975) and Pettijohn et al. (1987) are the most used.

Pyroclastic rock products are also classified according to grain size (Shelley 1992).

Limestone, the most important biochemical rock, is the subject of several current classification systems (Folk 1962; Dunham 1962 and others).



Petrographic Analysis, Fig. 3 Examples of alteration patterns of plagioclase in igneous plutonic rocks: (**a**), (**b**) hypidiomorphic to xenomorphic plagioclase (oligoclase) showing low weathering characterized by small lamellae of sericite growing mainly in their nuclei (*red arrow*); (**c**), (**d**) hypidiomorphic to idiomorphic plagioclase (oligoclase) showing moderate to strong hydrothermal alteration to sericite (*red arrow*) and epidote (*blue arrow*). *bi* biotite, *hb* hornblende, *Kf* K-feldspar, *plg* plagioclase, *qz* quartz

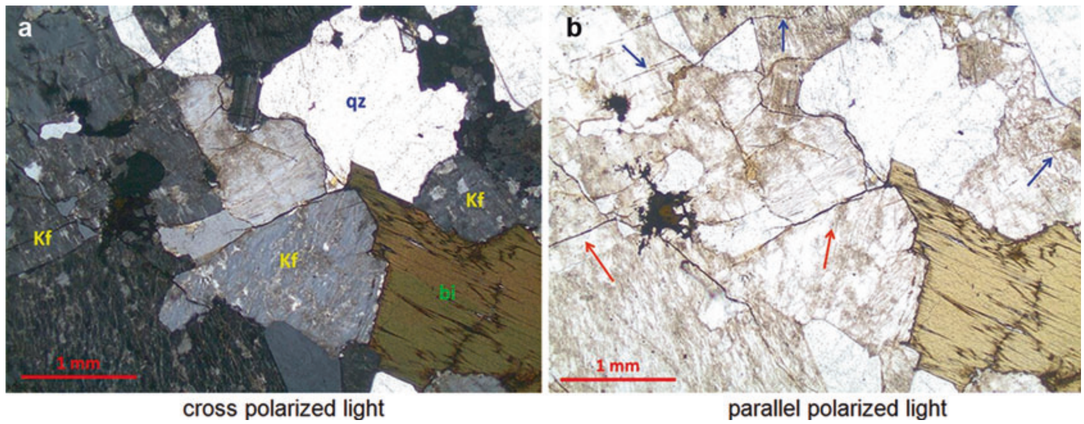
Petrographic Analysis, Table 6 Weathering and microcracking grades for igneous and metamorphic rocks identified in the microscopic examination

Grade	Weathering	Microcracking
Incipient	Mineral crystals are clear; iron hydroxides are not present, even in microfissures	Not perceptible
Low	Very slight turbidity in plagioclase crystals. Slight modifications in mafic minerals and some iron hydroxides are present	Microcracks are perceptible, mainly intragranular and sealed
Moderate	Plagioclase crystals show, especially in the nuclei, turbidity due to the presence of clay minerals, often with associated carbonate, sericite, and iron hydroxides. Biotite is partially altered with chlorite and some iron hydroxides. Other minerals may also be partly altered, as, for example, sillimanite, in phyllosilicates	Microcracks are fully perceptible, predominantly intra and intergranular, filled, and usually not very wide
Strong	Feldspars, especially plagioclase, are partly altered into clay and minor sericite, carbonate, and iron hydroxides. Other minerals, except quartz, show variable alteration degrees but generally partly to completely altered, especially into clay minerals and iron hydroxides	Microcracks fully perceptible; trans, inter and intragranular, unfilled and wide

Metamorphic Rocks

According to Schmid et al. (2007) and Bucher and Grapes (2011), the modal mineral composition and mesoscopic structure of the rock are the major features used for classification of metamorphic rocks. These are followed by the nature of the rock prior to metamorphism (protolith), the genetic conditions of metamorphism (increased pressure and/or temperature, with or without deformation), and the chemical composition of the rock.

In general, names of metamorphic rocks consist of a group name, or a root name, that may refer to the dominant structural



Petrographic Analysis, Fig. 4 Intergranular (*red arrow*) and intragranular (*blue arrow*) microcracking, filled with iron hydroxides in a biotite microcline granite (*Kf* microperthitic K-feldspar, *qz* quartz, *bi* biotite)

arrangement, such as phyllite, schist, and gneiss, or may be special names, such as marble, amphibolite, quartzite, serpentinite (which reflect mineralogy), or migmatite (which reflects the process of formation).

Petrography Analysis Applied to the Engineering Geology

The main use of petrographic analysis is to provide information on rocks that will have implications for the intended uses of rocks and for activities such as drilling and engineering.

It is very important to identify and explain the engineering properties of rocks or to predict these properties by the development of petrographic-based models such as those for the engineering petrography of a weathered granite (Irfan and Dearman 1978) or the uniaxial compressive strength of sandstones (Zorlu et al. 2008).

Petrographic analysis is also currently used to select aggregates used in concrete especially if these will be exposed to continuous moisture such as in water reservoirs, riprap, or hydroelectric plants (ASTM 2014) because of the possibility of alkali-silicate reactions between the alkalis of Portland cement and certain mineral components in some types of aggregates.

Summary

Petrography is the description and classification of rock. It is a fundamental tool for the geosciences and any other discipline application that deals with rocks.

Petrographic analysis includes the reporting of color, grain size, structure, and other macroscopic features observed either in hand specimen or in outcrops and, most importantly, the microscopic examination of thin sections of rock using petrographic microscopes, for the identification and quantification of mineral components, grain size, alteration, microcracking, etc. The mineral composition and some other features depending on the nature of the rock establish the petrographic classification.

The resulting information guides appropriate selection of rocks for engineering applications or as construction materials.

Cross-References

► Alkali-Silica Reaction
► Classification of Rocks
► Classification of Soils
► Clay
► Hydrothermal Alteration
► Igneous Rocks
► Metamorphic Rocks
► Sedimentary Rocks

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Photogrammetry

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Definition

Photogrammetry is a technique to obtain 3-dimensional measurements utilizing photographs as the fundamental medium for metrology (measurement).

The term, photogrammetry, was first used by the Prussian architect Albrecht Meydenbauer in 1867 during his work on some early topographic and elevation drawings (Albertz and Wiedemann 1996). The underlying principle used in photogrammetry is aerial triangulation. Photogrammetry relies on taking photographs from at least two different locations, which allows one to develop the "lines of sight" from each camera to points on the object. Further, the lines of sight are mathematically intersected to produce 3-dimensional coordinates of the points of interest.

From an early elementary use in topographic maps and elevation drawings, the applications of photogrammetry have now extended to diverse fields such as geology, engineering, architecture, industry, forensic, forensics, bathymetry, and medicine for the production of precise 3D data.

Photogrammetry is broadly classified into two branches based on data utilization:

Metric Photogrammetry: This makes precise measurements and computations on photographs of the size, shape, and position of objects captured in the photograph. It can also be utilized to compute relative locations (coordinates) of objects and their area and volume. This approach is widely used in engineering fields (e.g., surveying, asset management).

Interpretive Photogrammetry: This is a method of identifying features on a photograph, such as the shape of objects, size, shadow, and pattern to add value and intelligence to information captured in the image.

Another type of classification divides photogrammetry based on the mode of data collection:

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Aerial Photogrammetry: In this case, data are captured from an aircraft with a camera mounted vertically or obliquely towards the ground. During aircraft flight, multiple overlapping photos of the ground are captured. Traditionally, aerial photographs were captured using fixed-wing manned aircraft, but many projects now are done with unmanned aircrafts. Conventionally, these overlapping photographs were processed in a stereo-plotter, but currently are often processed by automated desktop systems (Mollard 2013).

Terrestrial and Close-range Photogrammetry: The camera is located on the ground and handheld, tripod, or pole mounted. The close-range photogrammetry is usually used for non-topographic applications (e.g., 3D architectural models) (Luhmann et al. 2006).

The extent of photogrammetric applications is continuously increasing especially with the improvements in computer technology. "Digital photogrammetry" is both aiding the professional to envisage complex applications as well as making the activity more user-friendly for beginners (Koschitzki et al. 2017). Recent applications of digital photogrammetry range from monitoring landslides (Bouali et al. 2017) to identifying underwater dive sites.

A multitude of software, both proprietary and freeware, are available for digital photogrammetric modeling and analysis (Oats et al. 2017). Proprietary software, such as Agisoft Photoscan, SocetGXP, PCI Geomatica, and Imagine Photogrammetry, has helped in instilling complex photogrammetric modeling into scientific research and surveying applications. There are freeware packages available albeit with limited functionalities. This includes programs such as Colmap and Micmac, both of which feature in a Linux platform. Apart from these, there are photogrammetric tool sets included within software packages, such as Matlab.

With its range of applications from reconnaissance surveying to terrain analysis and underwater bathymetry, photogrammetry is an invaluable asset in geospatial studies, environmental studies, and engineering geology. The 3D depiction of surfaces made possible through stereoscopic photo pairs extends their application to archaeological studies, planetary research, city planning and a wide range of scenarios.

Cross-References

- ► Aerial Photography
- ► Engineering Geological Maps
- ► Engineering Geomorphological Mapping
- ► Hazard Mapping
- ► InSAR
- ► Landforms
- ► Landslide
- ► Lidar
- ► Site Investigation

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Physical Weathering

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Synonyms

Disintegration; Fragmentation; Mechanical weathering; Slaking

Definition

Physical weathering of rocks consists of their physical disintegration, without chemical weathering, by several physical processes such as (a) significant diurnal and/or seasonal thermal variations, (b) expansion and fracturing of rock due either to stress relief or increase of pressure in rock pores and fissures by expansion volume associated with water freezing, and (c) mechanical actions of several weathering agents including water flow, glaciers, wind, living organisms (e.g., roots of trees, cavities made by rodents), and the growth of poorly soluble salts, in rock pores, either by crystallization pressure, such as hydration pressure or by differential thermal expansion. These are of fundamental importance in the breakdown and fragmentation of rocks.

Introduction

The weathering of rocks is an important geological phenomenon because it leads to the formation of soils (whether residual or sedimentary).

The disintegration and decomposition of a rock, through the process of weathering, by physical, chemical, and biological agents, will degrade it into small fragments and modify the rock properties leading to the formation of new minerals, turning it into a different natural product, and more adjusted and in physical and chemical equilibrium with the geological environment that exists on the surface of the Earth.

Another important process in the disintegration of rock material is salt weathering, common in hot regions with a desert climate. This disintegration is due to pressures made by the growth of less soluble salts by crystallization in the pores and fissures of the rock materials.

Weathering is a complex geological phenomenon because it depends on several factors such as the climate, relief, time of exposure, weathering agents, and types of rocks. Many authors have highlighted both the importance of analyzing combinations of weathering processes and to produce adequate test methods to real conditions and the need to consider conditions with different scales of observation and measurement, either temporal and/or spatial, of the factors affecting a weathering of rocks in different weathering processes. This fact has led to increased interest in producing new testing methods with multiple scales for a variety of weathering processes (e.g., McCabe et al. 2010).

Often, the weathering of rock masses is conditioned by their discontinuities (e.g., joints, faults, stratification, schistosity) that facilitate access to weathering agents. Weathering leads almost always to an increase in porosity and deformability of a rock, a decrease in bulk density and strength of rock and thus its durability.

Physical Actions and Agents

Physical or mechanical weathering is the set of the transformations undergone by a rock or a rock mass when exposed to the conditions of an exogenous geological environment in which weathering agents, such as the atmosphere, hydrosphere, and biosphere, produce a series of physical actions, the main ones of which are:

(a) Significant temperature changes:

Physical weathering predominates in dry climate regions. Extreme temperature changes can be favorable to the occurrence of exfoliation on rock mass surfaces. The temperature level and its consequence in rocks are conditioned by the thermal anisotropy of the mineralogical components.

Rock-forming minerals show different thermal conductivity, producing swelling and shrinkage at diverse rates. Temperature variation produces expansion and contraction of rocks, which consequently induces different stresses between mineral grains, leading to the mechanical breakdown of rock material.

Rocks are poor conductors of heat, so the differential thermal expansion between the surface and inner parts of rock masses leads to exfoliation and stresses forming cracks.

(b) Water freezing:

Cold temperatures can cause expansion of water, as ice, in rock joints and lead to widening of the discontinuities and consequently to rock crumbling and disintegration. This is frequently observed at high latitudes and high altitudes, where temperatures lower than 0 °C are common.

The freeze and thaw behavior is dependent on several characteristics such as mineralogical composition, texture, and pore properties (Lisø et al. 2007) and also of degree of saturation. Frost weathering is a major physical deterioration factor. Water congelation produces a volume increase of about 9%, applying a pressure that can overcome the strength of pores walls and cracks by several orders of magnitude (Matsuoka and Murton 2008). In areas where freezing and thawing happen several times a year, disaggregation by ice wedging is more frequent than in areas where water is permanently frozen. More important than volume expansion accompanying the phase change from water to ice is the growth of ice lenses. Alternation of freeze and thaw cycles can cause major damage to buildings constructed with natural stone during winter. In temperate humid climate areas, frost can reach 1–2 m deep. In periglacial zones, it is possible that the water freezes to a depth up to tens of meters, whereas in glacial areas water can freeze to more than 100 m.

(c) Crystallization of salts:

Crystal growth is caused by saline solution saturation, evaporation, and/or temperature variations or by mixing of salts in solution. Salt crystallization corresponds to a mechanical process that disturbs the integrity of the rock. It is one of the most significant weathering agents that a porous rock undergoes at/or near the Earth's surface, especially in arid, marine, or urban environments as well as in temperate climatic zones. Salt formation applies high crystallization pressure within rock pores, causing the deterioration of rock texture. Crystallization, or ice growth, in porous materials presents analogous mechanical behavior, with the increase of pressure between grains or within fractures or fissures. This pressure can vary according to the type and dimensions of the rock pore.

Crystallization of salts such as halite (NaCl), gypsum ($CaSO_4.2H_2O$), sodium sulfate (Na_2SO_4), sodium carbonate (Na_2CO_3), calcium carbonate ($CaCO_3$), sodium nitrate ($NaNO_3$), and potassium nitrate (KNO_3) can cause cracking, expansion, flaking, granular disintegration, and surface powdering. The carbonate $CaCO_3$ is frequent in semi-arid zones, sulfates occur predominately in dry areas, and chlorides are very common in coastal deserts (Cooke et al. 1993). Salt weathering can also be connected with the expansion of salts in limited spaces caused by heat and with stresses originated by hydration. Salt crystallization disturbs rocks and building stones and is an important cause of degradation of monuments and buildings.

(d) Wetting and drying and expansion and contraction of clay minerals:

Wetting and drying cycles are one of the most common phenomena to which rocks are exposed. The presence of water provokes the weakening of mechanical properties of rocks. The changes in water content cause linear or volumetric variation, which can lead to the degradation of rock material, particularly due to the presence of expansive clay materials. The swelling and shrinking of clay correspond, respectively, to the physical effects of water gain and water loss.

In the case of foliated rocks, wetting and drying cycles can lead to a progressive rock detachment and fragmentation, particularly along anisotropic surfaces (Andrade and Saraiva 2010). The cycles cause fissures, and enlargement of existing pores and fissures increase the disintegration of the geological materials. The wetting and drying action can provoke a physical breakdown known as slaking. Mudrocks and slates show a brittle nature and sensitivity to water and tend to easily disintegrate when submitted to wetting and drying cycles.

(e) Abrasion and stress relief:

Abrasion is mechanical wear on the rock by the action of wind, water, or ice. Abrasion effects are considerable in arid and glacial regions. Erosion processes decrease the overburden *in situ* stresses on the underlying rock mass, thus leading to a rock expansion by stress relief and increasing the exfoliation rate as well as the increasing of porosity and permeability of rock and rock mass, thereby allowing the access of water and, consequently, the occurrence of chemical weathering (Fig. 1).

Physical Weathering, Fig. 1 Relief joints in granites located in northwest region of Portugal (Photo by P.S. Andrade)

Characterization of Weathering

Description and classification of the state of weathering is fundamental for adequate characterization of rocks and rock masses, together with other important geological information, such as the mineralogy, lithology, grain size, texture, fabric, and strength of the rock material and the structural characteristics of the rock mass.

The state of weathering is a feature of a great importance in engineering works, particularly in selecting geological materials for construction and in studies for foundations and slope stability.

In site investigation, the variability of the geotechnical properties of rock materials due to weathering processes is a very important issue, and for that the state of weathering must be described and classified in an objective and consistent way. In fact, the description of a rock material is always somewhat subjective especially the assessment of the state of rock weathering by visual examination (Pinho et al. 2009).

Weathering can be described in terms of an increase in rock material discoloration as the degree of weathering increases. Discoloration of the rock, as well as the rock to soil ratio, may be good criteria for the evaluation of the degree of weathering by degradation and disintegration during the process of transformation of a rock into a soil (Dearman 1986).

In approaches to standardize the procedures concerning the description of the state of rock weathering, there is a certain uniformity among some of these systems such as the "*BS5930:1981*" of the British Standards Institution, the *Basic Geotechnical Description of Rock Masses* of the International Society of Rock Mechanics and *Rock and Soil Description and Classification for Engineering Geological Mapping* of the International Association of Engineering Geology, there is a diversity of opinion about the most appropriate method. There is some agreement regarding the fact that the rock can be classified into five grades, from the fresh state to the completely weathered state. Some authors consider these systems very simplistic, recommending a need for their review (Cragg and Ingman 1995).

Physical Weathering, Fig. 2 Aspect of the weathering profile of a rock mass of the Baixo Alentejo Flysch formations, showing the influence of the structure in the weathering of these rock masses (Photo by A. Pinho)

The characterization of the weathering profiles can be difficult because some geological formations consist of very heterogeneous and anisotropic rock materials with distinct grades of weathering and different geomechanical behavior (Hencher and McNicholl 1995). An example is the flysch-type deposits of the Baixo Alentejo Flysch Group that occur in the South of Portugal. These turbidites, fully described in Pinho (2003), consist of sequences of usually thick greywacke beds that alternate with thin shale beds and, thus, consist of both heterogeneous and anisotropic rock masses with a great structural complexity. This strongly influences and controls processes of weathering, as shown in Fig. 2.

The influence of the lithology on the weathering of the material should also be considered and understood. The consequences of the weathering may be different for different types of rocks, so a classification at the scale of rock material and for all rock materials does not seem feasible. Each situation needs to be considered separately.

It is essential to recognize that there is a need for the classification of the degree of weathering at two different dimensions: (i) small scale (rock materials) and (ii) large scale (rock mass).

The classification of material by weathering classes is important and should be required but, generally, is only applicable to small volumes of material. However, a classification by zones of weathering is not applicable to small samples such as cores from drill holes, although it is useful to group together large volumes of rock mass with altered characteristics that are approximately similar.

On the subject of the description and classification of rocks and rock masses, based on weathering profiles, the contribution of the Geological Society of London (Anon 1995) should be highlighted. This contribution provided a historical overview of classifications based on weathering profiles, as well as some recommendations about classifications for particular situations.

Products of Physical Weathering and Applications

Products of physical weathering of rocks are varied and diverse in their textural and mineralogical consequences. They comprise all loose material ranging in particle size from clay (<0.002 mm) to gravel and boulders, between 3 and 5 m in diameter.

First, there is disintegration, caused by wetting and drying or by freezing and thawing which produce cracks in the rock. Second, there is erosion, caused by the action of glaciers, water flow, or wind. These processes produce a set of particles of varying sizes, which are still composed of the same material as the parent rock.

Resulting from the first phase of the rock mass weathering, residual soils are the closest weathering products to the original rock because there was no transport of materials away from the site of origin. These geomaterials have a granular nature, in temperate and cold climates, known as saprolite, or young residual soil. In these types of climates, physical weathering predominates over chemical weathering, producing mainly sandy materials which may contain some gravel, silt, or even some minimal percentage of clay. These residual soils are distinguished from sedimentary soils or transported soils, because the grains are generally angular since they have not been transported and they preserve in their mineralogical composition many primary minerals inherited from the parent rock, such as quartz, feldspar, mica, and amphibole, among others (Duarte 2002).

Saprolitic residual soils are exploited in open-pit mines such as gravel and sand pits, since they are widely used as raw materials in geotechnical structures, particularly in the construction of embankments for roads, railways and Earth dams, and in the granular layers of pavements. The angularity of the grains and the mineralogical composition (primary and resistant minerals) provides good characteristics of shear strength and contributes to the stability of slopes.

On the other hand, sedimentary soils have rounded grains because of the main physical processes (fragmentation, abrasion, polishing) to which they were subjected during transport by the weathering agents.

Pebbles are small, smooth, and round due to the action of water or sand. They are comprised of diverse and hard materials and are widely used in construction as aggregates, in the construction of buildings; drainage structures, such as drainage spurs (100/200 mm in diameter) and drainage masks (100/500 mm in diameter) on the excavated slopes of roads and railways; drainage blankets at the foot of Earth dams or drainage layers in embankments, filling “gabions”, or even as decorative stones due to the diversity of colors, sizes, and attractive shapes.

Sand and silt particles produced by physical weathering generally consist of single rock minerals, rather than combinations of these, as is the case in their parent rock or in gravel-

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sized material. Sand is usually used in the manufacture of mortar, in the construction of buildings, and in the composition of vertical drains and filters in various geotechnical works.

According to Wesley (2010), the fine-grained materials produced by physical weathering never have the properties of clay because the chemical process does not form true clay mineral particles do not occur. This finest geomaterials (<0.002 mm) are used in the manufacture of mortar, cement, bricks, tiles, adobes, ceramics, pottery, and sealing material in sanitary or industrial landfills.

In contrast, the coarser products of physical weathering of the rock formations, boulders, are widely applied in the foundations of buildings, as aggregates in civil construction, breakwaters, maritime protection of beaches and ports, slope stabilization, rock embankments, riprap dams, raw material for crushed stone, and ballast.

Summary

Physical weathering is the fragmentation of rocks when exposed to the conditions of an exogenous geological environment in which weathering agents, such as the atmosphere, hydrosphere, and biosphere, produce a series of physical processes of which the most effective are the mechanical breakdown of rocks by water freezing in rock voids and joints and changes of temperature and water content in the unsaturated rock mass zone. Physical weathering processes do not change the chemical and mineralogical composition from that of the parent rock, and it is more effective near the ground surface. The physical weathering of the exposed rocks leads to their disintegration and fragmentation of debris into smaller dimensions without marked mineralogical changes.

Physical weathering and chemical weathering usually work together; however, the predominance of one or the other depends on the climatic conditions. The physical weathering of rocks is most intense under severe climatic conditions, in particular with freeze-thaw and thermal extreme variation, typically in regions of cold dry and desert climates.

Cross-References

▶ Abrasion
▶ Aeolian Processes
▶ Alteration
▶ Bedrock
▶ Biological Weathering
▶ Cap Rock
▶ Characterization of Soils
▶ Chemical Weathering
▶ Classification of Rocks
▶ Classification of Soils
▶ Clay
▶ Crushed Rock
▶ Deformation
▶ Desert Environments
▶ Durability
▶ Erosion
▶ Expansive Soils
▶ Glacier Environments
▶ Hydraulic Action
▶ Hydraulic Fracturing
▶ Hydrothermal Alteration
▶ Igneous Rocks
▶ Infiltration
▶ Karst
▶ Limestone
▶ Mechanical Properties
▶ Metamorphic Rocks
▶ Normal Stress
▶ Percolation
▶ Permafrost
▶ Petrographic Analysis
▶ Pore Pressure
▶ Pressure
▶ Residual Soils
▶ Rock Mass Classification
▶ Rock Mechanics
▶ Rock Properties
▶ Sand
▶ Sediments
▶ Shale
▶ Silt
▶ Soil Mechanics
▶ Soil Properties
▶ Strength
▶ Stress
▶ Tropical Environments
▶ Vegetation Cover
▶ Volcanic Environments
▶ Water

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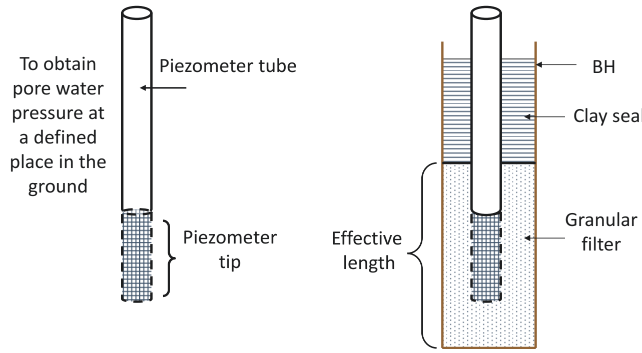
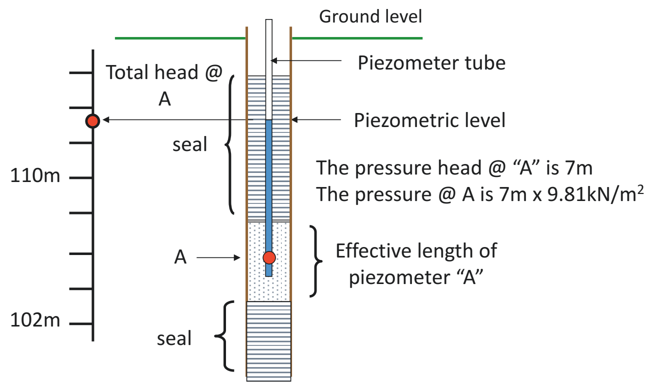


Piezometer, Fig. 1 A basic piezometer of the Casagrande type



Piezometer, Fig. 2 The installation of a piezometer and interpretation of the levels involved

Piezometer

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Definition

A pressure meter used to measure groundwater pressure at a point in the ground allowing it to be quantified and, from that, the pressure head (see Equipotential Lines) and pore pressure to be calculated.

The simplest piezometer is a tube (piezo-tube) on the end of which is a perforated section (piezo-tip). The tube and its tip are lowered into a borehole whose location has already been selected on the basis of either the likely pattern of groundwater flow or the need to know pore pressure at that location. The tip is placed at an elevation in the hole where a measurement of pressure is required; this elevation is normally chosen on the basis of the geology penetrated and pattern of flow expected. The tip is then surrounded by granular material which serves three purposes: (i) to prevent the ground around the tip from collapsing into the annulus between the tip and the borehole wall and in so doing possibly creating pathways in the surrounding ground which connect that point to others elsewhere (often above) where pressures may be different, (ii) to prevent fine particles of clay and silt size from being flushed from the ground into the tip itself, and (iii) to create in the ground a permeable insert of known length and diameter which can be used as a basis for conducting tests of *in situ* permeability via the piezometer tube; the length of this zone is often slightly greater than the length of the piezo-tip it surrounds and is known as the effective length of the tip.

The whole assembly is then sealed in the hole by a means appropriate for the depth of the instrument; deep locations may need clay-rich grout, whereas shallower locations can often be filled with clay pellets (bentonite rich) which can be poured down the hole in the annulus between the piezo-pipe and the borehole wall and then hydrated. Figure 1 illustrates such an arrangement; the effective length should not be so great as to undo the purpose of the instrument which is to measure pressure at a “point” in the ground. Figure 2 explains how the system is used *in situ*; groundwater flows from the ground into the granular filter and thus into the tip where it raises a column of water in the tube. The height of the column is taken to balance the pressure at the midpoint of the effective length of the tip. The level of water in the tube is the

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piezometric level for the groundwater at its point of measurement around the tip. The elevation of the point of measurement is the elevation head of the water at that point (meters above datum) and height of the column of water in the tube is its pressure head (also in meters). In Fig. 2, the elevation head at (A) + the pressure head at (A) = the total head at (A).

Such instruments are suitable for ground whose permeability allows water to move speedily between the ground and the tip, for example, silt, sand, gravel, and fissured and coarsely porous rock. In ground of lesser permeability (clay and clay-rich sediments, mudrocks, and essentially unjointed igneous and metamorphic rocks), the time taken for a quantity of water of sufficient volume to enter and leave the tip is so great that the response of the instrument lags behind changes occurring to the pressure of water in the ground (Hvorslev 1951). Under these circumstances, the permeable tip is replaced by an electronic device that measures pressure directly. Means for overcoming this problem are continually developing (McKenna 1995).

Care should be taken to appreciate the difference between a piezometer and a standpipe as the latter is designed to measure the elevation of a water table and does not provide a value for pressure at a "point."

Piezometers provide the key information needed to quantify the physical response of groundwater, the magnitude of groundwater pressure, and thus the value for effective stress *in situ*. Consequently, there are comprehensive guidelines governing their design and use which should be followed; examples are the British Standards BS EN ISO 22475 *Geotechnical investigation and testing – Sampling methods and groundwater measurements (Parts 1–3)* BS EN ISO 22476 *Geotechnical investigation and testing – Field testing (parts 1–12)* BS EN ISO 22282 *Geotechnical investigation and testing – Geohydraulic testing (parts 1–6).*

Cross-References

▶ Aquifer
▶ Artesian
▶ Boreholes
▶ Borehole Investigations
▶ Capillarity
▶ Casing
▶ Characterization of Soils
▶ Classification of Soils
▶ Clay
▶ Desiccation
▶ Dewatering
▶ Equipotential Lines
▶ Effective Stress
▶ Field Testing
▶ Fluid Withdrawal
▶ Groundwater
▶ Grouting
▶ Hydrogeology
▶ Hydrology
▶ Instrumentation
▶ Monitoring
▶ Percolation
▶ Pipes/Pipelines
▶ Pore Pressure
▶ Saturation
▶ Site Investigation
▶ Subsurface Exploration
▶ Voids
▶ Water
▶ Wells

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Pipes/Pipelines

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Synonyms

Conduit; Pipeline network; Tube

Definition

A long system of pipes, typically metal but which can be made of plastics or composite fibers, used to convey natural gas, oil, water, and related liquids across long distances in isolation from the environment, either above or below ground (but the latter is most common), and in which flow is controlled by a series of valves, pumps, and control devices along the network.

Introduction

The global pipeline network is vast, covering millions of kilometers of geology and terrain while transporting essential liquid goods within and between nations and to users around the world. In doing so, pipelines are exposed to more geological conditions than almost any other infrastructure resource, except perhaps roads and railways. In urban environments, pipes and pipelines distribute everything from water and sewage to gas. Pipe segments convey water beneath roadways as culverts and perform a critical part of the storm water drainage systems of a city.

Damage to any part of a pipeline system can have dramatic economic, environmental, and social impacts, threatening the viability for what remains, at least at present, a critical human infrastructure.

Engineering geology and geomorphology help to reduce threats to pipelines at every stage of their life cycle, from prefeasibility and routing, through operations and even deactivation, by delivering a deeper understanding of the ground conditions and processes acting on the network of pipe.

Prefeasibility

At prefeasibility stages, a pipeline or utility company must decide whether or not there is an economic route between source and destination to build a pipe. Considerations are increasingly social and environmental; however, engineering geology helps define and characterize the types of ground that various route options will traverse, and the relative geological challenges along those routes. Geology and geomorphology are interpreted in a general manner to provide route data related to:

Topography (where more relief in a shorter distance generally increases the challenge of building a pipeline)
Depth and type of bedrock (where harder bedrock close to the surface means expensive excavations)
Seismic conditions (does the route cross active faults)
Existence of landslides (ground movement presents significant challenges to the construction and operation of pipelines)
Existence of soluble bedrock (subsidence and cavities present significant challenges to the construction and operation of pipelines and present unique challenges to the environment should a release occur at any time)
Existence of permafrost and thermokarst terrain (where the potential to change existing conditions by melting of ground ice presents significant challenges to the construction and operation of pipelines)
The type and nature of rivers (particularly related to the potential depth of the active bed, the lateral and vertical stability, and the height and composition of the banks on either side). Sometimes at the prefeasibility stage it is sufficient to know how many major rivers must be crossed
The depth and general properties of soil and rock (particularly where soil may be aggressive and increase corrosive action on the pipeline, or where soil properties combined with topography, may present ground movement hazards – in cold regions this may include frost heave)
The amount of water bodies (including muskeg and organic soils) to be crossed that might require special treatment and equipment during construction

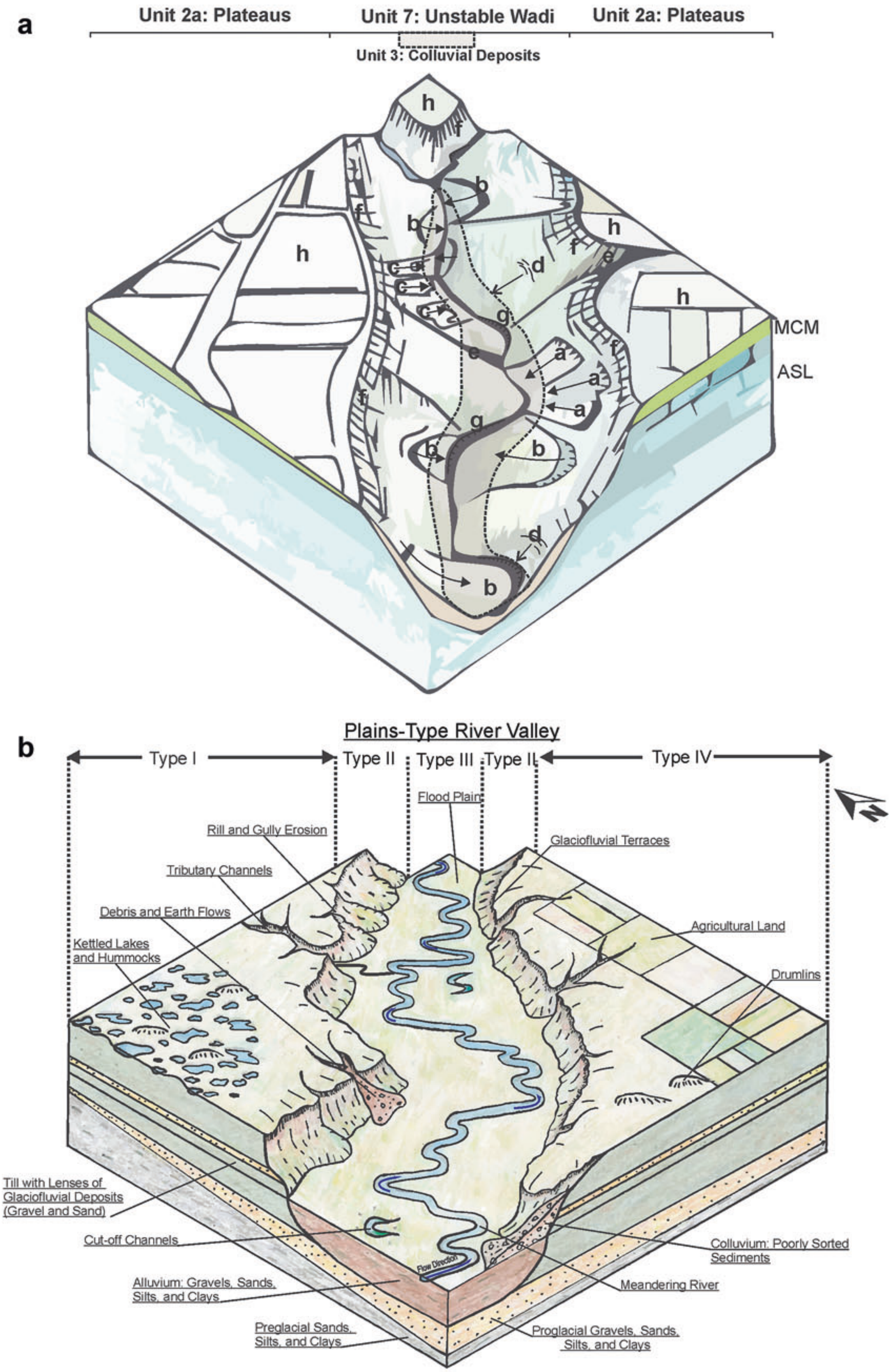
The engineering geologist may be asked to consider several route options and to provide an objective comparison between those routes and the challenges they present for environmental impact assessments. Specific types of constraints tend to be grouped to portions of the landscape, and comparisons may be made by creating geological and geomorphological models of the constraints and conditions that may be faced (Figs. 1 and 2). The routes are mapped according to the amount of time in each of the geological and geomorphological archetypes, and in this manner, they can be compared. Geological and geomorphological challenges are typically overlaid with social, biological, and other environmental studies to determine the ultimate path.

Feasibility and Design

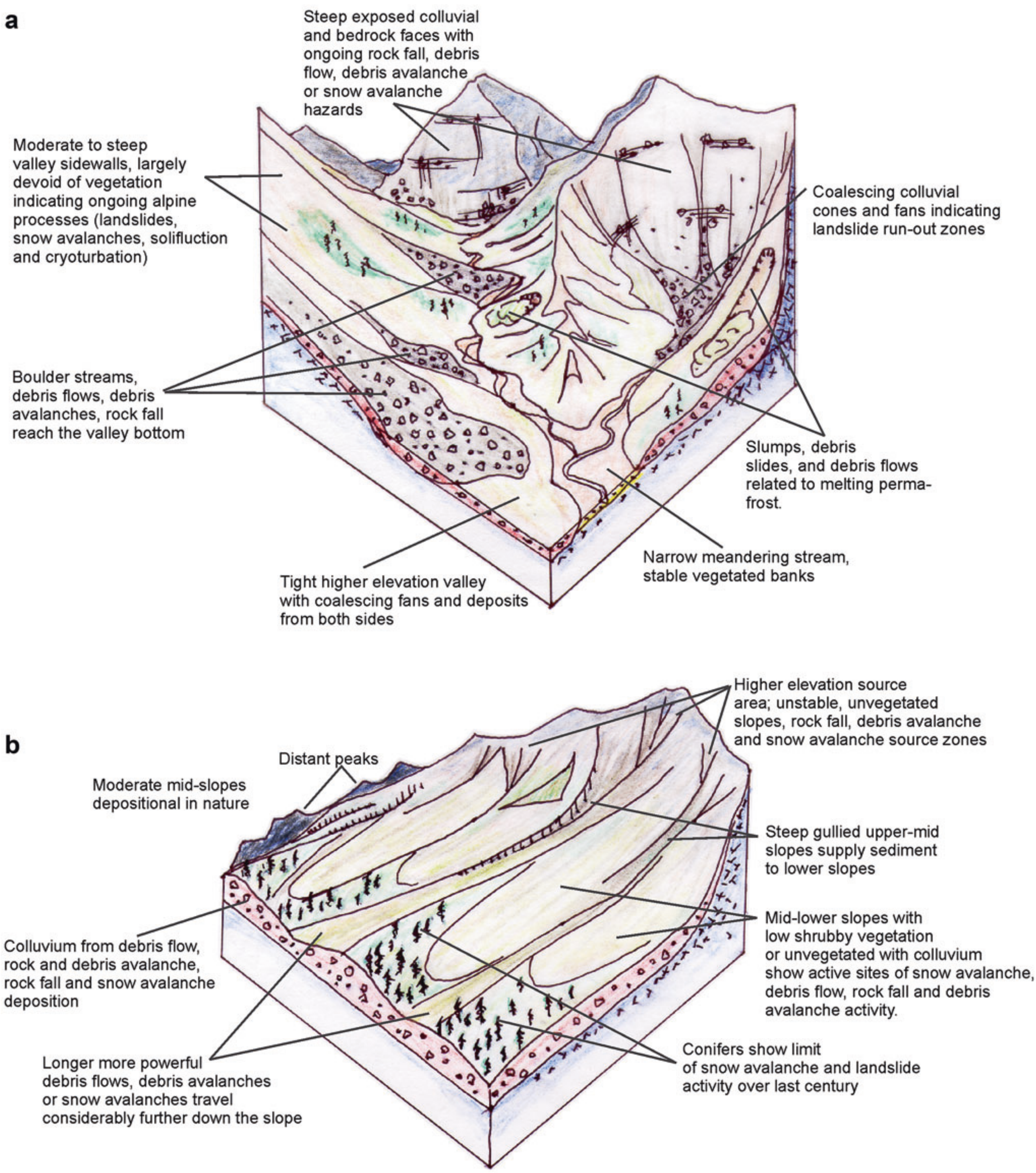
At the feasibility and design stages, considerably more detail is required to determine the nature of the ground to be crossed and how it might affect the pipeline. Typically, detailed terrain and hazard mapping is conducted to refine the route and provide specific information about the type and nature of constraints to pipeline construction or operations along the way. This information may include:

A landslide inventory that includes the type and nature of landslides crossed by the pipeline (or proposed to be crossed by the pipeline)
Seismicity of the region including presence of faults, amount of fault displacement, potential for liquefaction, and dynamic ground motion
Erosion potential related to the right of way, construction back fill, and subsurface (piping) erosion
Uplift displacement (frost heave)
Characteristics and extent of ground ice (permafrost) and thermokarst hazards
Potential geochemical concerns such acid rock drainage and/or karstification
Watercourses and associated hazards (scour, bank erosion and migration)
Unique soil structures such as boulders/cobbles in soil mass, presence of competent bedrock, or sensitive soils
Hydrotechnical hazards including scour, bank erosion, channel migration, and buoyancy

Detailed maps are normally produced along the route (Fig. 3) and spatial data are correlated to data gathered in



Pipes/Pipelines, Fig. 1 Geomorphological models of terrain in (**a**) arid wadis and (**b**) a plains-type river valley



Pipes/Pipelines, Fig. 2 Geomorphological models of two types of terrain in northern latitude mountains. Figure modified from Guthrie (2011)

subsurface investigations (geotechnical boreholes). In urban areas, a comprehensive or very targeted subsurface investigation is typical.

Additional geotechnical analyses may be required depending on the results of the surface and subsurface investigations. Such analysis often includes detailed slope models and factor of safety analysis under a variety of conditions. It may also include developing soil-spring interaction models with the pipeline engineers to better predict the performance of pipe under different conditions (grade, wall thickness, pressure, etc.).

Pipes/Pipelines, Fig. 3 Detailed mapping for a pipeline route. Terrain mapping (**a**) provides fundamental information about the type and nature of ground crossed by a pipeline. Codes are correlated to geotechnical and engineering properties of the soil. Hazard or Constraints mapping (**b**) provides specific construction advice based on constraints encountered on the route

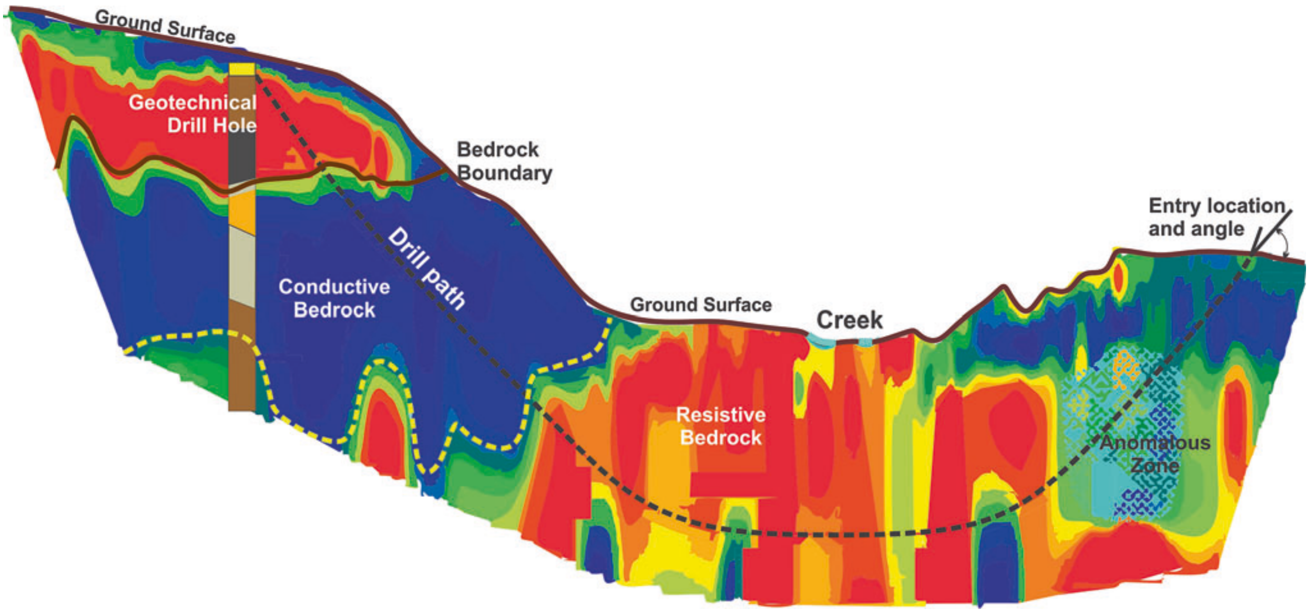
Constraints at major watercourse crossings are increasingly being solved using trenchless techniques (such as horizontal directional drilling). Here geotechnical engineering provides critical subsurface information about the conditions along the expected drill path. Data come from geotechnical drill holes and geophysical surveys are combined to give a clear picture of the subsurface conditions. Using these data, a construction crew can select the best boring method, drill bits and path, avoiding hydraulic fracturing, construction delays, cost overruns, and even failure to successfully complete the bore (Fig. 4).

Finally, traditional geotechnical inputs to foundation design provide support to the construction of facilities built to process and manage gas and liquids moved by pipelines.

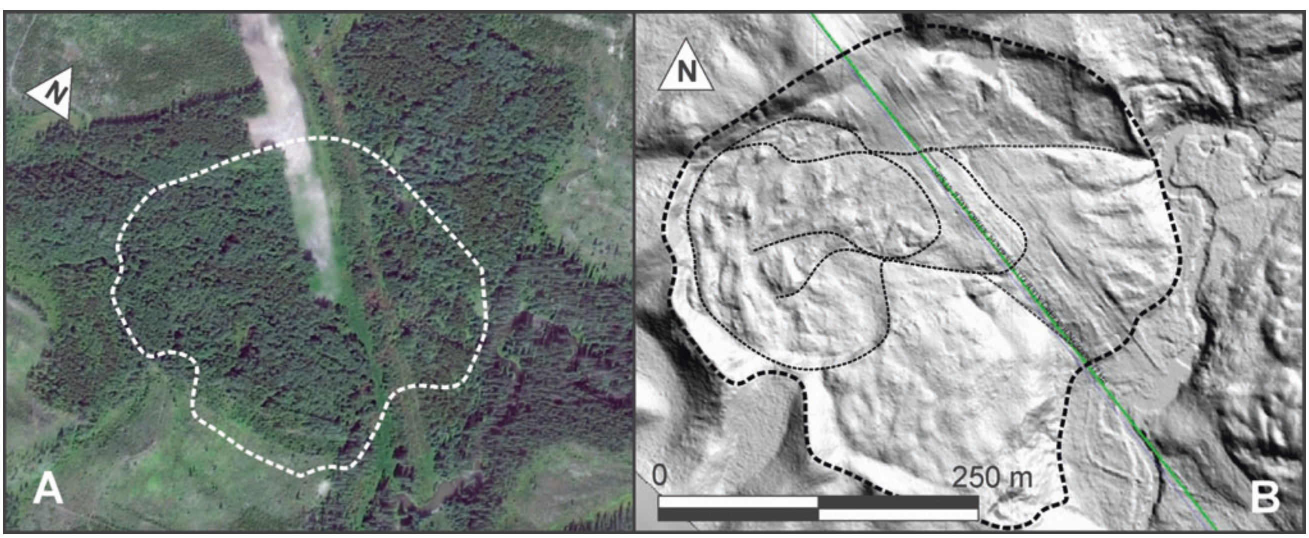
Operations and Maintenance

Pipelines have a limited design life; however, like many human structures, they frequently persist and perform beyond the original intended years. They are impacted over their lifespan by a multitude of factors that impact their integrity. Those factors include corrosion, metal loss, physical damage, and, importantly for engineering geology, ground movement.

Ground movement may be a result of frost heave, liquefaction, landslides, river scour (the movement of the active portion of the river bed), chemical weathering, or melting of ground ice. Whatever the reason, ground movement may induce sudden strain on a pipeline and threaten its overall integrity. Given the vast distances covered by pipelines, geotechnical and hydrotechnical hazards are common.



Pipes/Pipelines, Fig. 4 Subsurface investigation for a trenchless crossing of a river (also avoiding steep slopes on the left bank). Data from geotechnical boreholes (see column in left third of picture) are combined with geophysics to provide subsurface conditions along the drill path



Pipes/Pipelines, Fig. 5 Aerial photograph (**a**) and LiDAR (**b**) image of a pipeline right of way that crosses a large old landslide

Pipeline failures related to ground movement (in this case, failure is understood to mean an unintended condition where the pipeline no longer isolates its contents from the surrounding environment) occur at a rate between 0.02 and 0.03 failures/1000 km/year in North America and Europe, up to an order of magnitude higher in South American countries and up to an order of magnitude less in Australia (Porter et al. 2016). Not surprisingly, most of the reported failures are related to slopes, and statistics are as much a reflection of geological conditions as they are of practice.

Engineering geologists may be asked to examine moving or potentially unstable ground to determine the actual threat to a pipeline. These investigations can be substantially more detailed and focused than investigations at a routing stage because they relate to a clearly defined problem and location.

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Desktop studies include a review of the original terrain mapping or geomorphology, surficial geology, bedrock geology, and where available, the pipeline design documents. Updated aerial photograph, satellite imagery, LiDAR, and InSAR data may be available for interpretation providing a substantially superior understanding of ground conditions than were available when the pipeline was originally put into the ground (Fig. 5). Similarly, engineering geologists will work with integrity engineers to understand the results of in-line inspections that use inertial measurement units to detect bending strains and their locations on the pipeline itself.

For slopes or subsidence, a field program may include geotechnical drill holes, field inspections for and surveys of characteristic features of ground movement such as tension cracks, head scarps, transverse scarps, back tilted blocks or rotated ground, toe bulges, lateral shear zones, horst-and-graben features, isolated ponds, caves or lesser karst features, and disturbed vegetation.

For rivers, a field program may include geotechnical drill holes (on banks), field assessment of river morphologies and processes, assessments of vertical and lateral stability, and assessment of bank erosion hazard.

Probability of failure is often estimated based on the data gathered whereby the engineering geologist determines the likelihood that observed ground movement will induce a strain on the pipe (bending strain, vortex induced vibration, impact force, etc.) and the integrity engineers then identify the amount of strain that the specific pipe can handle (usually as a measure of elastic strain and total strain).

As technology and understanding improves, pipeline operators are increasingly focused on widespread geohazards programs, which aim to find and prioritize threats to existing lines before they become acute. There is, therefore, an operational component of pipeline integrity that is informed by engineering geologists and is similar to the feasibility and design stages described above. The fundamental difference is that, in this case, the pipeline is already fixed in place, and accurate identification of hazards (and possible mitigations) is critical.

Mitigation

The mere existence of a hazard or potential hazards does not necessarily mean that a pipeline will fail. Engineering geologists are therefore often asked to provide input to mitigation options. For very slow, episodic slopes, or slopes where movement is uncertain such as the case with very old landslides detected using LiDAR (Fig. 5), instrumentation or a repeatable monitoring program is often useful.

Monitoring is traditionally completed by surveying known points repeatedly through time or by installing slope inclinometers, slope accelerometer arrays, piezometers, extensometers, or other similar instrumentation. Ground instrumentation can be connected to emergency shut-down devices as required.

Increasingly, monitoring is conducted using differential LiDAR analysis and InSAR image stacks. These technologies allow much broader spatial coverage of the ground surface, and their use is likely to increase with time.

For ground that moves intermittently, monitored sites provide data that can be input into time to failure analysis (Voight 1989; Borsetto et al. 1991), with thresholds related to the actual strength of the pipe.

Watercourse monitoring may similarly include repeated ground investigations, the installation of erosion pins, and surveys of depth of cover compared to the predicted behavior of the river. Stream gauges may be installed and tied to emergency shut-down devices that close pipe valves in the event that flows are particularly high.

Mitigation options begin to fall into the exclusive realm of geotechnical and civil engineering, but engineering geologists are often part of the mitigation team and provide critical inputs and data about the ground conditions, the observations, and the processes that threaten the pipe.

In urban areas, where mitigation often means excavations, engineering geologists are still used to provide critical input and data about ground conditions.

Summary

Pipelines extend across millions of kilometers of ground, intersecting a wide variety of conditions, terrain, geological, and geomorphological processes along the way and are essential infrastructure in urban areas. Engineering geology provides fundamental understanding of the nature and characteristics of this ground, necessary to safely and effectively build, operate, and maintain a pipeline.

Cross-References

► Erosion
► Factor of Safety
► Geophysical Methods
► Geotechnical Engineering
► Hazard Mapping
► Hazard
► InSAR

▶ Instrumentation
▶ Karst
▶ Landslide
▶ LiDAR
▶ Liquefaction
▶ Monitoring
▶ Permafrost
▶ Strain

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Plastic Limit

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Definition

The plastic limit is one of the measured parameters of the Atterberg limits test (ASTM, 2010), which is used for differentiating consistency states of finer particles in soil material. If coarser particles are present (coarse sand, gravel, cobbles), the finer particles act as matrix and may govern the behavior of the soil mass. Consistency states depend on water content; with increasing water, the consistency states are solid, semisolid, plastic, and liquid.

The plastic limit is the water content at which a soil-water paste changes from a semisolid to a plastic consistency as it is rolled into a 3.175-mm (1/8-inch) diameter thread in a standard test. The second measured parameter of Atterberg limits test (ASTM, 2010) is the liquid limit. The Atterberg limits test also includes the plasticity index, which is calculated as the difference between the liquid limit and the plastic limit. All Atterberg limits are determined on samples of soil that pass the #40 sieve (ASTM, 2009), which has 0.42-mm openings (medium sand size and smaller, including silt and clay sizes, that may be part of the soil material).

Cross-References

▶ Atterberg Limits
▶ Characterization of Soils
▶ Classification of Soils
▶ Plastic Limit
▶ Plasticity Index
▶ Soil Laboratory Tests

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Plasticity Index

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Definition

The plasticity index (*PI*) is calculated as the numerical difference between the liquid limit (*LL*) and plastic limit (*PL*); $PI = LL - PL$. These three parameters collectively are the Atterberg limits (ASTM 2010). The liquid limit and the plasticity index are the axes of the plasticity chart (Fig. 1), which is used in engineering to classify fine-grained soils, which are defined as soils with 50% or more passing the #200 sieve (ASTM 2011). The #200 sieve has 200 openings per inch, or 200 openings per 25.4 mm, with 0.074-mm openings (ASTM 2009). Particles that pass through a #200 sieve are classified as fine-grained (silt, clay), and typically referred to as “fines,” whereas particles that are retained on a #200 sieve are classified as coarse-grained (sand, gravel, cobbles, boulders). The Atterberg limits test also is performed on the matrix of coarse-grained soils, which is the fraction of a soil sample that passes the #40 sieve (0.42-mm diameter openings; medium sand size and smaller, including silt and clay sizes).

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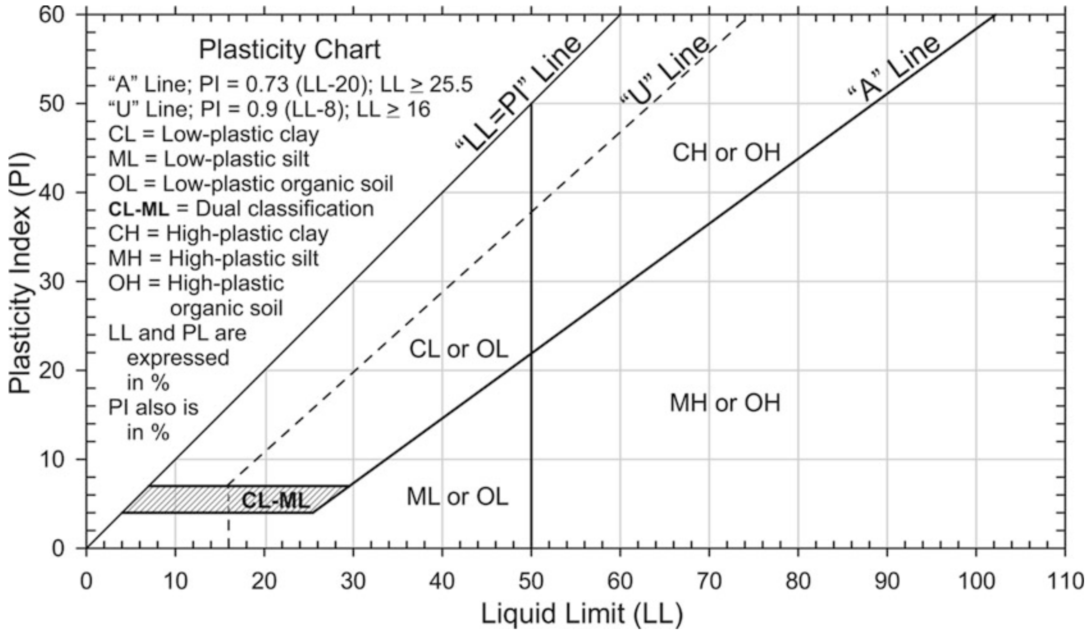

Plasticity Index, Fig. 1 Plasticity chart used with the Unified Soil Classification System (ASTM 2011). The plasticity index is defined as the difference between the liquid limit and the plastic limit. The plasticity index must be ≥0; therefore, the lower bound of the chart is defined by the LL = PI boundary

Cross-References

- ► Atterberg Limits
- ► Classification of Soils
- ► Liquid Limit
- ► Plastic Limit

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Poisson's Ratio

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Definition

Poisson's ratio is the name given to a ratio of lateral strain to axial strain. Its recognition as a property of materials is attributed to Siméon-Denis Poisson (1781–1840), a French mathematician and student of Laplace. Uniaxial extension of a solid rod causes it not only to elongate a small amount but also to become more slender or to undergo a reduction in diameter. Conversely, uniaxial compression of a solid rod or rock core sample causes it to shorten a small amount, but also to expand in diameter or become thicker. Therefore, principal stress in only one direction (uniaxial extension or compression) produces strain in all three directions. Hooke's law explained the proportional change in length of a spring to the application of a load. Generalized Hooke's law (Pariseau, 2012) recognizes that strain ($\varepsilon_x = \Delta l / l_o$) is proportional to the applied stress (σ_x), and the constant of proportionality for an isotropic material being stressed and strained in the elastic range of material behavior is the modulus of elasticity (E).

$$E = \frac{\sigma_x}{\varepsilon_x}; \varepsilon_x = \frac{\sigma_x}{E} \quad (1)$$

$$\varepsilon_y = \varepsilon_z = -\nu\varepsilon_x = -\nu\frac{\sigma_x}{E} \quad (2)$$

where ν, Poisson's ratio, is the constant of proportionality between uniaxial strain and transverse strain; the minus sign in Eq. 2 is needed because strain in the y and z directions produced by uniaxial stress in the x direction has an opposite effect. In other words, the strain in the y and z directions resulting from extension in the x direction produces reduction in diameter, whereas an increase in diameter results from shortening in the x direction (Holmes, 2016). An object composed of isotropic material being elongated or shortened by uniaxial stress (principal stress on a plane in which shear stresses are equal to zero) within the elastic range changes shape, but its volume is preserved.

In a laboratory uniaxial compression test on a rock core sample, two strain gauges are applied to the sample, one oriented parallel to the axis and the other oriented transverse to the axis (ASTM, 2014). As the load is applied to the rock core, calculated stress and two values of measured strain are recorded and plotted. The ratio of the transverse strain to the axial strain is calculated for the linear parts of the two stress-strain curves to determine the Poisson's ratio value for the specimen.

A dynamic Poisson's ratio (v_d) can be calculated from surface or downhole geophysical measurements of compression-wave (Vp) and shear-wave (Vs) velocities

$$v_d = \frac{1}{2} \frac{\left(\frac{Vp^2}{Vs^2}\right) - 2}{\left(\frac{Vp^2}{Vs^2}\right) - 1} \tag{3}$$

Cross-References

- ▶ Hooke's Law
- ▶ Modulus of Elasticity
- ▶ Strain
- ▶ Stress

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Pollution

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Synonyms

Contamination

Definition

Accidental or intentional release of harmful substances into the environment.

Introduction

There is an extensive legacy of past and contemporary pollution, in many places, affecting arable land, groundwater and surface water, causing damage to water quality, and short or long-term problems for human health, agriculture, and ecosystems. The term pollution is often used for air and water impacts, whereas contamination is applied to soils (the approach used in this Encyclopedia) but, sometimes, these are treated as synonyms. Effects on soils are dealt with in the chapter on ▶ "Contamination."

Sources

Sources of pollution are diverse including industry, agriculture, domestic and military activities, infrastructure, and transport. Instances may arise because of poor regulation and practices, or equipment failure and unfortunate accidents (such as leakages from storage tanks, vehicles and ships, or fires and explosions).

Pollution can be caused by local problems or can be pervasive over a wider geographical area, particularly as a result of air pollution (e.g., from industrial emissions such as those from smelters). Sources may be localized (point sources, such as leakages) or dispersed (nonpoint sources), such as deposition from air plumes or pollutants spread widely by rivers. It is often more straightforward to deal with point sources than nonpoint sources (Balkis 2012).

Unregulated and poorly designed landfills have been particularly associated with pollution but such facilities are now better regulated in many countries, requiring liners to isolate the landfill. Still, leachates need to be collected and treated before release into the environment (Wiszniowski et al. 2006). Leaching of mine waste tips can cause significant pollution as can discharges of acidic water from some types of underground mines (Johnson and Hallberg 2005).

Types of Pollutants

Pollutants (Laws 2000) include:

- Potentially harmful chemical elements (e.g., arsenic, mercury, lead)
- Manufactured compounds (e.g., fertilizers, pesticides, cleansing agents)

Pollution, Fig. 1 Sample of oil taken from the top of the groundwater at an old uncontrolled landfill, Honshu, Japan (Photograph by the author)

- Organic chemicals (e.g., halogenated and polycyclic aromatic hydrocarbons)
- Pathogens such as fecal bacteria from leaking sewers or discharged directly into the ground from soak-away latrines and cow pastures, or discharges from storm relief drains
- High quantities of sediments, some of which may carry pollutants, that can overwhelm the capacity of drainage systems for steady removal

Transport and Deposition

Particulate, gaseous, and volatile pollutants can enter the atmosphere where they either disperse, and cease to be a problem, or, if in large quantities or seriously hazardous materials, become a dangerous problem if they are retained in the atmosphere or are deposited on the ground or in bodies of surface water. Atmospheric transport is particularly rapid (Gurjar et al. 2010).

Pollutants that quickly seep thorough soils into, or directly enter, groundwater usually cause problems, sometimes short term, but often long term (Bedient et al. 1999) (Fig. 1). This is a major issue for engineering and hydrogeologists. The rate of dispersal depends on the nature of the strata. Infiltration is rapid in porous soils and slower in clay soils. The rate of penetration depends on connectivity, or lack of connectivity, with aquifers through porous strata or fractures and fissures. Transport is particularly rapid in karstic terrains but that can lead to rapid dilution if quantities of pollutants are small and not too hazardous, but can also disperse more hazardous compounds quickly for long distances. Pollution plumes in groundwater can extend for tens of kilometers from the source.

Pollution, Fig. 2 Small dam reducing the rate of water flow from a lagoon at a mineral working to slow the rate of discharge of particulates into a water course, Dorset, United Kingdom (Photograph by the author)

Pollution, Fig. 3 Algal bloom in ditch water due to high levels of pollution containing nutrients, Nottinghamshire, United Kingdom (Photograph by the author)

Pollutants that enter surface water from discharges or the air are not necessarily a problem if the quantities are small and can be easily dispersed, but larger amounts, especially with continuous delivery, cause major environmental incidents (Fig. 2).

A particular problem in agricultural areas is widespread and excessive use of pesticides and fertilizers (Merrington et al. 2002) which infiltrate into the ground and then flush into drains and surface water bodies by rainfall run-off leading to high levels of nitrogen and phosphorus. Phosphorus can stimulate the formation of algal blooms in surface waters (Fig. 3), which disrupt the ecosystem.

Sampling, Testing, and Monitoring

Potability tests of water for human consumption concentrate on detecting harmful pathogens (bacteria, viruses, fungi), observing the need for filtering to remove particulates. However, comprehensive tests are needed to fully characterize pollution (Bedient et al. 1999). Observation wells situated around the margins of landfills and containment facilities are regularly sampled and tested as part of monitoring to ensure that precautions are working adequately (Neilsen 1991). Where a groundwater pollution plume is suspected, it is necessary to check water quality in existing wells and to drill new observation boreholes to establish the extent of the plume and the rate of flow. Electrical resistivity is used to examine levels of pollution (Moghaddam et al. 2017).

Reduction of Risks

Risks of pollution can be reduced by using less harmful products where possible and minimizing the use of potentially damaging chemicals in the environment, for example, not over-using agricultural fertilizers and pesticides. In industrial processes, it is wise to recycle water in closed systems, and minimize the amount of water that needs to go into long-term storage or treatment (Khitoliya 2006). Significantly polluted water should be retained in well-designed lagoons to prevent leakage into the environment for long-term storage or temporary storage until treatment (Fig. 4). In some cases, seepage through reed-beds can allow pollutants to be reduced to safer levels. During ground investigations, caution is needed because site investigation boreholes may mobilize existing pollution.

Pollution, Fig. 4 Discharge into a containment facility of strongly acidic water from cleaning of silica sand, Cheshire, United Kingdom (Photograph by the author)

Treatment

Although pollution can occur easily, remediation is often difficult or, sometimes, impossible in the short, or even long, term. If quantities of pollution emissions are small and sporadic, natural groundwater and surface water flow can attenuate these hazards. However, large emissions of pollutants overwhelm natural processes and become serious hazards to people and ecosystems. Some atmospheric emissions can be reduced at source. There are a variety of approaches to treating water including boiling/distillation; disinfecting using chlorine or by exposure to ultraviolet light or ozone; filtering; or use of activated charcoal to fix contaminants (Drinan and Spellman 2012; Quinn 2013).

Summary

Pollution causes damage to surface and groundwater quality triggering short or long-term problems to human health, agriculture, and ecosystems. Incidents may arise from human activities including accidents. It is necessary to identify potential sources of pollution and, where possible, to prevent these at source by suitable treatment and containment measures and good management practices. Where past or contemporary pollution occurs, it is necessary to examine the nature and extent of the problem through sampling, testing, and monitoring. Treatment is difficult when pollution is severe, but a variety of physical and chemical techniques are available to improve water quality. This topic requires close collaboration between a variety of professionals including geochemists, hydrologists, hydrogeologists, engineering geologists, and environmental and biological scientists.

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Cross-References

- ▶ Acid Mine Drainage
- ▶ Aquifer
- ▶ Aquitard
- ▶ Drilling Hazards
- ▶ Groundwater
- ▶ Hydrogeology
- ▶ Infiltration
- ▶ Karst
- ▶ Liners
- ▶ Water Testing

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Pore Pressure

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Definition

Pore pressure (u) within a soil is the net pressure of the fluids within the voids that exist between the soil particles. Pore pressure is most commonly associated with pore water, which has a pore-water pressure (u_w). Pressures of other fluids such as pore-air pressure (u_a) and other non-soluble materials may be considered for specialized analyses.

The simplest spatial distribution of u_w is the result of hydrostatic pressure, where the pressure at a given point is the result depth below the phreatic surface, and thus u_w increases with increasing depth. Where the phreatic surface or water table is the height of water within observation wells that correspond to a u_w equal to zero. The hydrostatic pressure can be calculated from the unit weight of water (γ_w), and the depth (d) below the phreatic surface (Terzaghi et al. 1996).

$$u_w = \gamma_w d$$

The surface tension of the water interacts with the fine interconnected voids within soil and results in a capillary rise of the water above the phreatic surface to a height that is a function of the effective diameter of the interconnected voids. This rise of water above the phreatic surface and tension within the water results in a negative u_w (Terzaghi et al. 1996).

From Darcy's law, where there is a flow of pore fluids, there must also exist a corresponding gradient of u_w in the direction of that flow.

Cross-References

- ▶ Aquifer
- ▶ Artesian
- ▶ Boreholes
- ▶ Capillarity
- ▶ Characterization of Soils
- ▶ Clay
- ▶ Darcy's Law
- ▶ Dewatering
- ▶ Effective Stress
- ▶ Equipotential Lines
- ▶ Expansive Soils
- ▶ Fluid Withdrawal
- ▶ Groundwater
- ▶ Groundwater Rebound
- ▶ Hydraulic Action
- ▶ Hydrogeology
- ▶ Hydrology
- ▶ Infiltration
- ▶ Liquefaction
- ▶ Monitoring
- ▶ Percolation
- ▶ Piezometer
- ▶ Pressure
- ▶ Saturation
- ▶ Soil Properties
- ▶ Water

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Pressure

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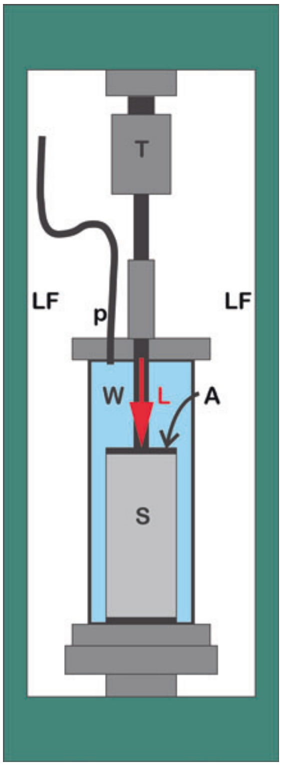
Definition

Pressure is a scalar quantity in fluids and gasses that have no shear strength, such as water or air, and can be used as a scalar or vector quantity with Earth materials. Pressure is force per unit area and expressed in units of Pa. Hydrostatic fluid pressure would be the unit weight of water (mass time acceleration of gravity, 1 kN/m^3) acting over a unit area, or 1 kPa. In geology, for example, consider a tunnel at a depth of 11 m below the groundwater table in silty sand soil material, the pore-water pressure would be equal to the unit weight of water (1 kN/m^3) multiplied by the depth of interest below the groundwater table (11 m), or 11 $kN/m^2 = 11$ kPa.

The weight of rock material acting over a unit area is geostatic pressure, sometimes called lithostatic or overburden pressure. Geostatic pressure actually is a stress acting in a vertical direction, which is a vector quantity. A typical sandstone with a specific gravity of about 2.65 would have a unit weight of about 26 kN/m^3 and exert a unit pressure of about 26 kPa in a vertical direction acting under the influence of gravity. The pressure of a constructed work, such as a building, supported by the ground or Earth material, such as soil or rock, would be the weight of the constructed work divided by the area of the contact that the building has with the earth material. In a laboratory unconfined compression test on a core sample of granite, the axial load acting on the sample cross sectional area that causes it to break is the unconfined compressive strength, which is the maximum pressure the sample can sustain at the moment it breaks.

A triaxial compression test (Fig. 1) is useful for illustrating hydrostatic pressure, which confines the sample, and axial load stress (pressure) in combination (Coduto et al. 2011). The pressurized fluid transmits pressure equally in all directions (principal stress), including to the plate on top of the sample, without generating shear stresses because fluids cannot transmit them. The axial load transmits pressure only to the top of the sample; shear stresses are transmitted to the sample because solids can transmit them. The numerical difference between the stress transmitted axially to the sample and the hydrostatic pressure is called the deviatoric stress.



Pressure, Fig. 1 Schematic diagram of a triaxial compression test setup. A soil or rock sample, S, inside a cell that can be pressurized, p, with fluid, such as de-aired water, W, or oil, is subjected to a load, L, delivered by means of a load frame, LF, with a load cell transducer, T, to a steel plate that distributes it over the area of the sample, A, as a stress or pressure

Cross-References

▶ Deviatoric Stress
▶ Geostatic Stress
▶ Ground Pressure
▶ Lateral Pressure
▶ Pore Pressure
▶ Stress

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Probabilistic Hazard Assessment

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Definition

Evaluation of the severity of a process or event that has the potential to cause damage to property or harm to life and health based on the realization that the severity can range from a level that causes slight damage or harm to a level that causes extensive damage or harm at a particular place.

Synonyms

Annualized frequency of a given severity; Severity for a given average return period

Context

Probabilistic methods were introduced in the late 1960s (Cornell 1968) and expanded in the 1980s to characterize hazards associated with extremely rare earthquakes for design of nuclear power facilities that could have extremely severe consequences if they were damaged (Kemmerer 2013;

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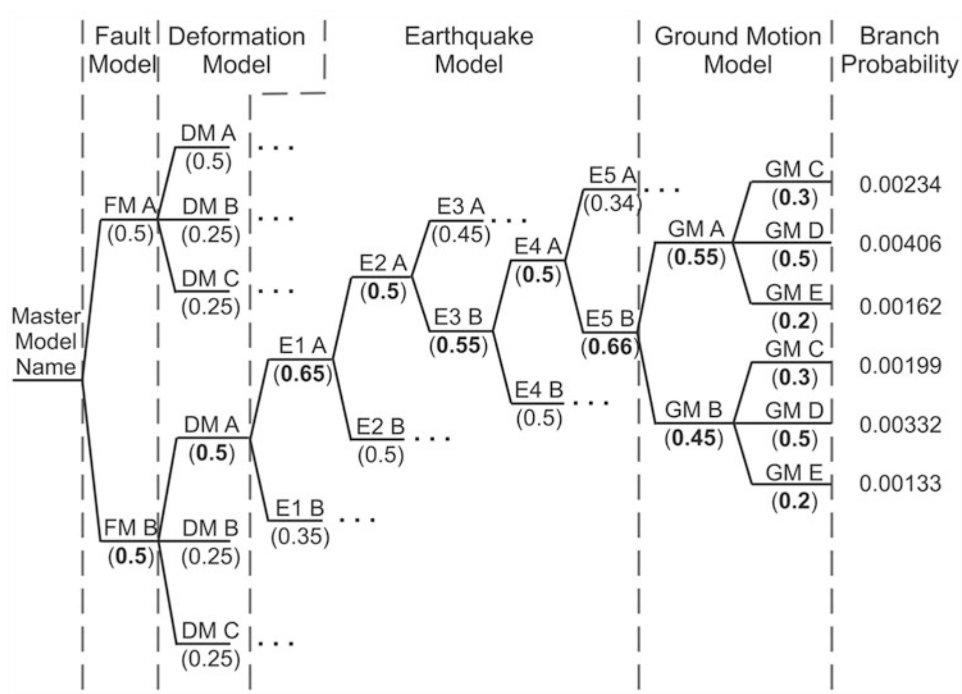
U.S. NRC 2017). Common challenges among natural hazard assessments (Kemmerer 2013) include (1) limited availability of measurements or observations of extremely large events; (2) large uncertainties in return periods of rare events; (3) data can be subject to alternative interpretations, requiring expert judgment; (4) both best estimates and uncertainties are needed; and (5) variability of nature (aleatoric uncertainty) and uncertainties introduced by lack of knowledge or inherent in the model used to assess the hazard (epistemic uncertainty).

Natural hazards can be primary, such as earthquake shaking and volcanic eruption, but most are secondary, such as riverine flooding and landslide movement. The general approach to assessing earthquake shaking recognizes that earthquakes are generated by faults. Faults have different types of displacement, fault displacement may produce earthquakes of different magnitudes and rates, and earthquakes generate ground vibrations that have different characteristics. Probabilistic models of earthquakes use a logic tree approach (Fig. 1) that have multiple nodes and branches, each of which is assigned a weight, such that the sum of the weights of branches at a node equals 1.0; the individual weights are considered in terms of "relative degree of belief" in each value in the model (Kemmerer 2013) such that the product of the weights along a branch is the probability that that branch gives the true result. Ground motion amplitudes are maximum at the epicenter and diminish with distance; this quality is related to attenuation of energy with distance. Models of ground motion attenuation are based on response of sites with certain characteristics, typically hard rock, soft rock, stiff soil, soft soil, and very soft soil. The results of a probabilistic seismic hazard assessment of a particular location typically are expressed in terms of particle acceleration on a standard soft rock site associated with contributions from a suite of earthquakes with magnitude-distance combinations that are considered possible to occur with a specific exceedance probability. If the site being assessed has characteristics that match the standard soft rock site, then the adjustment factor for the site-specific acceleration is 1.0; however, if the site characteristics differ from the standard soft rock site, then the site-specific ground motion associated with the specific exceedance probability is adjusted by a factor <1 or >1 according to a model for that aspect of the earthquake hazard.

Probabilistic assessment of flooding hazard can use stream gauge data and precipitation estimates in the watershed upstream of a particular location near a river or stream channel. Stream gauge data are the measure of water level in a channel at a stable location where the channel cross section has been determined. Ungauged channels require estimates of precipitation across the upstream watershed to be routed to the tributary channels and the main channel and ultimately to the point of interest before the flood level can be estimated. The challenges associated with extreme floods are similar to extreme earthquakes; however, earthquakes are considered to be independent of climate changes, whereas precipitation clearly is a direct result of weather, which is a major component of climate. Flooding associated with a particular return period, notably the 100-year flood, is described by water depths or water surface elevations throughout a watershed or a region. Site-specific aspects of flood hazard are associated with upstream and downstream development since the



Probabilistic Hazard Assessment, Fig. 1 Example logic tree for probabilistic hazard assessment model for earthquake ground motion flood-hazard model results were developed. Upstream changes that affect flood levels are increased impervious surfaces (rooftops and pavement) and flood control measures (dams and reservoirs). Downstream changes that affect flood levels are channel obstructions by buildings or embankments.

Probabilistic assessment of landslide hazards has not developed very much because landslides are secondary hazards that are triggered by primary or other secondary hazards. Assessment of seismically induced landslides benefits from probabilistic seismic hazard assessment results that are widely available. A particular site on sloping ground is characterized for stability using the acceleration associated with a particular return-period ground motion. Beyond the return period of the ground motion, deterministic assumptions tend to be made about the position of groundwater in the slope, which results in a hazard assessment that has some probabilistic component, but is not truly probabilistic. A major challenge with landslides is that a severe storm may trigger widespread landslides that cause substantial damage; however, a subsequent severe storm that is very similar to the one that triggered widespread landslides may result in no movement on any slopes in the same region where substantial damage occurred before. Alternatively, in urban locations, buried utilities leaking water into the subsurface can trigger landslide movement that is independent of rainfall.

More research is needed to support probabilistic methods in assessing natural hazards. Note that the hazard assessment is independent of facilities or populations that might be exposed to damage or harm by the hazard if it occurred. Consideration of facilities and populations is part of risk assessment.

Cross-References

- ▶ Earthquake
- ▶ Floods
- ▶ Hazard Assessment
- ▶ Landslide
- ▶ Probability
- ▶ Risk Assessment
- ▶ Sinkholes

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Probability

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Definition

In simple terms, the likelihood that a specified event will occur.

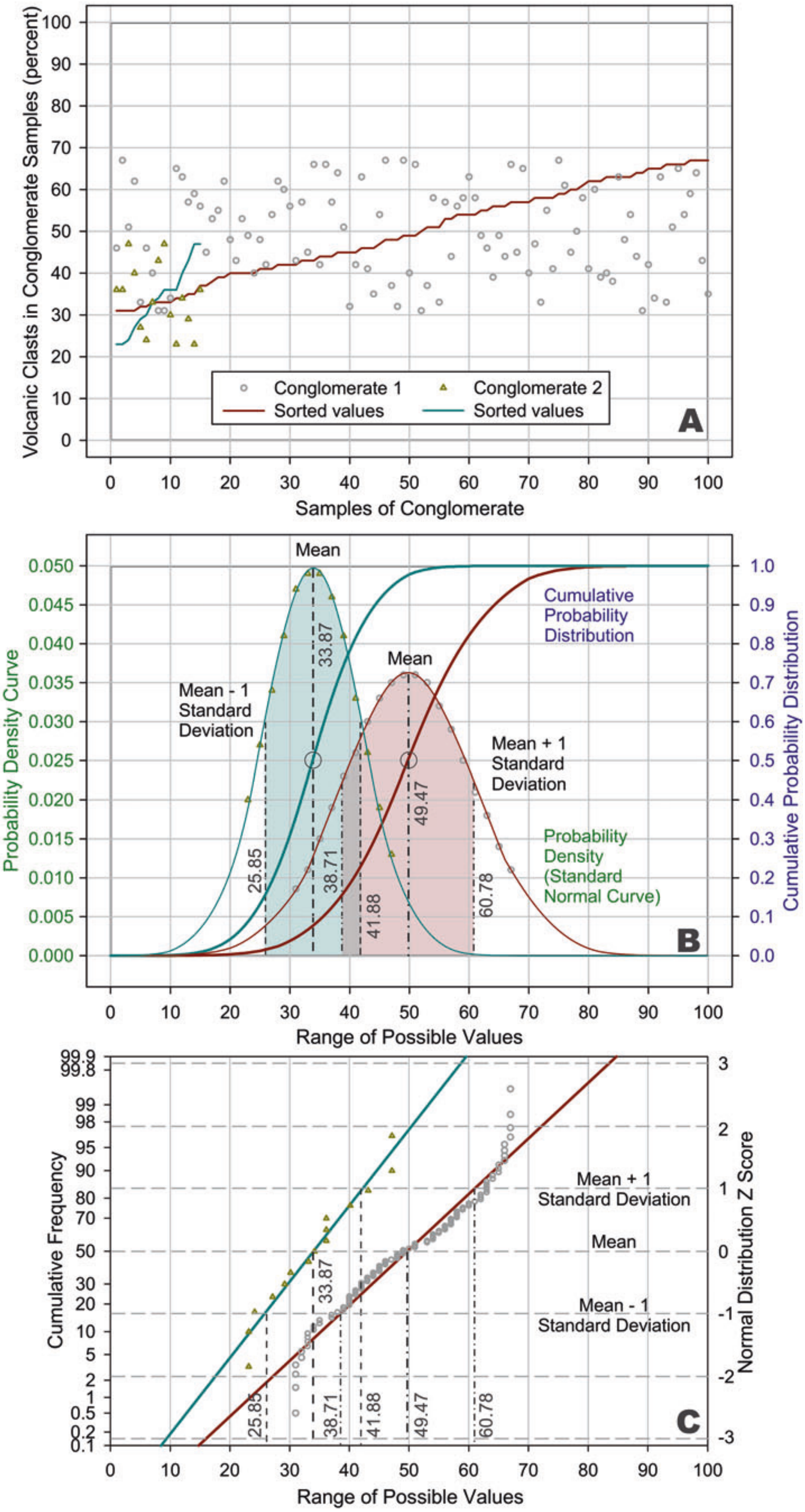
The likelihood can be expressed qualitatively (e.g. a low-to-moderate chance of rain tomorrow) or quantitatively (e.g. a 30% probability of measureable rain in the forecast area on Monday before 3 pm local time). Event likelihood ranges from 0 to 1, meaning that the event is either certain to not occur or certain to occur, respectively. A toss of a fair coin has an equal probability of landing head or tail up. The probability of one outcome, either head or tail, out of two possible outcomes is $1/2 = 0.5$. In cases where the outcome can range continuously from zero to 100, as in an analysis of the percentage of volcanic clasts in a conglomerate formation (Fig. 1), the mean and standard deviation values can be used to define a range of expected values and to infer the probability that conglomerate 2 and conglomerate 1 have the same provenance.

Events of interest in engineering geology tend not to be simple, often involving processes with possible outcomes that range from small to large, such as water flow in rivers. Flows contained by the banks may be of interest to hydrogeologists, but do not threaten property. Flows exceeding channel capacity may threaten adjacent properties and become of greater interest. Such events are characterized by exceedance probabilities, the probability associated with an event that equals or exceeds a certain size, such as the flood that equals or exceeds a 100-year return period, or is less than or equal to an annual frequency of 1/100 year = 0.01/year (USGS 2016). Floods caused by successive storms or by storms in successive years are independent of each other. Statistically, the exceedance probability, p_e, is related to the annual frequency of the exceedance event, λ_e, and the return period of the exceedance event, RP_e, over an exposure time period, t_e, by.

$$\mathrm{P}_e = 1 - \exp(-t_e\,.\,\lambda_e);\, \lambda_e = \frac{-\mathrm{l}_n(1 - P_e)}{\mathrm{t}_e} = \frac{1}{RP_e} \quad (1)$$

Thus, in an exposure period of 100 years, the 100-year return period event would have an exceedance probability of 0.632, demonstrating that the so-called 100-year flood is not expected to occur every 100 years, or even once in every 100-year period. A similar exceedance probability approach is used for earthquake ground motion used in building codes. The earthquake characterization is based on a 50-year design

Probability, Fig. 1 Percentage of volcanic clasts in conglomerate 1 (100 samples) compared to percentage of volcanic clasts in conglomerate 2 (15 samples). (**a**) Plot of values as tested (points) and values sorted from smallest to largest (lines). (**b**) Standard normal probability density and cumulative probability distribution. (**c**) Cumulative frequency (Gaussian distribution plot) on a "probability" scale corresponding to the area under a standard normal curve



life (t_e) and a designated exceedance probability ($p_e = 2\%$), resulting in a 2474.9 years average return period (RP_e; $\lambda_e = 0.000404$/year) for the design ground motion.

The exceedance probability described for flooding is based on random floods and reliable annual frequencies. Many geologic processes have complexities that do not allow them to be characterized by direct observation, such as fall of a rock block from a steep slope which rolls out from the base and stops. Events associated with such processes have uncertainties related to modelling simplifications or lack of knowledge (epistemic uncertainties) and require judgement, in addition to variability in properties that can be measured (aleatoric uncertainties) and dealt with statistically. Probability approaches that involve judgment about state of knowledge or degree of belief are called Bayesian probabilities. Bayes' Theorem is based on prior knowledge of condition B informing the probability of A; the conditional probability of A given that B has occurred [p(A|B)].

Cross-References

▶ Hazard assessment
▶ Risk assessment

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Professional Practice

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Definition

A term that describes the technical/scientific skill set and experience and also the conduct of a person with specialist knowledge who receives remuneration for the work that they perform (i.e., is not an amateur).

For engineering geologists, minimum professional practice standards may be regulated (i.e., set out in legislation or regulation and enforceable by law), or they may be unregulated (set by professional geoscience or engineering organizations with monitoring/enforcement by peers). The principal purpose of setting Professional Practice standards in geoscience, including and particularly engineering geology, is to ensure competence in the *protection of the public and service of society*. The concept of protection of the public encapsulates not only the health and safety of people and their property but also investors who may be impacted if projects fail through poor professional practice (lack of competence or negligence).

Context

Professional engineering geologists can be found in a wide variety of positions in the civil sector ("industry" and consulting), academia, government and regulation, research, environmental protection, and emergency relief, to name a few. The focus of their activities is in establishing a reliable ground model (representing the geology, geomorphology, hydrogeology, and all relevant material classification and parameters) as the basis for engineering design or hazard characterization. They work in a wide range of practice areas that have implications for public safety and well-being (e.g. construction, hazard mapping, natural hazard mitigation, spatial planning, environmental assessment, etc.).

Professional engineering geologists today are held accountable through Codes of Ethics (also known as Codes of Conduct) and the Complaints and Discipline processes of the professional geoscience organizations to which they belong (whether they practice in a regulated or an unregulated jurisdiction). The requirements of these codes provide the underpinning framework for the application processes for registration and/or licensure and for professional titles (e.g., PGeo, CGeol, CEng, CSci, etc.) awarded by professional geoscience bodies. In most jurisdictions, professional practice standards are assessed using five general quality criteria that, together, define the "*competence*" of the practitioner (after Andrews 2014):

- **Application of theory** (analysis, design and synthesis, testing methods, and project implementation)
- **Practical experience** (appreciation of limitations of theory, equipment, systems, procedures and standards in the specialist discipline of the practitioner)
- **Management of projects** (planning, scheduling, budgeting, supervision, project control, and risk assessment)
- **Communication skills** (ability to communicate clearly in writing and orally and in a variety of contexts: formal reports, correspondence, design specifications and standards, contracts, etc.)
- **Social and ethical implications of projects** (understanding of the necessity to protect life, health, property and the environment, and that these obligations override any contractual duty to an employer or client)

Where engineering geologists hold a professional title, it signifies that they are judged by their peers to meet or exceed standards that demonstrate "competence" as defined above. It also signifies that they are committed to "lifelong learning" through a formal or informal program of continuing professional development ("CPD"), agree to be bound by a code of ethics/code of conduct, and to be subject to the disciplinary procedures of the organization awarding their professional title. Finally, practicing engineering geologists and/or their employers need to be covered by professional indemnity insurance (PII) so as to be able to defend themselves in the event of a claim being made of professional negligence (whether successful or not). Most clients of consulting engineering geologists require them to prove that they have adequate valid PII before hiring them.

Cross-References

▶ Engineering Geological Maps
▶ Engineering Geology
▶ Engineering Geomorphological Mapping
▶ Engineering Geomorphology
▶ Engineering Properties
▶ Ethics
▶ Geohazards
▶ Hazard
▶ Hazard Mapping
▶ Modelling
▶ Site Investigation

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Q

Quick Clay

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Synonyms

Glaciomarine sediment; Leda clay; Sensitive clay

Definition

Quick clay is a special type of clay prone to sudden strength loss upon disturbance. From a relatively stiff material in the undisturbed condition, an imposed stress can turn such clay into a liquid slurry.

Discussion

Quick clay is defined as a clay where the undisturbed shear strength of the soil is at least 30 times greater than the remoulded (or disturbed) shear strength (Torrance 1983). The ratio of undisturbed to disturbed strength is termed *sensitivity*. Thus a quick clay is very sensitive.

Quick clay is common along previously glaciated coastlines in parts of Canada and Scandinavia, and has also been found in Japan and in Alaska. Coastlines in these areas were submerged by the weight of glaciers during glaciation. As glaciers retreated, seas migrated inland with retreating icefronts. Glacially ground sediments, transported in plumes of glacial meltwater were deposited in the salt water of these ice marginal seas.

In freshwater, clay particles settle more slowly and separate from the larger silt particles. In salt water, clay and silt flocculate and settle together with a random orientation. Negative, repulsive charges on the clay particles are neutralized by Na^+ and Ca^{2+} in sea water. The resulting sediment has an open structure with high water content. The positive charges of the salts maintain the interparticle bonds that allow the open structure to persist.

As the glaciers disappeared the land began to rebound isostatically, rising as much as 300 m above present-day sea level in the Hudson Bay area of Canada. Exposure of the uplifted glaciomarine sediments to rainfall and groundwater leached salt from the porewater. Salt content would decrease from initial concentrations as high as thirty grams per liter in the sea to less than 1 g per liter. With reduced salt content repulsive forces between particles increased, leaving the saturated, porous sediment prone to collapse. Given the right conditions, sediments that underwent this process could become quick clay.

An imposed load, vibration, or bank erosion, can collapse the sedimentary structure in sensitive clay, often causing liquefaction. During liquefaction, the weight of the soil is transferred from the solids to the porewater.

Sensitive and quick clay are hazardous because they can host sudden rapid landslides on extremely low gradients. On 29 April 1978, a destructive landslide occurred near the town of Rissa, Norway (Gregersen 1981). The largest landslide of the century in Norway, it covered 33 ha, involved 5–6 million m^3 of quick clay and caused a displacement wave. The landslide was triggered by a small external load; Earth fill from the excavation of a barn. Seven farms and five houses were destroyed.

On 4 May 1971, 7 million m^3 of quick clay at Saint Jean Vianney, Quebec, Canada, suddenly began to flow at a rate of more than 25 km/h into the Rivière du Petit-Bras carrying

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with it some 40 homes (Tavenas et al. 1971). The crater left by the landslide was 32 ha in area and up to 30 m deep.

Cross-References

- ▶ Clay
- ▶ Collapsible Soils
- ▶ Drilling Hazards
- ▶ Erosion
- ▶ Expansive Soils
- ▶ Hydrocompaction
- ▶ Landslide
- ▶ Land Use
- ▶ Liquefaction
- ▶ Mass Movement
- ▶ Pore Pressure
- ▶ Quicksand
- ▶ Subsidence

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Quicksand

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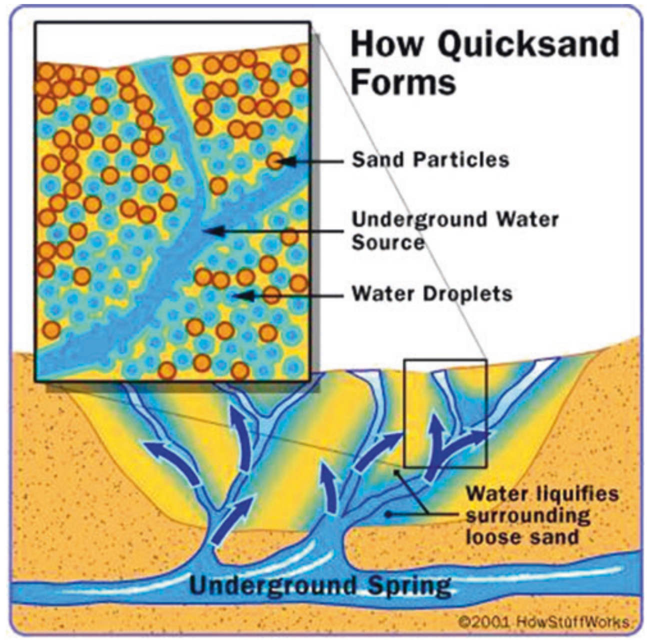
Synonyms

Drift sand; Mire; Quagmire

Definition

Quicksand is sediment of low bearing capacity caused by high volumes of water, or more rarely air, and a resultant decrease in the intergranular pressure due to disturbance. Quicksand is not any special type of sand, but instead it is a condition of zero effective stress.

The condition may occur in sediment with either standing water or flowing water. In the former case, the condition



Quicksand, Fig. 1 Schematic diagram of how quicksand is often formed. Upward flowing water decreases the effective stress holding the sediment together. The sand particles rearrange or collapse

can be triggered by ground shaking (liquefaction). Most quicksand occurs in settings where there are natural springs, either at the base of alluvial fans, along riverbanks, or on beaches at low tide. In such cases, the loose packing is maintained by the upward movement of water (Fig. 1; Long 2002).

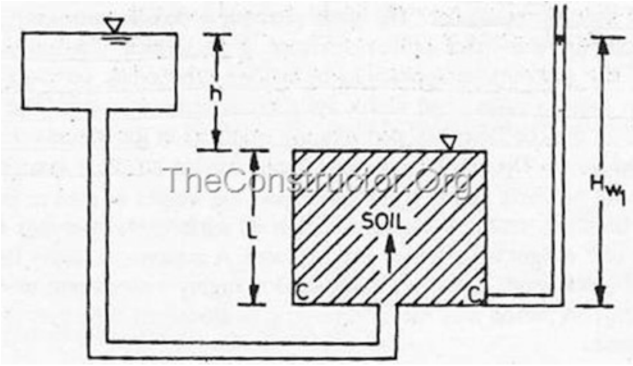
Quicksand can occur in dry sand, where loosely packed sand can occur, such as on the down-wind (lee) side of desert dunes. In such cases, a disturbance causes the air voids to collapse and the amount of sinking is limited to a few centimeters, as the grains become more densely packed (Long 2002).

For quicksand in cohesionless soil and flowing water, the shear strength of the soil depends upon the effective stress (https://theconstructor.org/geotechnical/quick-sand-condition/3455/). The shear strength is given by

$$S = \bar{\sigma} \tan \phi$$

where $\bar{\sigma}$ = effective stress and ϕ = angle of shearing resistance.

The effective stress is reduced due to the upward movement (flow) of water. When the head causing upward flow is increased, a stage is reached where the effective stress is reduced to zero. At this point, the soil loses all strength and collapses. This is also known as the quicksand condition (Fig. 2).



Quicksand, Fig. 2 Diagram of quicksand formation illustrating differential pressure causing flow into a soil element

Effective stress is given by

$$\begin{aligned}\bar{\sigma} &= \sigma - u \\ &= \gamma_{sat}L - \gamma_w H_{w1} \\ &= (\gamma' + \gamma_w)L - \gamma_w(L + h) \\ &= \gamma' L - \gamma_w h\end{aligned}$$

Since $\gamma_w h = \gamma_w \cdot \frac{h}{L} L = \gamma_w i L$

$$\bar{\sigma} = \gamma' L - \gamma_w i L$$

For effective stress to become zero, $\gamma' L = \gamma_w i L$.
Therefore, $i = \frac{\gamma'}{\gamma_w}$.

Substituting the value of submerged unit weight in terms of void ratio,

$$i_c = {}^{c}\frac{G - 1}{1 + e}$$

Taking G = 2.67, and e = 0.67

$$i_c = {}^{c}\frac{2.67 - 1}{1 + 0.67} = 1$$

Thus, the effective stress becomes zero for the soil with above values of G and e and when the hydraulic gradient "i" is unity, that is, the head causing the flow is equal to the length of the specimen.

If the critical gradient is exceeded the soil moves upward and the soil surface appears to be boiling. This quick condition is also known as the boiling condition. During this stage, a violent and visible agitation of particles often occurs. The discharge suddenly increases due to an increase in the coefficient of permeability occurring in the process. If a weight/load is placed on the surface of the soil, it sinks. In this case, the soil behaves as a liquid having no shear strength.

In engineering, the quicksand condition can lead to structural failures. This can occur during earthquake liquefaction. It may also occur if seepage is occurring beneath a structure due to differential pressure on each side. If seepage is high enough, the zero effective stress case discussed elsewhere can be met. To mitigate this condition, the seepage quantities must be reduced. One way to do this is to insert a cut-off wall between both sides of the structure, which increases the seepage path and therefore reduces the seepage (https://www.youtube.com/watch?v=eImtYyuQCZ8).

Whereas the examples are of simple sand and flowing water, quicksand in reality may consist of a fragile sand structure, held together by clay. The clay acts to stop the sand structure from falling apart (https://www.europhysicsnews.org/articles/epn/pdf/2006/04/epn06404.pdf). When a pressure is applied, the cohesion of the clay is not sufficiently solid to support any weight, the structure is compressed, and again the zero effective stress failure criteria are met.

Quicksand cannot support the weight of a person and it behaves like a liquid. However, with a density twice that of water, a person can easily float in the mixture with about one-third of their body out of the quicksand. The potential for death may arise if a person struggles for a long period and lets their head fall into the quicksand (possibly from exhaustion). Dangerous conditions also occur in the event of a rising tide on an intertidal flat, where quicksand commonly occurs.

Cross-References

- ▶ Effective Stress
- ▶ Characterization of Soils
- ▶ Liquefaction
- ▶ Shear Strength
- ▶ Shear Stress

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https://www.youtube.com/watch?v=eImtYyuQCZ8

R

Reduced Stress

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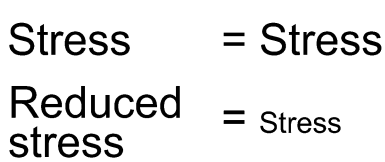
Definition

Reduced stress is very much like stress, only smaller (Fig. 1). Find the term "stress" in the *Encyclopedia of Engineering Geology* (Bobrowsky and Marker 2018). Copy the definition, paste it into a word processing application, and then make the font smaller. Individuals who follow these steps will experience reduced stress.



Reduced Stress, Fig. 1 Reduced stress

Cross-References

► Stress

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Remote Sensing

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Definition

Remote sensing – "The science and art of obtaining information about an object, area, or phenomenon through the analysis of data acquired by a device that is not in contact with the object, area, or phenomenon under investigation," as defined by Lillesand et al. (2015) in their textbook on remote sensing and image interpretation.

Introduction

The first uses of remote sensing in engineering geology practice date back to the late 1920s and early 1930s, when aerial photo interpretation and photogrammetry methods assisted engineers in terrain reconnaissance and site evaluation, flood control surveillance, and topographic mapping (Barr 1984). Since then the use of information retrieved from remotely sensed data by research and professional engineering geologists has become more diversified and more common. However, the application potential of remote sensing in ground engineering is still considered to be little explored in comparison to the uses of remotely sensed data by

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geologists or applied geomorphologists. We foresee an increasingly greater uptake of remotely sensed data by engineering geologists in the near future, because presently, the new-generation high-resolution optical and radar sensors and the improved digital image processing techniques developed in this century are now capable of delivering more rapidly high-quality information that is sufficiently detailed (and cost-effective) for many practical engineering applications.

We focus on the new tools and techniques of Earth surface sensing, which hold the most promise for profitable exploitation in engineering geology research and practice. The emphasis is on selected space and airborne, as well as ground-based, imaging systems, where the most innovation has been taking place since the beginning of this century.

Furthermore, we consider the wide field of engineering geology, ranging from the traditional ground engineering to the multidisciplinary socioeconomic domains (e.g., natural hazards, environmental protection, and sustainable development), in which the applied geologists and geotechnical engineers have become increasingly more involved in the recent decades (Juang et al. 2016). We highlight both the well-recognized as well as currently little-exploited opportunities offered by innovative remote sensing techniques.

For details on remote sensing principles and digital image processing and interpretation, the interested reader is referred to selected textbooks and manuals (Drury 2001; Khorram et al. 2016; Lillesand et al. 2015; Njoku 2014). We also provide references to review articles on specific applications of new remote sensing techniques in engineering geology.

New Remote Sensing Tools and Applications

Very-High-Resolution Optical Satellites

The availability of high-quality imagery provided initially (since the early 2000s) at about 1 m resolution by the first commercial satellites (e.g., IKONOS, QUICKBIRD) can be considered as a major breakthrough in the practical applicability of spaceborne optical sensing to geological engineering. Such resolution meant that the level of detail of information obtained from satellite imagery is comparable to that attainable from high-quality digital aerial photography. The trend toward improving resolutions (sub-0,5 m as of 2015, e.g., WorldView-3) and decreasing prices of the imagery and the growing number of satellite constellations that can offer daily (or even intraday) revisits of the area of interest and rapid data products delivery through web-based access imply the greater and more profitable use of space imagery.

In addition to detailed terrain and site characterization or mapping natural hazards (e.g., floods, landslides), which until recently relied only on aerial photo interpretation, satellite imagery can be uniquely exploited for disaster management and post-event damage assessment (e.g., Bally 2013). One important limitation of the use of satellite optical data in emergency situations (especially flood events) is the presence of persistent cloud cover in certain regions (e.g., tropical regions with long rainy seasons).

Unmanned Aerial Vehicles (UAV)

These inexpensive airborne platforms, also called unmanned aerial systems (UAS), remotely piloted aircraft systems (RPAS), or simply drones, are usually operated by a person on the ground (Barnhart et al. 2012). They can carry sophisticated imaging sensors but most often include light digital cameras used to acquire very-high-resolution (cm-dcm) images. This, as well as the flexibility in survey scheduling, makes UAV technology particularly attractive for rapid response and initial surveys of damaging natural or human-made hazards (e.g., Giordan et al. 2015). With UAV flight endurance on the order of several hours or more, a nearly all-day surveillance capability can be assured for management of evolving hazards.

UAV are typically low-flying platforms and can also acquire imagery even in the presence of low-altitude clouds. However, the presence of strong wind can preclude or restrict their use. The use of UAV is also limited by stringent aviation regulations. In comparison to wide-area coverage typical of satellites, UAV are best fitted to acquire very-high-resolution imagery over smaller areas and are well suited for engineering applications (e.g., Nex and Remondino 2014).

Spaceborne Synthetic Aperture Radar (SAR) Multi-temporal Interferometry (MTI)

MTI refers to a series of advanced synthetic aperture radar differential interferometry (DInSAR) techniques, including Permanent/Persistent Scatterers Interferometry – PSInSAR™/PSI and similar methods – as well as Small Baseline Subset, SBAS, and related/hybrid approaches. Simply stated, with radar satellites periodically revisiting the same area, DInSAR and MTI are used to provide information on distance changes between the onboard radar sensor and targets on the ground (e.g., rock outcrops and bare ground, human-made structures such as buildings, roads, and corner reflectors).

In settings with limited vegetation cover, these techniques can deliver precise (mm-cm resolution), spatially dense information (from hundreds to thousands measurement points/km^2) on slow rate (mm-dcm/year) deformations affecting the ground or engineering structures. Radar satellites guarantee wide-area coverage (thousands km^2); the sensors that actively emit electromagnetic radiation can “see” through the clouds, and the deformation measurements are rarely affected by bad weather conditions. Since 2008 the application potential of MTI has increased thanks to the improved capabilities of the new radar sensors (COSMO-SkyMed constellation and TerraSAR-X) in terms of resolution (from 3 to 1 m) and revisit time (from 11 to 4 days).

Recent literature reviews (e.g., Wasowski and Bovenga 2014a, b) suggest that so far MTI has been mostly used in research-oriented engineering geology investigations, especially those regarding slope and subsidence hazards. However, MTI is also often employed to assist in management of oil/gas field operations (e.g., Ferretti 2014; Singhroy et al. 2015), especially for monitoring ground instabilities induced by the fluid/gas injection and withdrawal. With the steadily growing number of radar satellites, the global coverage and free data availability offered by the recent (2014) European Space Agency Sentinel-1 mission, and continuous improvements of radar data processing methods, MTI is expected to become soon a standard operational tool (like Global Positioning System – GPS) for detecting and monitoring ground deformations and structural distress.

Ground-Based Interferometric SAR (GBInSAR)

As with InSAR or DInSAR, the GBInSAR (also called GBSAR) technology relies on a synthetic aperture radar imaging and exploits the principles of interferometry. In a common operating setup, GBInSAR consists of a radar sensor that moves along a fixed rail (up to 2–3 m long) while sending microwaves toward the target area (e.g., quarry slope) and receiving back the reflected radar signal. Radar images repeatedly acquired in this mode can be used to retrieve very detailed surface morphology of a target area and detect possible deformations. In comparison to MTI techniques, the unique feature of GBInSAR is the capability to provide precise measurements for a wide range of deformation rates (from mm/year to m/hour).

GBInSAR systems achieve millimeter measurement precision and are suitable for local-scale or site-specific monitoring, with up to few kilometer remote surveying range. With its high-frequency (minutes) measurements, day/night and all-weather operational capability, and very rapid processing and delivery of measurement results (within hours), GBInSAR can be exploited for near real-time monitoring and early warning. The equipment, however, is expensive and requires human assistance in the field. Therefore, GBInSAR is most cost-effective for high-risk, short-term (e.g., daily–weekly) monitoring, high-value infrastructure (e.g., dams, bridges), and human activities (e.g., mining).

More information on the principles of ground-based interferometry, data acquisition modes, and processing is available in recent review articles of Monserrat et al. (2014) and Caduff et al. (2015). These works also discuss different examples of ground and structure deformation monitoring via GBInSAR.

LiDAR (Light Detection and Ranging)

Tratt (2014) offers a comprehensive overview of LiDAR technology. LiDAR technique is based on a laser beam scanning which results in spatially “continuous” very-high-resolution imagery (clouds of points) of the ground surface and associated natural and artificial features. A distinction is made between airborne laser scanner (ALS), also called airborne laser swath mapping (ALSM), and terrestrial laser scanner (TLS) applications, as this implies differences in scale (regional or local to site specific) of investigation and in data resolution. ALS and TLS attain, respectively, dcm and cm spatial resolutions and dcm and sub-cm measurement precisions. Importantly, useful results can be obtained even in the presence of dense vegetation.

ALS can be used to generate high-resolution topographic maps and digital elevation models (DEM) for local to large-area investigations; often high-resolution optical imagery is contemporaneously acquired (using digital cameras) during airborne LiDAR surveys. By repeating TLS or ALS surveys, change detection is possible and, e.g., ground surface displacements or soil erosion volume estimates can be obtained (e.g., DeLong et al. 2012).

TLS setup on the ground is relatively easy, but human assistance is also required during the scanning operations. The ALS and TLS instrumentation is expensive. Furthermore, significant costs of airborne surveys tend to preclude the use of ALS for frequent/systematic repetition of measurements.

Summary

New remote sensing technologies can now provide very high spatial resolution imagery for producing detailed topographic maps and DEM. Very-high-precision measurements of ground surface and infrastructure deformations can also be obtained. Spaceborne radar sensors offer great potential for multi-scale (from regional scale to site specific) deformation monitoring because of wide-area coverage and regular schedule with increasing revisit frequency, while maintaining high spatial resolution and millimeter precision of measurement. The high resolutions of the new-generation satellite sensors imply now the possibility to derive very detailed information that fits the requirements of engineers and is relevant to many engineering geology investigations, both in research and practice. For example, remotely sensed data can assist in:

- Terrain mapping (e.g., for lifeline routing)
- Site selection and characterization
- Natural resource mapping and characterization
- Natural hazard (geologic and hydrologic) assessment and monitoring (e.g., subsidence, landslides, ground deformations in general, floods)
- Monitoring human-induced hazards (e.g., landfill deformations, subsidence due to groundwater withdrawal)
- Monitoring engineering structures (e.g., stability of transportation infrastructure, dams)

– Monitoring mining operations (e.g., slope instability issues in opencast mines)
– Monitoring and management of oil/gas field operations (e.g., addressing ground instability issues)
– Engineering structure damage assessment (e.g., building structural damage after an earthquake)

Remote sensing technologies are only starting to gain significant visibility within the engineering geology community. Therefore, a greater opening of the profession to closer multidisciplinary collaborations is needed to fully benefit from the enormous quantities of information the innovative remote sensing can now produce. New collaborations have to be established, particularly with physicists and electronic engineers specializing in advanced image/signal processing and big data management, and geologists with expertise in interpretation of digital remotely sensed data.

Cross-References

► Aerial Photography
► InSAR
► LiDAR
► Photogrammetry

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Reservoirs

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Definition

Reservoirs are typically artificial water bodies that are constructed behind a dam or they are natural water storage entities such as lakes and rivers that are used for different purposes including drinking, irrigation, and industry.

Introduction

Water is recognized as the most important factor for economic and social development in developed and less developed countries around the world. Given that there is an inconsistency between the rainfall seasons and high water demand in arid and semi-arid regions, groundwater resources are the primary water source to satisfy various water demands. Surface water reservoirs are constructed to collect and store water during seasons of high rainfall and river flow when there is relatively lower water uses.

Due to water-related problems around the world and the potential for severe drought conditions in the future, proper design of new water reservoirs as well as the utilization and conservation of current reservoirs are crucial. A dam is defined as an engineered structure constructed across a valley or natural depression to create a water storage reservoir. Such reservoirs are required for three main purposes: (1) provision of a dependable water supply for domestic and/or irrigation use, (2) flood mitigation, and (3) generation of electric power (Best 1998*)*. In addition, a reservoir is also a place where many aquatic and nonaquatic animals exist and thrive (Thornton et al. 1996).

Reservoirs are mainly used to hold a water resource for domestic, industrial, and agricultural use but also to control unexpected floods, so they can be found in those areas that have problems caused either by an excess of water or water scarcity. In arid areas where water resources are limited, reservoirs are used to collect and protect available water for applications such as drinking and agriculture during periods with high water demand. On the other hand, during periods of flooding, reservoirs can play an important role in the control of floodwater and reduction or even prevention of damage in downstream regions (Thornton et al. 1996).

Note that a reservoir system is fed by annual precipitation including rain and snow melt, runoff, and river flow. In contrast, water loss may be caused through evaporation of water, which is a common problem especially in arid areas, and also by percolation through the reservoir bed (Dettinger and Anderson 2015; Takemon 2006).

In providing a water supply, the reservoir is filled during periods of above-average stream flow, thus ensuring a steady supply of water during periods of little or no stream flow. For flood mitigation, the storage reservoir is kept nearly empty during drought and periods of low rainfall, so that, when the flood-generating rainstorms occur, the storage volume available in the reservoir provides a buffer against severe flooding in the river valley downstream of the dam (Best 1998).

The earliest known reservoir was constructed in about 3000 BC for the purposes of irrigation and watering of crops (UNEP 1991; Smith et al. 2006).

Location

Reservoirs can be classified into two types, based on situation. Reservoirs can be located either below the ground surface (like aquifers) or at the surface (such as impounded water behind a dam, natural lakes, wetlands, etc.) according to the regional climatic and geological conditions. Each of these reservoirs has advantages and disadvantages. Most drinking water reservoirs are designed above the ground and may be vulnerable to contamination from chemicals, harmful sediments, human activities, and so on. Also, water loss (evaporation) from open-air reservoirs is considered an important challenge especially in arid and semi-arid areas when they need to be covered to prevent and reduce evaporation.

Nevertheless, open air reservoirs have some benefits, including beautifying the environment, contributing to the growth of plants and animals, boosting the ecosystem, adjusting local temperature, etc.

Underground water reservoirs like aquifers are important water resources for the collection and storage of rainwater for different purposes such as irrigation. Given that they are not directly in contact with atmospheric air, their water losses (evaporation) are much lower than in surface reservoirs. Other positive points of these reservoirs include less water pollution, higher temperature stability, etc. But, difficulty of access is the most important problem of these reservoirs.

Construction

Reservoirs can also be classified according to their method of construction into two categories: natural reservoirs and artificial (man-made) reservoirs. The most important natural reservoirs are seas and lakes, wetlands, and aquifers. Artificial water reservoirs include lakes created behind dams, artificial wetlands, and flood-spreading sites.

Natural reservoirs were more important in the past, but today artificial water reservoirs are more useful for humans in order to store and use water. Some reservoirs are used to supply electricity, but, such reservoirs must also be of a sufficiently high quality for use in agriculture, industry, and even drinking (the need for a treatment plant to operate such reservoirs is a prerequisite). The most important problems of artificial reservoirs are their gradual salinization, as well as sediment buildup, and the need for dredging.

Geological Issues

The most important factors are foundation conditions and the porosity of construction materials. Therefore, selecting the best method for reservoir design as well as estimating construction costs should be based on a holistic view of the geological knowledge of the prospective dam site and its environs, including the nature and distribution of the various rock types in the area, the weathering profile, and details of the structural geology. The required information can be obtained through the site investigation programs by using various data-gathering techniques, including outcrop mapping, bulldozed trenching to expose bedrock below overburden, diamond core drilling, water pressure testing, geophysical surveys, joint surveys, and laboratory testing of rock samples. By integrating this geological and physical information, a geo-mechanical site model is then formed, which provides the engineers with a realistic and quantitative knowledge on which the reservoir and its associated structures will be designed. Collecting the relevant geological information and presenting them in an applicable and useful form for the engineers are the main functions of the engineering geologist (Rezaei et al. 2017).

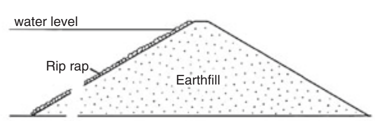
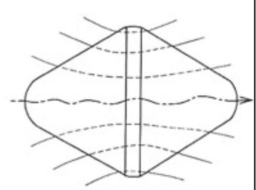
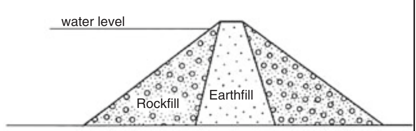
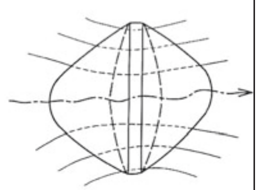
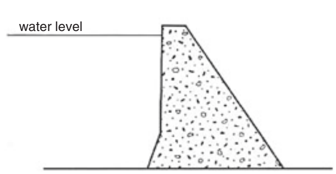
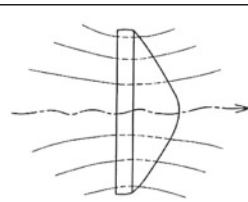
Two criteria must be satisfied during the design of large dams: (1) they should be reasonably watertight, and (2) they should be stable. Such dams are constructed of impermeable materials (e.g., concrete) or impermeable membranes (e.g., an Earth core) are incorporated in their structures to achieve the first criteria. Moreover, the dams' foundations must be made watertight by using grouting or other means. To achieve the second criteria, the movement and deformation of the dams and their foundations cannot be ignored and must be considered through the design procedure.

Types of Dams

There are several basic types of dams that can be selected by engineers during the designing phase for a particular location. Figs. 1 and 2 show the summary of the layout and characteristics of these basic types of dams. At some dam sites, the most economical design has been a composite of two or more basic dam types. One particular type of composite concrete dam is the multiple arch design, which consists of several cylindrical arches supported by buttresses. This type is well suited to sites with geologically variable foundations. The buttresses are located on strong zones of the foundations, whereas the arches are located to bridge weak parts in the foundations.

Reservoir Foundations

According to the dam types shown in Figs. 1 and 2, there is a progressive decline in the foundation area for a given dam height between the Earth dam (largest area) to the double curvature arch dam (smallest area). This also means that the bearing pressure which must be supported by the foundations progressively increases from a minimum for the Earth dam to a maximum for the arch dam (Best 1998). Thus, the sequence of dam types from (1) to (6) in Figs. 1 and 2 requires progressively stronger foundations. It follows that foundation geology at a proposed dam site is an important factor in deciding the most economic type of dam for the site.

Type	Cross section	Plan	Main characteristics
Earth Dam			Has gentle slopes, and hence a large volume and foundation area. Made of compacted earth.
Earth Cored Rock fill Dam			Impermeable earth core, supported by outer zones of compacted broken rock. Steeper slopes than an earth dam. Similar effect may be achieved by an impermeable membrane of concrete, bitumen, steel, or other materials at or near the upstream face.
Concrete Gravity Dam			Water held back by the weight of the structure (hence the name) Construction material (concrete) easier for engineers to control than earth and rock

Reservoirs, Fig. 1 Basic type of dam designs (Modified after Best 1981)

Construction Materials

Reservoirs are constructed from large volumes of naturally occurring Earth materials – broken rock for rockfill and concrete aggregate, sand, gravel, and slopewash or highly weathered regolith for the earth core. Such construction materials must be available near the dam site in order to reduce construction costs (Best 1981). Therefore, the location and cost of construction materials and their extraction are other important factors in determining the type of dam. For instance, a site with highly weathered bedrock is likely to be only suitable for the foundations of an Earth dam. If the overburden close to the site is thin and suitable bedrock occurs at depth, it may well be more economical to excavate the foundations to a depth suitable for a concrete dam than to transport earth material over a long distance to the site.

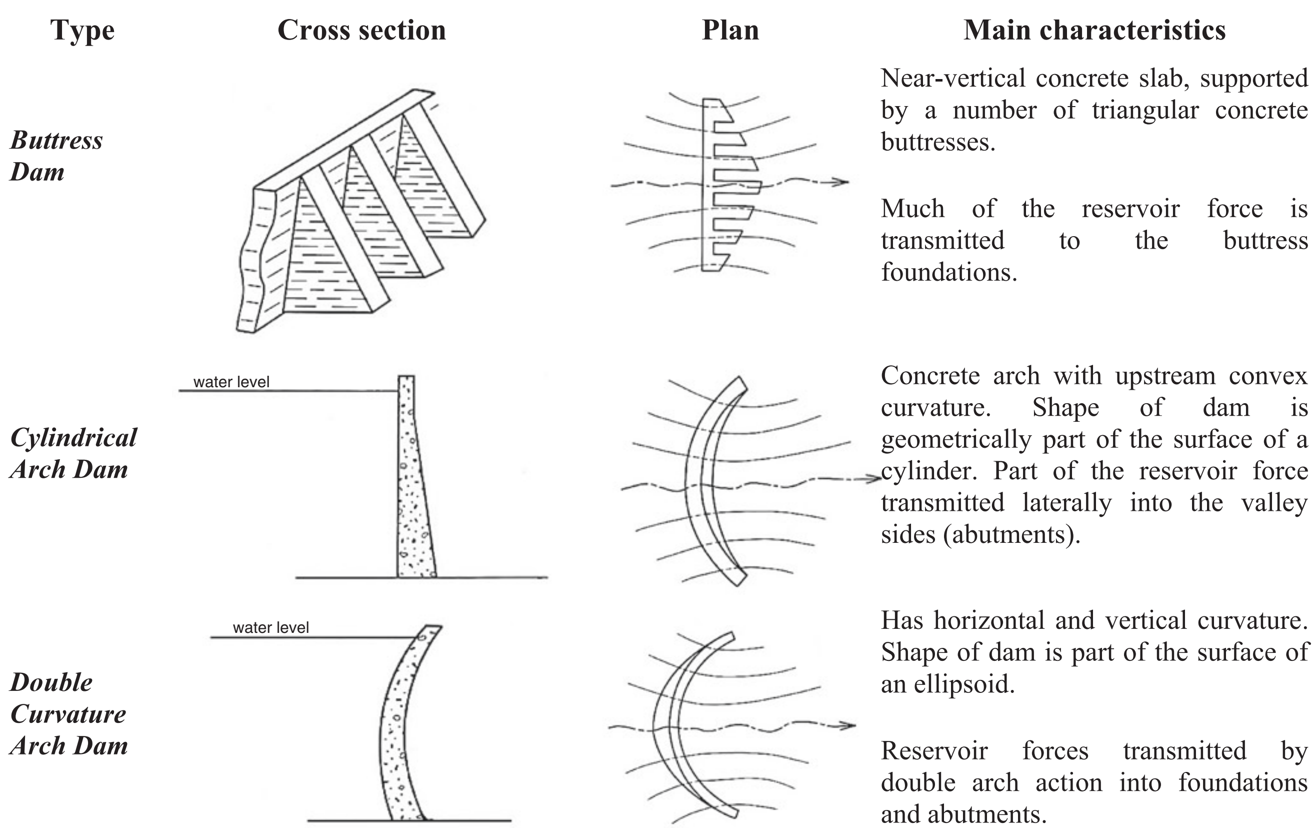
Choice of Dam Type

During the design stage, several types of dam are considered and quantities of materials, cost of materials, amount of foundation excavation, type and amount of foundation treatment, and so on are estimated for each type. The final decision for dam type is based on a cost-benefit analysis to design the lowest estimated construction cost and the safest standards. In general, there are three factors which control this final decision: (1) topography of the dam site and reservoir area; (2) strength and variability of the foundations; and (3) availability and suitability of construction materials. These factors are largely controlled by the geological structure and history of the site. The final decision needs considerable geological data analysis and interpretation, particularly for factors (2) and (3), presented in a manner which the engineer can use in design calculations (Rezaei et al. 2017).

Water Quality

The climate characteristics, the quantity and quality of water inflow to the reservoirs, as well as the evaporation rate in the reservoir surface are the most important parameters affecting water quality, including physical, chemical, and biological issues in reservoirs.

Because, lakes and water reservoir dams are considered as important sources of drinking water supply, agriculture, and industry for human societies, the optimal use of these resources requires proper water quality according to factors



Reservoirs, Fig. 2 Basic type of dam designs (Modified after Best 1981) such as nitrate, nitrite, dissolved oxygen, electrical conductivity, and pH. Salinity and concentration of inlet sediment are also important aspects to evaluate water quality in the reservoirs, especially for drinking and irrigation uses. Thus, it is important to know the details of water quality changes in the dam which must be achieved before any corrective action and operation.

Reservoir Management

As a result of population growth, socioeconomic development, coupled with occurrence of severe drought, there are widespread serious problems facing water security especially in reservoirs.

One of the main problems, especially in arid areas, is high values of evaporation from water bodies, increasing the concentration of salt and decreasing the quality of water. In many cases, the reduction of evaporation is much cheaper than collecting and storing the same amount of water from other sources.

Sedimentation is always one of the main challenges for dam operation. Various methods are proposed to predict the sedimentation and reduction of sediment deposited in dams (Piri et al. 2011). It reduces the effective storage volume of the reservoir, adds to a decline in dam stability, as well as disruptions in functioning of the lower valves.

Important parameters of reservoir water quality are salinity, the amount of sediment entering the reservoir (turbidity), and chemical contamination that confronts the use and exploitation of the reservoir with many difficulties and limitations.

Summary

Reservoirs are typically artificial water bodies that are constructed behind a dam or they are natural water storage entities such as lakes and rivers that are used for different purposes including drinking, water irrigation, and industry. It is necessary to manage extraction of water (from lakes and rivers) and apply effective strategies to optimal operation of artificial reservoirs. Notably, salinity and concentration of inlet sediment are important factors to evaluate in relation to water quality in reservoirs, so, the parameters relating to salinity of the reservoir and the increase of sediment inputs to these resources should be constantly addressed. Dam site selection is an extremely important issue in terms of dam safety and environmental impact. A detailed knowledge of the geology of the dam site and the future reservoir, as well as its catchment area, is necessary before the dam site is selected; acquiring such knowledge is vital in the siting, design, and construction (Best 1981).

Cross-References

▶ Catchment
▶ Climate Change
▶ Dams
▶ Desert Environments
▶ Desiccation
▶ Erosion
▶ Fluid Withdrawal
▶ Groundwater
▶ Hydraulic Action
▶ Hydrogeology
▶ Hydrology
▶ Infiltration
▶ Instrumentation
▶ Lacustrine Deposits
▶ Land Use
▶ Levees
▶ Pollution
▶ Pressure
▶ Risk Assessment
▶ Sabkha
▶ Saline Soils
▶ Saturation
▶ Sequence Stratigraphy
▶ Site Investigation
▶ Soil Field Tests
▶ Soil Mechanics
▶ Strain
▶ Strength
▶ Stress
▶ Tailings
▶ Water
▶ Water Testing

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Residual Soils

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Synonyms

Alteration products; Lateritic soils; Saprolites; Weathering products

Definition

Residual soil is the material resulting from the *in situ* weathering of the parent rock.

Residual soils are distributed throughout many regions of the world, such as Africa, South Asia, Australia, Southeastern North America, Central and South America, and considerable regions of Europe. The largest areas and thickness of these soils occur normally in humid tropical regions, such as Brazil, Nigeria, South India, Singapore, and the Philippines.

Characteristics

According to Duarte (2002), the diversity exhibited by residual soils is due, not so much to the lithology of the original rock, but mainly to external factors such as climate, topography, and vegetation cover; factors that provide distinctive weathering processes and, consequently, distinctive weathering products – the residual soils. At the first International Conference on Tropical Residual Soils, it was proposed to divide such soils into two classes (Brand and Phillipson 1985): (i) Lateritic soils are those that belong to a higher level, well drained and leached, in which the predominant clay belongs to the kaolinite group and contain hydrated iron oxides that give them a reddish color. Generally these do not include primary minerals, and the structure of the parent rock has been totally destroyed. (ii) Saprolite or saprolitic soils, sometimes referred to as young soil, are the residual soils that maintain relic structures from the parent rock, which generally are situated in the levels directly above the original rock, usually contain small amounts of clay minerals, and include primary minerals.

Lateritic residual soils predominate in tropical regions, within latitudes 30° N and 30° S, whereas saprolitic soils are common in temperate regions, for instance, in Portugal, France, Turkey, Piedmont (eastern USA), or in subtropical regions (e.g., Hong Kong and South Africa). The formation of saprolites, which is essentially related to granular rocks, includes primary and secondary minerals in its silt-clay fraction, the nature and quantity of which depends upon parent rock characteristics and on degree of weathering achieved.

The specific characteristics of residual soils in contrast to those of transported soils, are generally attributed either to the presence of clay minerals specific to residual soils (physical composition and mineralogical composition), or to particular structural characteristics of soil in its undisturbed *in situ* state, such as: (i) Macrostructure: includes the presence of unweathered or partially weathered rock, and relic discontinuities or other weakness planes and structures inherited from the original rock mass; Microstructure – includes rock fabric, interparticle bonds or cementation, particle aggregates, dimension and shape of micropores (Vaughan 1988; Duarte 2002; Wesley 2010). These specific characteristics influence the geotechnical behavior *in situ*, thus permeability is governed by the micro and macro-structure, as well as the strength and deformability of the residual soil masses (Townsend 1985; Blight 1997).

According to Gomes (1988), the clay of residual soils formed in temperate climates are intermediate, sharing characteristics both of soils from cold or desert climates, where physical weathering prevails, through the disintegration (mechanical breakdown) of phyllosilicates (mica and chlorite) from the parent rock, and those of tropical climates, where chemical weathering prevails, producing kaolinite, gibbsite or smectite, depending upon local conditions. In regions of temperate climate, soils can be derived from either mechanical weathering or chemical weathering. These soils show little evolution, since precipitation and temperature facilitate the moderate hydrolysis of silicates. In the weathering profiles, both neoformed and transformed clay minerals may be present (Fig. 1).

Cross-References

▶ Alteration
▶ Biological Weathering
▶ Chemical Weathering
▶ Classification of Rocks
▶ Classification of Soils
▶ Collapsible Soils

Residual Soils, Fig. 1 Weathering profile of a granitic massif in southern Portugal, under temperate climate, with a saprolitic residual soil cover of about 10 m thick (Photo by I. Duarte)

▶ Landslide
▶ Physical Weathering
▶ Sediments
▶ Soil Mechanics
▶ Soil Properties

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Restoration

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Synonyms

Rehabilitation; Restoration

Definitions

1. "…the combined process of land treatment that minimizes water degradation, air pollution, damage to aquatic or wildlife habitat, flooding, erosion, and other adverse effects from surface mining operations…so that mined lands are reclaimed to a useable condition which is readily adaptable for alternative land uses and create no danger to public health or safety" (OMR 2015).
2. To make "........land capable of more intensive use by changing its general character, as by drainage of excessively wet land; irrigation of arid or semiarid land; or recovery of submerged land from seas, lakes and rivers" (EEA 2015).

Restoration, Fig. 1 Reclamation at Wishon Quarry in the foothills of the central Sierra Nevada, California (USA) involved filling excavated shallow pit following removal of rock for facing an earthen dam. Stockpiled soil is being applied to a refilled section of the pit. (Photo by J. De Graff)

Restoration, Fig. 2 Kinderdijk, a UNESCO World Heritage site between Rotterdam and Dordrecht, Netherlands, preserves windmills, pumping stations, low and high storage basins ("boezems"), dikes, ditches, and sluices which have kept the low-lying peat land of the Alblasserwaard dry since 1758. This polder landscape illustrates land reclamation through water management over a nearly 1000-year period (Photo by J. De Graff)

Characteristics

Mining is one of the primary human activities responsible for disturbing land to an extent that reclamation is necessary. Disturbance is due to the extraction of near-surface deposits of metallic and nonmetallic mineral resources or from activities incidental to underground mining such as ore storage, ore processing, and stockpiling of tailings and waste rock (Fig. 1).

Reclamation is needed for abandoned or inactive mined areas and for those areas where mining is actively being undertaken. Re-establishing natural drainage patterns, preventing accelerated erosion, especially slope instability, and promoting desirable vegetation growth are all important aspects in reclaiming abandoned and inactive mined area (Newton and Claassen 2003). An especially important component of reclamation for surface disturbance at underground mines is ensuring effective closure of hazardous mine openings. The need for building materials including sand, gravel, and crushed rock near expanding urban areas is typically satisfied by nearby active surface mining operations. Whereas reclamation of rock quarries can be difficult, sand and gravel can be infilled with soil generated from pit development or restored as wetland areas. This is an important aspect of local land use planning to ensure access to needed aggregate resources and subsequent utilization of the mined areas (Arbogast et al. 2000).

Altering the natural landscape to increase its suitability for human activities is a form of reclamation with a long history. Arguably, one of the most extensive reclamation efforts is the dike and polder system in The Netherlands (Fig. 2). From the twelfth century to present, the Dutch have created extensive areas of arable land while providing flood control along rivers and the shoreline of the North Sea. Urban locations along coast such as Rio de Janeiro and Cape Town and islands like Singapore and Hong Kong have commonly modified their nearshore environments to accommodate additional buildings, port facilities, airport runways, or other amenities. Boston provides a good example of how coastal artificial fills placed without regard to material properties and subsurface conditions can even create unintended hazards, for example, liquefaction potential (Brankman and Baise 2008). Natural landscape reclamation can require engineering geologic information ranging from general aspects such as site subsurface conditions or characteristics of geologic materials present or being used in construction work to more specific ones such as the stability of an engineered slopes or seepage conditions within an embankment.

Cross-References

► Land Use
► Liquefaction
► Mining
► Tailings

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Retaining Structures

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Amec Foster Wheeler, Los Angeles, CA, USA

Synonyms

Retaining walls

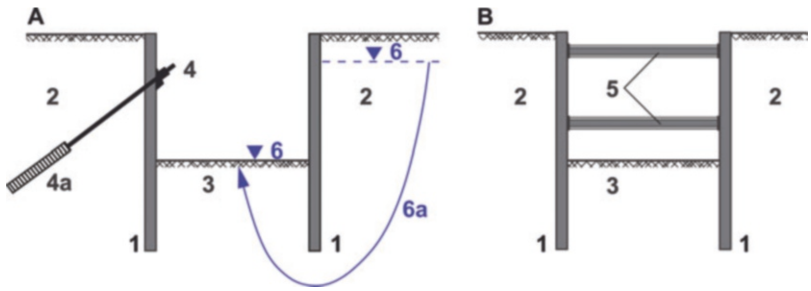
Definition

Retaining structures are walls, dams, barriers, or bins that hold Earth materials or water in place or keep Earth materials or water from encroaching into an area. Retaining structures also are used to create stable surfaces for building pads, roads, bridge abutments, or wharves. Retaining structures can be used to limit the volume of excavations or to allow utilization of space near the boundary of a particular piece of land. Other structures that appear to be earth-retaining structures may have erosion protection as their primary purpose.

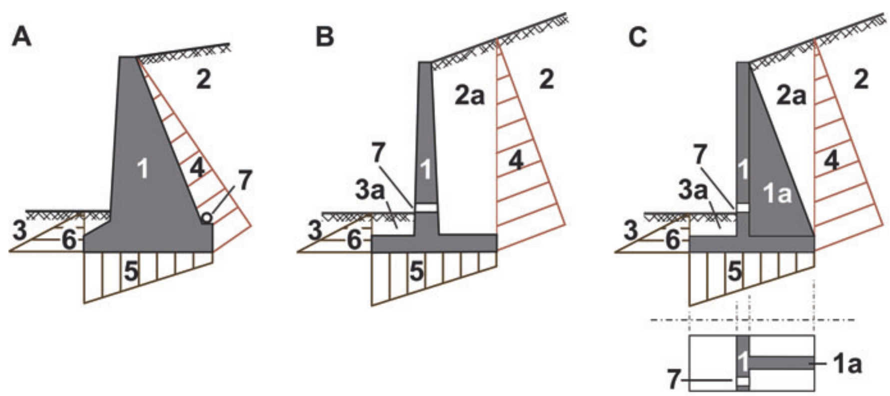
Introduction

Retaining structures commonly are engineered features that are designed and constructed to hold soil or water in place. Structures that retain water are called dams, levees, or flood walls; structures that retain Earth are called earth-retaining structures or retaining walls, which are described here. Retaining structures can be installed at a site prior to excavation as structures that become retaining walls as excavation progresses (Fig. 1) or they can be constructed on sloping ground or to provide a terraced configuration with soil backfill placed behind the walls (Fig. 2). Descriptions and design guidance is provided by NAVFAC (1986), USACE (1989), and USACE (1994).

Stability of retaining walls is provided by simple mass, mass created by mechanically stabilized Earth systems, cantilevered overturning resistance, anchored lateral



Retaining Structures, Fig. 1 Schematic diagram showing elements of retaining structures for stabilizing excavations. (**a**) Sheet pile or soldier pile retaining structure; (**b**) Braced retaining structure. Notations: *1* Retaining element (sheet pile for soft ground, driven or drilled H-pile with lagging placed as excavation progresses); *2* Retained soil. *3* Soil below excavated bottom. *4* Tieback element. *4a* Anchor on tieback. *5* Bracing struts or elements. *6* Groundwater level in retained soil and at bottom of excavation (sump pump would be required to remove water from excavation). *6a* Hypothetical groundwater flow line



Retaining Structures, Fig. 2 Schematic diagram showing elements of earth-retaining walls. (**a**) Concrete gravity retaining wall; (**b**) Concrete cantilevered retaining wall; (**c**) Concrete counterfort retaining wall. Notations: *1* Primary concrete element of wall. *1a* Counterfort element. *2* Retained soil backfill. *2a* Part of retained soil backfill directly over wall foundation element that contributes to the mass of the wall system. *3* Soil backfill or native soil that contributes to sliding resistance of the wall system. *3a* Part of the soil backfill directly over the wall foundation element that contributes to the mass of the wall system. *4* Representation of geostatic stress that contributes to "active" Earth pressure on the wall system. *5* Representation of bearing capacity that resists overturning tendency of retained earth. *6* Representation of "passive" earth pressure that resists sliding tendency of retained Earth. *7* Drainage pipe or conduit to limit the hydrostatic stress that can occur behind the retaining wall

resistance, and braced lateral resistance. Examples of a variety of earth retaining systems are illustrated in Fig. 3. Gravity walls (Fig. 2a) rely on the mass of stable material to resist sliding and overturning. The mass of material can be stacked stones (Fig. 3a), mortared stones (Fig. 3b), stacked sacks of soil-cement mixtures (Fig. 3c) or sacks of pre-mixed concrete (Fig. 3d); interlocking concrete elements filled with soil (crib wall; Fig. 3e); or steel, concrete, or synthetic material cells filled with soil (bin wall; Fig. 3f); gabion baskets filled with durable rock fragments (Fig. 3g, h, i, and j); and stabilizing layers of welded wire or high-density polyethylene (HDPE) that creates mechanically stabilized Earth systems (Fig. 3g, h, and i). Retaining walls also can be cast-in-place reinforced concrete with decorative rock finishing (Fig. 3k) or soldier-pile and lagging systems with tiebacks and bracing elements (Fig. 3l). The stabilizing mass in a gravity retaining wall is designed to resist lateral Earth pressures, including the hydrostatic effects of groundwater and transient impulse effects of earthquake shaking. Geosynthetic filter fabric commonly is used to prevent migration of soil particles from the subgrade into pore space in gabion baskets or to prevent migration of soil particles from crib walls or bin walls into the subgrade, depending upon the grain size distributions. The examples of welded-wire walls, welded-wire steepened slope, and gabion baskets (Fig. 3g, h, i, and l) are discussed in Keaton et al. (2011).

Sheet pile and soldier pile walls (Fig. 1a) rely on the stiffness of the structural wall element to resist lateral Earth pressures and hydrostatic pressures, as well as effects of earthquake shaking. Soldier pile walls typically are constructed with steel H-beams spaced 1–3 m apart that are driven into the ground vertically, or placed into drilled holes that are then backfilled with concrete. The H-configuration is controlled so that the open ends are aligned to permit placement of timber elements, called lagging, into the slot created by the aligned H-piles that retains the soil. After the H-piles

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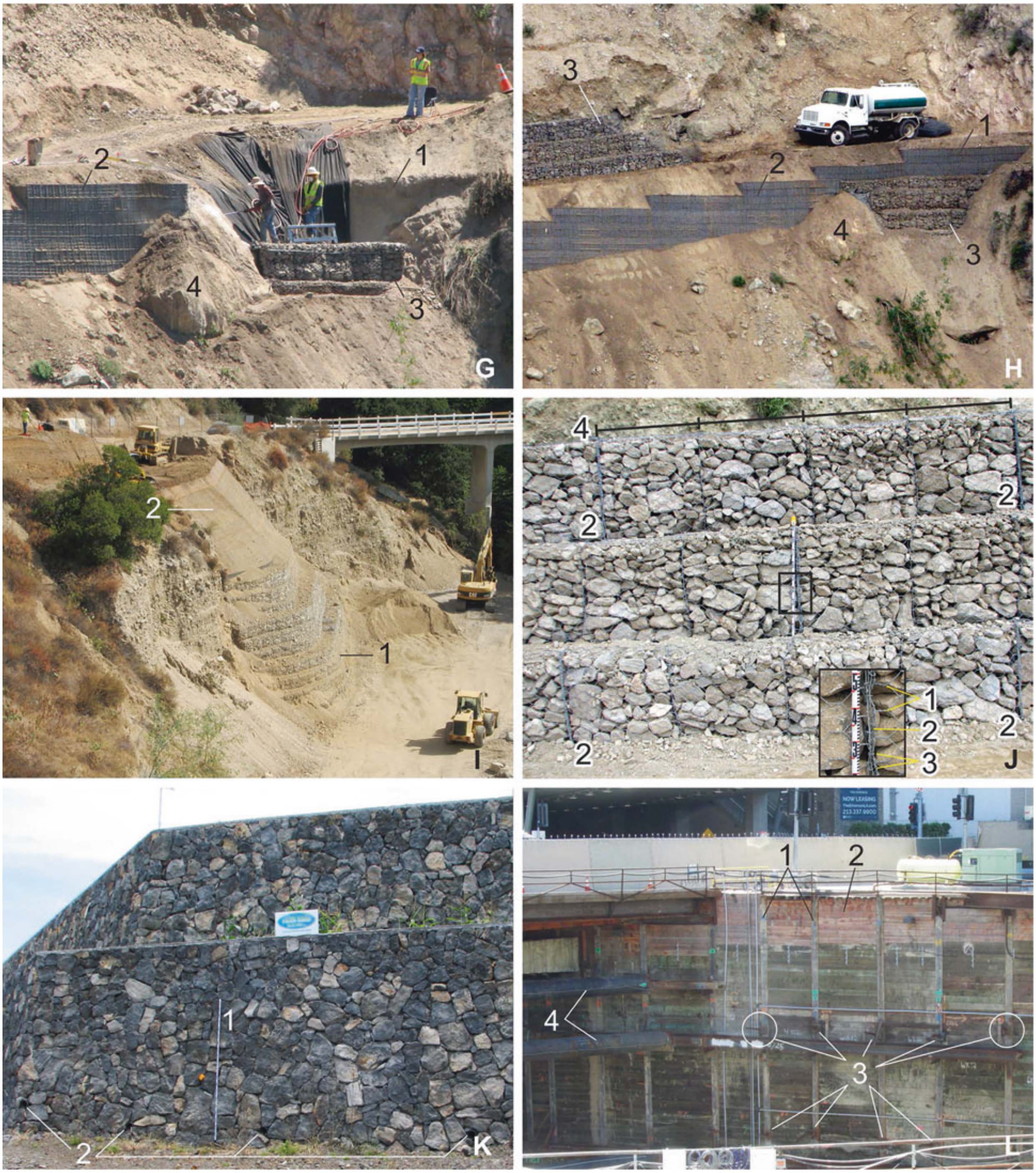
are in place, an excavation is advanced incrementally and lagging timbers placed to retain the soil. For shallow excavations in soft soils, relatively short-cantilevered retaining walls may be made of steel sheet piles. Anchored retaining walls are similar to cantilevered walls with anchor elements supplementing the stiffness of the structural wall elements. Some retaining systems can use soil nails that stabilize the soil mass with increased shear resistance rather than lateral resistance provided by anchors for structural wall elements. Soil nail systems may have surface elements or coatings, such as shotcrete, for erosion control.

Mechanically stabilized Earth (MSE) retaining walls use reinforcing elements in soil backfill to create a mass of stable soil that acts partly as a gravity wall and partly like a soil nail wall. Reinforcing elements can be strips or grids of galvanized or coated steel, or high-density polyethylene (HDPE)

Retaining Structures, Fig. 3 (continued)



Retaining Structures, Fig. 3 Photographs of a variety of earth-retaining walls. (**a**) Hand-placed dry-stacked stone wall with two rows of fired bricks at the top. (**b**) Hand-placed stone-and-mortar wall. (**c**) Hand-placed sacks of soil-cement mixtures; original sacks probably were burlap fabric that has rotted away over several decades of exposure (concrete feature is bridge abutment placed in 1945). (**d**) Hand-placed paper sacks of commercially available dry pre-mixed concrete. (**e**) Concrete crib walls forming an inside corner adjacent to an unsurfaced road. (**f**) Galvanized steel bin wall. (**g**) and (**h**) Retaining systems composed of an old cast-in-place concrete (*1*), new welded-wire wall elements (*2*), and new gabion baskets (*3*), to enable restoration of vehicle access on an unsurfaced road across a landslide; complications were caused by the presence of a large block of rock (*4*) which was left in place. (**i**) Gabion-basket wall (*1*) topped by welded-wire steepened-slope elements (*2*). (**j**) Detail of gabion basket wall visible in panel **h**; notations: (*1*) hexagonal double-twist galvanized wire basket; (*2*) separation between gabion baskets; (*3*) pneumatically secured wire fasteners that hold baskets together into an integrated wall; (*4*) line defining a single, four-compartment gabion basket that is 91 mm high, 91 mm deep, and 4 × 91 mm long (3 ft × 3 ft × 12 ft in U.S customary units). (**k**) Cast-in-place concrete wall with hand-placed decorative stone facing; 2 m long ruler (*1*), drain hole outlets (*2*). (**l**) Soldier-pile and lagging retaining wall; steel H-beam soldier piles (*1*); timber lagging (*2*); tieback elements (*3*), and pipe struts used as corner bracing elements (*4*) (All photographs by Jeffrey Keaton) grid elements. MSE walls require facing elements that can be precast concrete panels, wire mesh, or ultra violet-resistant HDPE grid elements. Geosynthetic filter fabric is needed behind mesh and grid facings to prevent soil particles from moving through the facing.

Cross-References

- ► Bearing Capacity
- ► Cofferdam
- ► Foundations
- ► Gabions
- ► Geostatic Stress
- ► Geotechnical Engineering
- ► Geotextiles
- ► Groundwater
- ► Lateral Pressure
- ► Pore Pressure
- ► Soil Nails

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Risk Assessment

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Definition

Risk assessment is a fundamental step in the management and reduction of risks. Risk assessments require inputs from experts in different hazard-related fields, and in the case of risks associated to geologic hazards, will involve engineering geologists in this process. Risk assessments also involve assessing vulnerabilities and finally the potential losses that may occur, as well as their associated likelihood. The risk assessment process therefore integrates multidisciplinary efforts, aiming to produce a result that is useful for decision-making on how to manage the risk.

Risk, Hazard, and Vulnerability

Risk can be broadly defined as the possibility of the potential loss of something of value. Assessing the risk involves identifying, describing, and, when possible, measuring the potential for such loss. The loss could be of human lives, public or private property, and other less tangible societal or natural assets. The potential loss is caused by a hazard phenomenon or event. In the context of engineering geology, hazards are related to particular Earth processes, like earthquakes, volcanic activity, landslides, etc. Assessing the potential loss also requires knowledge of the entities (people, communities, etc.) that may suffer the loss, that is, the vulnerable system or elements. The vulnerability encompasses the characteristics and conditions that may contribute to an increased risk and, therefore, potential losses. Many formal definitions and conceptions of risk have been proposed by a variety of authors involving the concepts of hazard and vulnerability and other variables (Wisner et al. 2012). Often the definition of risk is presented in the form of an equation. A general form of the risk equation is:

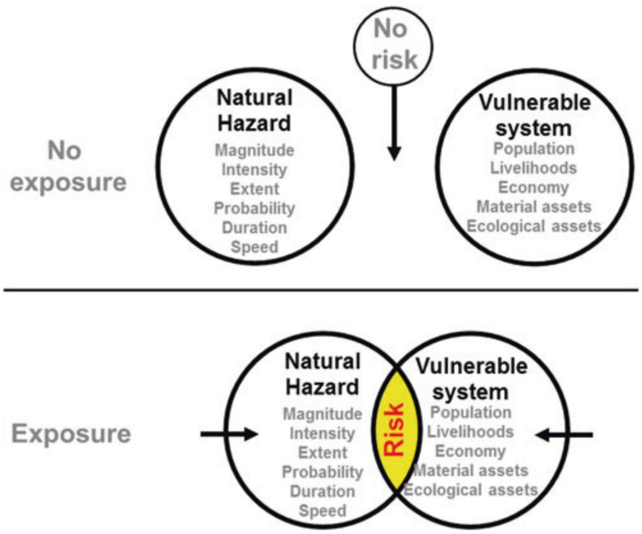
$$\text{Risk} = f(\text{Hazard, Vulnerability, other variables}) \quad (1)$$

The functional type for Eq. 1 can adopt many forms, but it is often defined as a product, as:

$$\text{Risk} = H \; x \; V \quad (2)$$

In this definition, the H and V variables are usually assumed to be positive numbers measuring the intensity, probability, severity, or some other aspect of the hazard and vulnerability, respectively. The central idea behind this definition is to show that the risk increases with both hazard and vulnerability, but if one of the variables (H or V) decreases or becomes zero, the risk will also decrease or become zero, even if the other variable does not change. What this shows is that risk can be reduced (or increased) by either reducing (or increasing) the hazard, the vulnerability, or both.

Graphically, the concept of risk can also be illustrated as shown in Fig. 1 (Wood 2011). Risk only exists when vulnerability (or a vulnerable entity) intersects with (i.e., is exposed to) a hazard. The risk will be modulated by the magnitude of the hazard and the vulnerability, but it is also important to



Risk Assessment, Fig. 1 Graphical representation of the relationship between risk, hazard, and vulnerability. Risk arises from the intersection of hazards and vulnerabilities, when vulnerable systems are exposed to natural hazards. Modified from Wood 2011

notice that the extent of the intersection or exposure will also determine the risk, even if the hazard or magnitude does not change individually.

These definitions emphasize the role of vulnerability in contributing to risk generation. Historically, the hazard variable, and natural hazards in particular, has received most of the attention in both theoretical risk work and practical applications of risk assessment and management (White et al. 2001).

The vulnerability analysis usually falls outside the field of engineering geology and is undertaken within other disciplines of engineering, social sciences, economics, etc. For that reason, involvement of engineering geologists is usually limited to the hazard assessment component of risk assessment. It is, however, important for the engineering geologist to be aware of the broader context.

Risk Assessment and the Risk Management Process

Risk assessment has a crucial role in the risk management process, and it is in this broader context that the importance of risk assessment should be understood and appreciated. Risk management is defined by the United Nations Office for Disaster Risk Reduction as "The systematic approach and practice of managing uncertainty to minimize potential harm and loss" (UNISDR 2016); the stated goal in this definition is to minimize harm and loss, but this has to be done in a context of uncertainty. Risk always implies uncertainty (Rougier et al. 2013). The uncertainty factor is unavoidable in risk management; however, a minimum knowledge of the potential causes for loss and their associated likelihoods is necessary to implement any risk management process. The risk assessment provides basic information and knowledge about the problem and sets the stage for potential courses of action (i.e., solutions to the problem) in the management process. The risk management process can be illustrated by the diagram shown in Fig. 2, in which the risk assessment is a fundamental component.

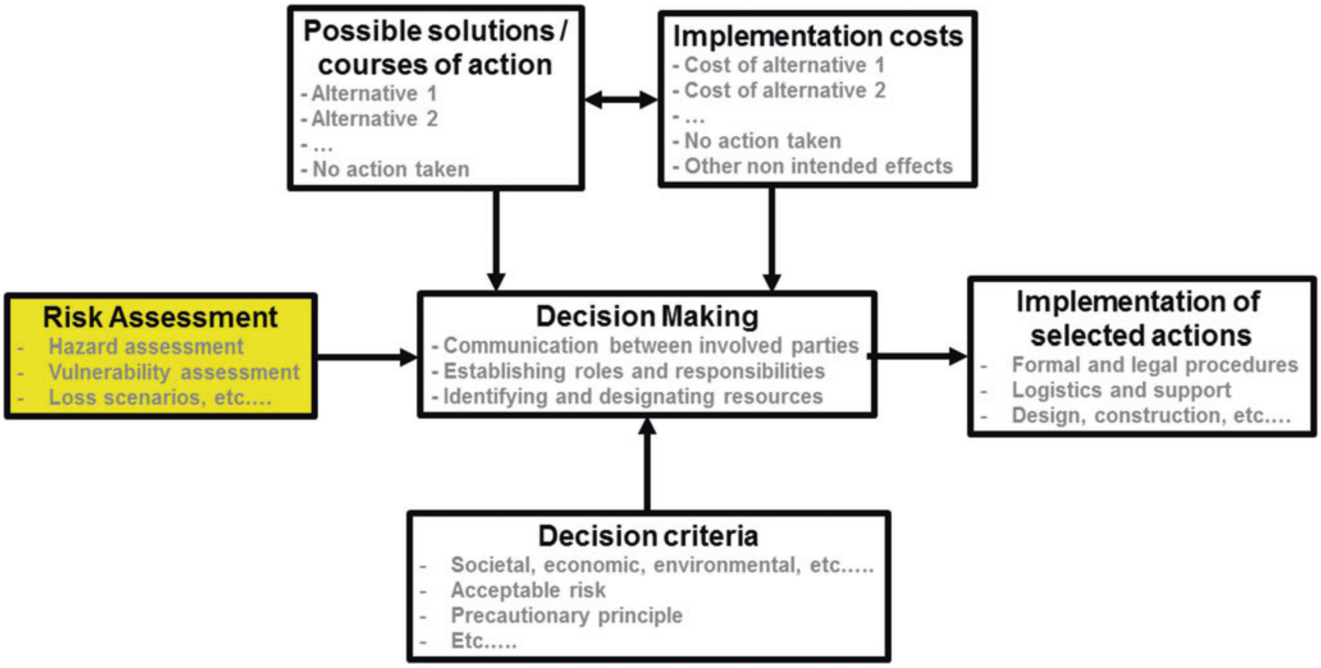
The risk management process involves decision-making on whether to invest or spend resources to reduce a given risk or not. For instance: Is the cost of designing and building more earthquake-resistant structures justified? or Is hardship and potential economic losses from the evacuations of population due to a potential volcanic risk necessary? The risk assessment aims to inform such a decision-making process by providing estimates of the potential losses that would result from different risk scenarios, for example, earthquakes of different magnitudes, occurrence of different volcanic hazards, etc.

The decision-making process does not only depend on the information provided by the risk assessment but also depends on the value judgements that society, or whoever represents its interests in the decision-making process (e.g., the authority), make about the different potential outcomes (Fischhoff and Lichtenstein 1984). This is reflected in the definition of criteria such as acceptable risk levels, the precautionary principle, etc.

Uncertainty in risk assessment is unavoidably transferred to the risk management decision-making process. Reducing uncertainty in risk assessment is therefore highly desirable, but doing so may come at a high cost (e.g., collecting more data, doing more analysis), and will be constrained at some point by practical and even fundamental limits (Rougier et al. 2013). Being unavoidable, uncertainty has to be represented and formalized in an adequate way in the risk assessment. Usually this is done through probabilistic analysis, in which the probabilities of different risk scenarios or outcomes are estimated through some appropriate model. In the decision-making process, the losses for each potential outcome or scenario are weighted by their estimated probability of occurrence to obtain an expected loss. Sometimes an "event" or "probability tree" formalization is used for that effect.

Assessing Hazard and Vulnerability and Integrating Them into a Risk Assessment

In the context of engineering geology, the hazard assessment methodologies depend on the type of geologic processes or phenomena involved, but they often share general characteristics. A source process is usually identified at the beginning of the assessment, be it a seismic source, unstable slope area, volcanic system, etc. A consideration of potential scenarios for the process is then defined, usually considering a range of



Risk Assessment, Fig. 2 Risk management process

different magnitudes and locations for the phenomena involved. Different types of phenomena and their interactions can also be considered, for instance, landslides triggered by earthquakes. A source process may be of limited areal extent, but its effects could propagate over a much more extensive area; therefore, a model for propagation is usually also involved. Using the source locations and propagation models, it may be possible to map the geographic extent of the area that could potentially be impacted by the hazard. Figure 3 shows a schematic diagram of this process.

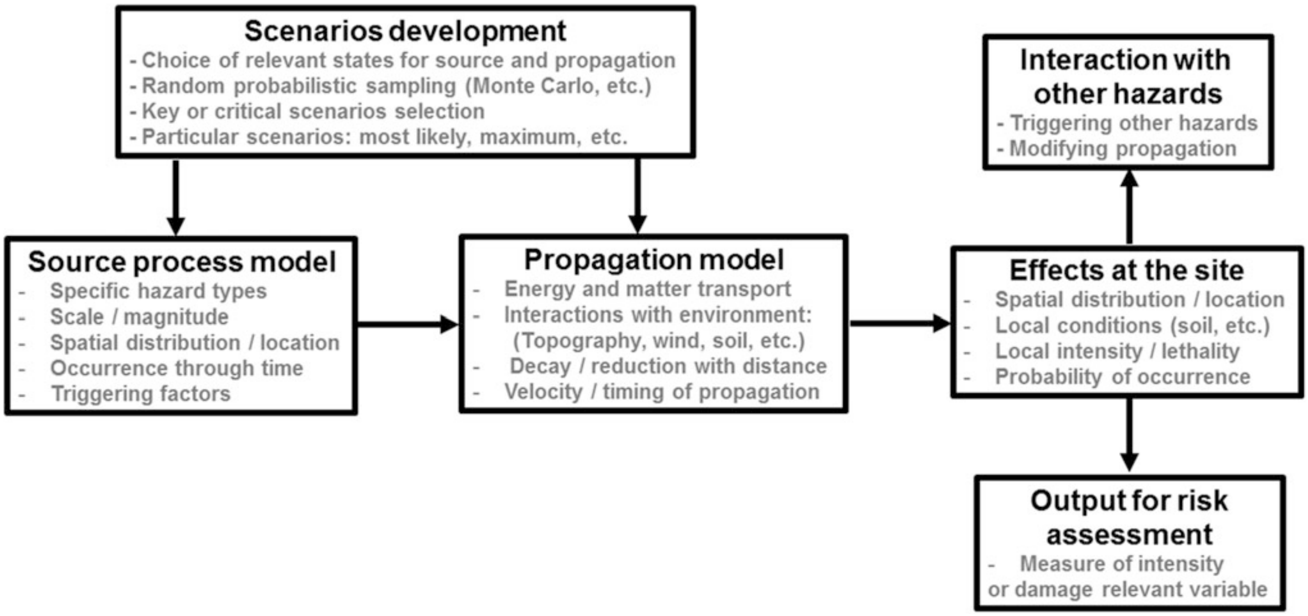
Multiple scenarios, assuming different conditions for the source and propagation models, can be defined. If probabilities can be attached to each of them, a full probabilistic analysis may be possible (Rougier et al. 2013). Probabilistic analysis strategies may involve the random sampling of the input variables and parameters for the source and propagation models to produce a Monte Carlo simulation for the output of the models, that is, a probabilistic hazard map. Choosing the right distribution for the input parameters can be difficult and usually requires extensive historical data on previous occurrences of the hazard phenomena.

Vulnerability assessment is usually done by professionals in fields other than engineering geology, depending on the type of vulnerability being assessed. Structural vulnerability can be evaluated by structural and civil engineers, such as in terms of expected damage that a structure may experience under a given seismic ground acceleration or the maximum load of volcanic ash that a roof can withstand. It is important to notice that in these examples the structural vulnerability analysis uses information produced by hazard analyses (e.g. ground acceleration, ash loading) as an input; this is usually the case and illustrates the intimate interaction between hazard and vulnerability assessments. Other types of vulnerability, for example, economic, social, etc., could in principle also be assessed in a similar way but are in practice sometimes more difficult to establish in a quantitative manner. Economic vulnerability could be related to people's livelihood through exposure to the hazard, such as agricultural land exposed to landslide hazard, but is often also heavily dependent on the internal dynamics of the economic system in which people are embedded (Blaikie et al. 2004). This results in a less straightforward relationship to the hazards. The situation can be even more complex for other types of vulnerability, resulting in a less interactive analysis with respect to hazards.

Integrating hazard and vulnerability analyses into the risk assessment will depend on the format and nature of the assessment. In a quantitative, probabilistic risk assessment, both hazard and vulnerability inputs need to provide relevant information in that format. When the aim is to assess the geographic distribution of risk, both hazard and vulnerability inputs have to be in a geographic format, such as GIS layers. In other cases, the hazard and, particularly, the vulnerability inputs cannot be provided in an easily quantifiable format, which will result in a risk assessment that is more qualitative in nature.

Summary and Conclusions

Risk assessment involves estimating risks based on an analysis of the relevant hazards and vulnerabilities. The risk assessment



Risk Assessment, Fig. 3 General source-propagation-site process involved in many hazard modeling methods

is a crucial component of risk management, as it provides the input for informed decision-making on risk reduction actions. Uncertainty in risk assessment is unavoidable but should be minimized as much as possible; uncertainty can be incorporated in the analysis by using probabilistic methods and in the final decision-making process. Hazard and risk assessment methodologies produce results that can be integrated into a final risk assessment and, for that purpose, the output from hazard and risk analyses have to be in a compatible format.

Cross-References

- ▶ Earthquake
- ▶ Engineering Geomorphology
- ▶ Geohazards
- ▶ Hazard Assessment
- ▶ Landslide
- ▶ Mass Movement
- ▶ Risk Mapping
- ▶ Subsidence
- ▶ Volcanic Environments

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Risk Mapping

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Definition

A process to determine the probability of losses by analyzing potential hazards and evaluating existing conditions of vulnerability that could pose a threat of harm to property, people, livelihoods, and the environment on which they depend (UN-ISDR 2009)

Introduction

The Earth is shaped by endogenic processes, caused by forces from within the Earth, resulting in hazardous events like earthquakes or volcanic eruptions, and exogenic processes, caused by forces related to the Earth's atmosphere, hydrosphere, geosphere, biosphere, and cryosphere and their interactions. Anthropogenic activities have had a very important influence on a number of these processes, especially in the last 200 years, for instance, through the increase of greenhouse gases, leading to global warming, but also through dramatic changes in the land cover and land use and overexploitation of scarce resources. The above mentioned processes from endogenic, exogenic, and anthropogenic sources may lead to potentially catastrophic events, even in locations that may be far away. For instance, earthquakes might trigger landslides which may lead to landslide-dammed lakes that may break out breach and cause flooding downstream. Or the dams of large reservoirs in mountains, constructed for hydropower, irrigation, or drinking water, may fail under an earthquake or extreme rainfall event and cause a similar flood wave.

These potentially harmful events are called hazards. They pose a level of threat to life, health, property, or environment. They may be classified in different ways, for instance, according to the main origin of the hazard: geophysical, meteorological, hydrological, climatological, biological, extraterrestrial, and technological (see Table 1, from Guha-Sapir et al. 2016). Such classifications are always somewhat arbitrary, and several hazard types could be grouped in different categories, for instance, landslides could be caused by earthquakes, extreme precipitation, or human interventions.

Hazards have a number of characteristics that should be understood in order to assess and subsequently reduce their potential damage. Hazards with certain magnitudes may occur with certain frequencies, as small events may occur often and large events seldom. In order to be able to establish a magnitude-frequency relationship for hazard events, it is generally necessary to collect historical data (e.g., from

Risk Mapping, Table 1 Classification of hazard types as used by the International Disaster Database EM-DAT (Guha-Sapir et al. 2016), which is based on and adapted from the IRDR Peril Classification and Hazard Glossary (IRDR 2014)

Main group	Main subgroup	Main type	Subtype
Natural	**Geophysical**: A hazard originating from solid Earth. This term is used interchangeably with the term geological hazard	Earthquake	Ground shaking, tsunami
		Mass movement	
		Volcanic	Ashfall, lahar, pyroclastic flow, lava flow
	Meteorological: A hazard caused by short-lived, micro to mesoscale extreme weather and atmospheric conditions that last from minutes to days	Storm	Extratropical storm, tropical storm, convective storm
		Extreme temperature	Cold wave, heat wave, severe winter conditions
		Fog	
	Hydrological: A hazard caused by the occurrence, movement, and distribution of surface and subsurface freshwater and saltwater	Flood	Coastal flood, riverine flood, flash flood, ice jam flood
		Landslide	Avalanche (snow, debris), mudflow, rockfall
		Wave action	Rogue wave, seiche
	Climatological: A hazard caused by long-lived, meso to macroscale atmospheric processes ranging from intra-seasonal to multi-decadal climate variability	Drought	
		Glacial lake outburst	
		Wildfire	Forest fire, land fire (bush, pasture)
	Biological: A hazard caused by the exposure to living organisms and their toxic substances or vector-borne diseases that they may carry. Examples are venomous wildlife and insects, poisonous plants, and mosquitoes carrying disease-causing agents such as parasites, bacteria, or viruses (e.g., malaria)	Epidemic	Viral , bacterial, parasitic, fungal, prion disease
		Insect infestation	Grasshopper, locust
		Animal accident	
	Extraterrestrial: A hazard caused by asteroids, meteoroids, and comets as they pass near Earth, enter the Earth's atmosphere, and/or strike the Earth and by changes in interplanetary conditions that affect the Earth's magnetosphere, ionosphere, and thermosphere	Impact	
		Space weather	Energetic particles, geomagnetic storm
Technological	Industrial accident		Chemical spills, collapse, explosion, fire, gas leak, poisoning, radiation, others
	Transport accident		Air, road, rail, water
	Miscellaneous accident		Collapse, explosion, fire, others

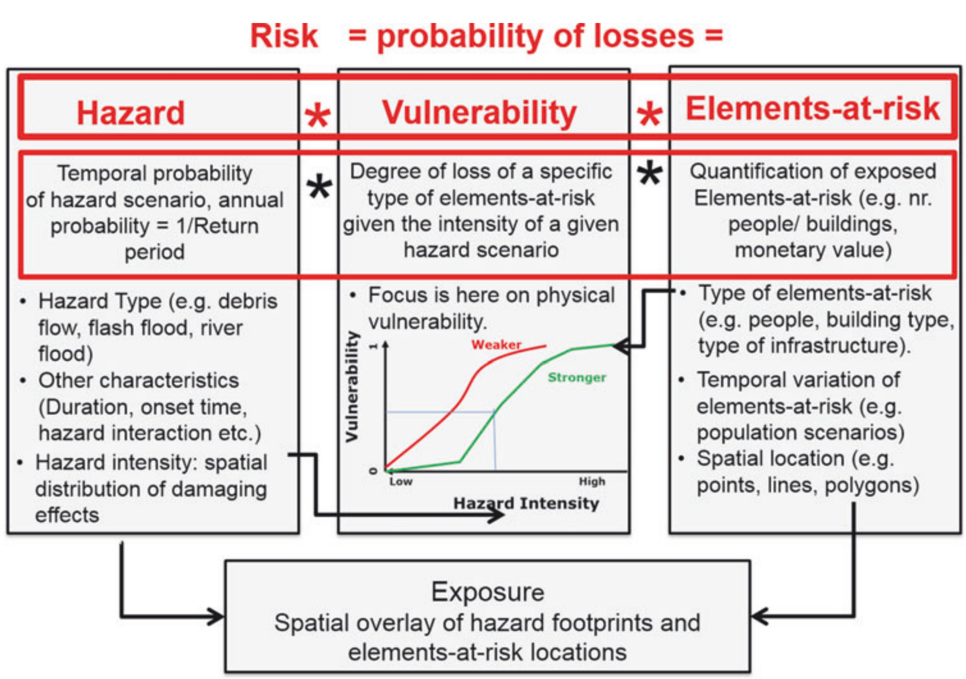
seismograph, meteorological stations, stream gauges, historical archives, remote sensing, field investigations, etc.) and carry out statistical analysis (e.g., using extreme event analysis such as Gumbel analysis) (Van Westen et al. 2008). The magnitude of the hazard gives an indication of the size of the event, or the energy released, whereas the intensity of a hazard refers to the spatially varying effects. For example, earthquake magnitude refers to the energy released by the ruptured fault (e.g., measured on the Richter scale), whereas the intensity refers to the amount of ground shaking which varies with the distance to the epicenter (e.g., measured on Modified Mercalli scale). The magnitude of floods may be measured as the discharge in the main channel at the outlet of a watershed before leaving the mountainous area, whereas the intensity may be measured as the water height or velocity which is spatially distributed and depends on the local terrain. For some types of hazards, there is no unique intensity scale defined, for instance, for landslides (Corominas et al. 2014).

These events may be potentially harmful to people, property, infrastructure, economy, and activities but also to the environment, which are all grouped together under the term "elements at risk" or assets. Also the term exposure is used to indicate those elements at risk that are subject to potential losses. Important elements at risk that should be considered in analyzing potential damage of hazards are population, building stock, essential facilities, and critical infrastructure. Critical infrastructure consists of the primary physical structures, technical facilities, and systems which are socially, economically, or operationally essential to the functioning of a society or community, both in routine circumstances and in the extreme circumstances of an emergency (UN-ISDR 2009). Elements at risk have a certain level of vulnerability, which can be defined in a number of different ways. The general definition is that vulnerability describes the characteristics and circumstances of a community, system, or asset that make it susceptible to the damaging effects of a hazard (UN-ISDR 2009). There are many aspects of vulnerability, related to physical, social, economic, and environmental conditions (see, e.g., Birkmann 2006). When considering physical vulnerability only, it can be defined as the degree of damage to an object (e.g., building) exposed to a given level of hazard intensity (e.g., water height, ground shaking, impact pressure).

Risk mapping is defined as the probability of harmful consequences or expected losses (deaths, injuries, property, livelihoods, economic activity disrupted, or environment damaged) resulting from interactions between natural or human-induced hazards and vulnerable conditions (UN-ISDR 2009; EC 2011). Risk can be presented conceptually with the basic equation indicated in Fig. 1.

Risk Assessment and Mapping

ISO 31000 (2009) defines risk assessment as a process made up of three processes: risk identification, risk analysis, and risk evaluation. Risk identification is the process that is used to find, recognize, and describe the risks that could affect the achievement of objectives. Risk analysis is the process that is



Risk Mapping, Fig. 1 Schematic representation of risk as the multiplication of hazard, vulnerability, and quantification of the exposed elements at risk. The various aspects of hazards, vulnerability, and elements at risk and their interactions are also indicated. This framework focuses on the analysis of physical losses, using physical vulnerability data used to understand the nature, sources, and causes of the risks that have been identified and to estimate the level of risk. It is also used to study impacts and consequences and to examine the controls that currently exist. Risk evaluation is the process that is used to compare risk analysis results with risk criteria in order to determine whether or not a specified level of risk is acceptable or tolerable.

The term risk mapping is often used as being synonymous with risk analysis in the overall framework of risk management. Risk assessments (and associated risk mapping) include a review of the technical characteristics of hazards such as their location, intensity, frequency, and probability; the analysis of exposure and vulnerability including the physical, social, health, economic, and environmental dimensions; and the evaluation of the effectiveness of prevailing and alternative coping capacities in respect to likely risk scenarios (UN-ISDR 2009; EC 2011; ISO 31000 2009). In the framework of natural hazard risk assessment, the term risk mapping also indicates the importance of the spatial aspects of risk assessment. All components of the risk equation (Fig. 1) are spatially varying, and the risk assessment is carried out in order to express the risk within certain areas. To be able to evaluate these components, we need to have spatially distributed information. Computerized systems for the collection, management, analysis, and dissemination of spatial information, so-called Geographic Information Systems (GIS), are used to generate the data on the various risk components and to analyze the risk (OAS 1991; Coppock 1995; Cova 1999; Van Westen 2013). Hazard data are generally the most difficult to generate. For each hazard type (e.g., flooding, debris flow, rockfall), so-called hazard scenarios should be defined, which are hazard events with a certain magnitude/intensity/frequency relationship (e.g., flood depth maps for 10-, 50-, and 100-year return periods). Different types of modeling approaches are required for the hazard scenario analysis, depending on the hazard type, scale of analysis, availability of input data, and availability of models. Generally speaking, a separate analysis is required to determine the probability of occurrence for a given magnitude of events, followed by an analysis of the initiation of the hazard (e.g., hydrological modeling or landslide initiation modeling) and of the runout or spreading of the hazard (e.g., hydrodynamic modeling or landslide run-out modeling). Overviews of hazard and risk assessment methods for landslides, for example, can be found in Corominas et al. (2014) and for floods in Prinos (2008). Elements-at-risk data are very often based on building footprint maps, which represent the location of buildings, with attributes related to their use, size, type, and number of people during different periods of the year (e.g., daytime, night time). Remote sensing is often used to extract these building maps if existing cadastral maps are not available. For other elements at risk like transportation infrastructure and land cover maps, also remote sensing data are used as important inputs. Vulnerability data are often collected in the form of vulnerability curves, fragility curves, or vulnerability matrices, which indicate the relationship between the levels of damage to a particular type of element at risk (e.g., single-story masonry building) given intensity levels of a particular hazard type (e.g., debris flow impact pressure). Generation of vulnerability curves is a complicated issue, as they can be generated empirically from past damage event for which intensity and damage are available for many elements at risk or through numerical modeling (Roberts et al. 2009).

Risk mapping for natural hazard risk can be carried out at a number of scales and for different purposes. Table 2 gives a summary. In the following sections, four methods of risk mapping will be discussed: quantitative risk assessment (QRA), event tree analysis (ETA), risk matrix approach (RMA), and indicator-based approach (IBA).

Quantitative Risk Assessment

If the various components of the risk equation can be spatially quantified for a given set of hazard scenarios and elements at risk, the risk can be analyzed using the following equation:

Risk Mapping, Table 2 Indication of scales of analysis with associated objectives and data characteristics (approaches: *QRA* quantitative risk assessment; *EVA* event tree analysis; *RMA* risk matrix approach; *IBA* indicator-based approach)

Scale of analysis	Scale	Possible objectives	Possible approaches
International, global	<1:1 million	Prioritization of countries/regions; early warning	Simplified RMA and IBA
Small: provincial to national scale	<1:100,000	Prioritization of regions; analysis of triggering events; implementation of national programs; strategic environmental assessment; insurance	Simplified EVA, RMA, and IBA
Medium: municipality to provincial level	1:100,000–1:25,000	Analyzing the effect of changes; analysis of triggering events; regional development plans	RMA/IBA
Local: community to municipality	1:25,000–1:5,000	Land use zoning; analyzing the effect of changes; Environmental Impact Assessments; design of risk reduction measures	QRA/EVA/ RMA IBA
Site specific	1:5,000 or larger	Design of risk reduction measures; early warning systems; detailed land use zoning	QRA/EVA/RMA

$$\text{Risk} = \sum_{\text{All hazards}} \left(\int_{P_T=0}^{P_T=1} P_{(T|HS)} \times \left(\sum_{\text{All EaR}} \left(P_{S|HS} \times \left(A_{ER|HS} \times V_{ER|HS} \right) \right) \right) \right) \tag{1}$$

in which:

$P_{(T\,|\,HS)}$ = the temporal probability of a certain hazard scenario (HS). A hazard scenario is a hazard event of a certain type (e.g., flooding) with a certain magnitude and frequency.

$P_{(S\,|\,HS)}$ = the spatial probability that a particular location is affected given a certain hazard scenario.

$A_{(ER\,|\,HS)}$ = the quantification of the amount of exposed elements at risk, given a certain hazard scenario (e.g., number of people, number of buildings, monetary values, hectares of land).

$V_{(ER\,|\,HS)}$ = the vulnerability of elements at risk given the hazard intensity under the specific hazard scenario (as a value between 0 and 1).

The method is schematically indicated in Fig. 2. GIS operations are used to analyze the exposure as the intersection between the elements at risk and the hazard footprint area for each hazard scenario. For each element at risk also, the level of intensity is recorded through a GIS overlay operation. These intensity values are used in combination with the element-at-risk type to find the corresponding vulnerability curve, which is then used as a look-up table to find the vulnerability value. The manner in which the amount of elements at risk are characterized (e.g., as number of buildings, number of people, economic value) also defines the way in which the risk is calculated. The multiplication of exposed amounts and vulnerability should be done for all elements at risk for the same hazard scenario. The results are multiplied with the spatial probability that the hazard footprint actually intersects with the element at risk for the given hazard scenario $P_{(S\,|\,HS)}$ to account for uncertainties in the hazard modeling. The resulting value represents the losses, which are plotted against the temporal probability of occurrence for the same hazard scenario in a so-called risk curve. This is repeated for all available hazard scenarios. At least three individual scenarios should be used, although it is preferred to use at least six events with different return periods (FEMA 2004) to better represent the risk curve. The area under the curve is then calculated by integrating all losses with their respective annual probabilities. It is possible to create risk curves for the entire study area, or for different spatial units, such as administrative units, census tracks, road or railway sections, etc. Risk can be presented in a number of different ways, depending on the objectives of the risk assessment (Birkmann 2007). Risk can be expressed in absolute or relative terms. Absolute population risk can be expressed as individual risk (the annual probability of a single exposed person to be killed) or as societal risk (the relation between the annual probability and the number of people that could be killed). Absolute economic risk can be expressed in terms of average annual loss, maximum probable loss, or other indices that are calculated from a series of loss scenarios, each with a relation between frequency and expected monetary losses (Jonkman et al. 2003).

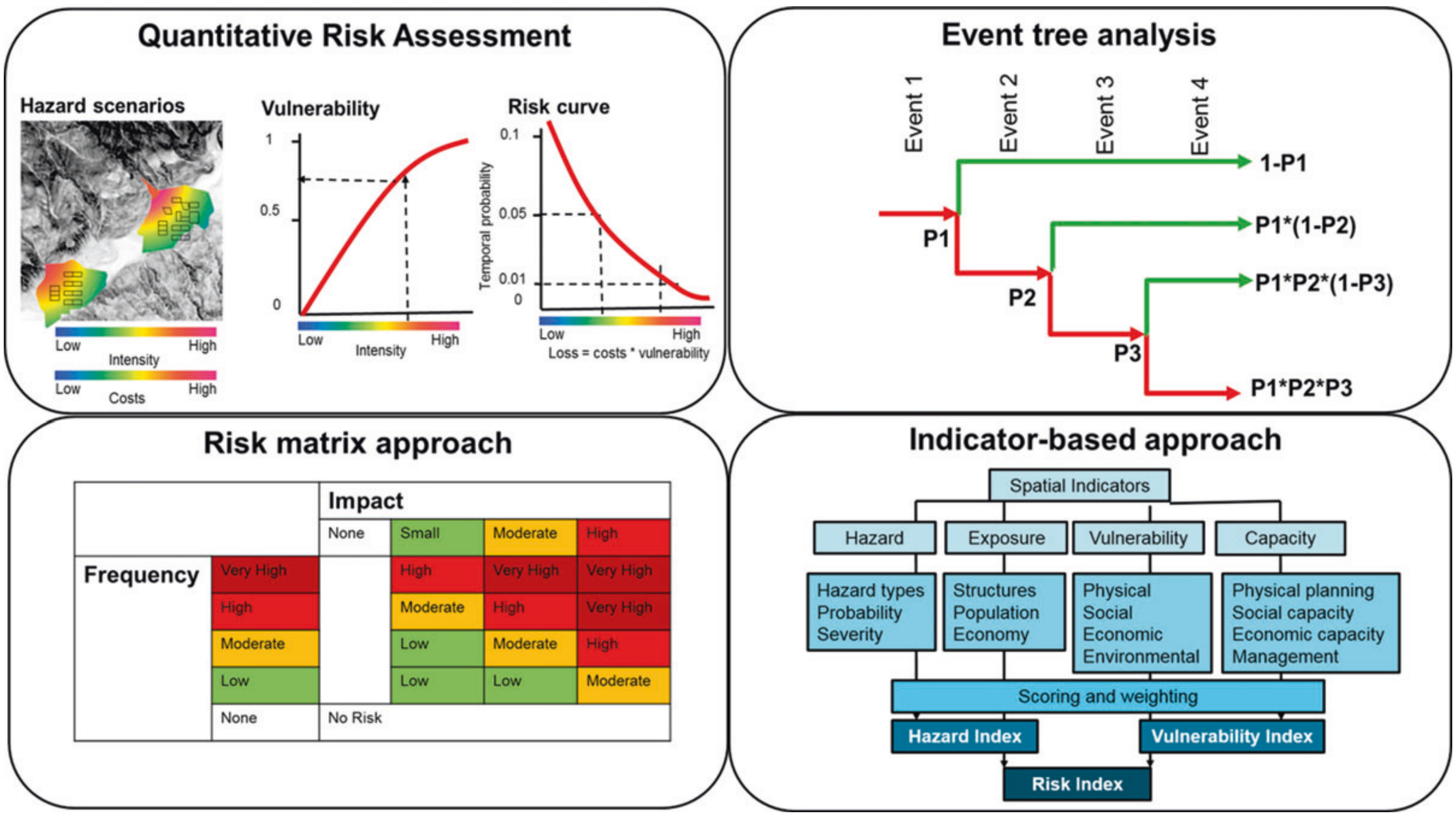
The components that are involved in risk assessment have a high degree of uncertainty. Aleatory uncertainty is associated with the variation of the input data used in the risk assessment, for example, the variations in soil characteristics used to model landslide probability, surface characteristics, building characteristics, etc. These are normally incorporated in probabilistic risk analysis (Bedford and Cooke 2001) which calculates thousands of hazard and risk scenarios taking the variations of the input factors and calculating exceedance probabilities using techniques such as Monte Carlo simulation. Epistemic uncertainty refers to uncertainty associated with incomplete or imperfect knowledge about the processes involved and lack of sufficient data. This is often a serious problem as there may not be enough data available to determine individual hazard scenarios or there are no vulnerability curves for the types of elements at risk within the study area.

Risk assessment is computationally intensive. It can be carried out using conventional GIS systems, although it is advisable to use specific software tools. A number of software tools have been developed for multi-hazard risk assessment, for example, HAZUS in the USA (Schneider and Schauer 2006), RiskScape in New Zealand (Schmidt et al. 2011), CAPRA in Central America (CAPRA 2013), and MATRIX (Garcia-Aristizabal and Marzocchi 2013) and RISK-GIS in Australia (Granger et al. 1999). The common aspect of these software programs is that they are used to analyze damages and replacement costs, casualties, disruption, and number of people affected by various hazards. They differ in terms of the methods used for hazard assessment, asset exposure analysis, and vulnerability assessment and the method for risk calculation. What they also have in common is that these methods are very data demanding.

R

Event Tree Analysis

One of the difficult issues in natural hazard risk assessment is how to analyze the risk for more than one hazard in the same area and the way they interact. The simplest approach is to consider that the hazards are independent and caused by different triggers. If that is the case, the risk can be calculated by adding the average annual losses for the different types of hazard. Compared to single processes, standard approaches



Risk Mapping, Fig. 2 Components relevant for risk assessment and the four major types of risk mapping that are presented in this entry and methodological frameworks for multi-hazard risk assessment are less common in the literature (Kappes et al. 2012), which is related to the complex nature of the interaction between the hazards and the difficulty to quantify these. Hazard may occur in sequence, where one hazard may trigger the next, as is the case in the example mentioned above on earthquake-triggered landslide-dam break-out flooding. These hazard chains or domino effects are extremely difficult to quantify over certain areas, although good results have been obtained at a local level (e.g., Peila and Guardini 2008). Hazards may also occur simultaneously, caused by the same triggering event, and may affect the same area, for example, as flash flooding or debris flows that affect the same area. One hazard may also alter the existing conditions so that a subsequent hazard could occur in different locations and with a higher frequency, for example, the higher hazard for debris flows after forest fires. The best approach for analyzing such hazard chains is to use a so-called event tree. An event tree analysis is a system which is applied to analyze all the combinations (and the associated probability of occurrence) of the parameters that affect the system under analysis. All the analyzed events are linked to each other by means of nodes (see Fig. 2), all possible states of the system are considered at each node, and each state (branch of the event tree) is characterized by a defined value of probability of occurrence.

Risk Matrix Approach

Risk assessments are often complex and do not allow to develop a full numerical approach, since many aspects are not fully quantifiable or have a very large degree of uncertainty. This may be related to the difficulty to define hazard scenarios, map and characterize the elements at risk, or define the vulnerability using vulnerability curves. In order to overcome these problems, the risk is often assessed using so-called risk matrices or consequences-frequency matrices (CFM) (see Fig. 2). They permit the classification of risks based on expert knowledge with limited quantitative data (Haimes 2008; Jaboyedoff et al. 2014). The risk matrix is made of classes of frequency of the hazardous events on one axis and the consequences (or expected losses) on the other axis. Instead of using fixed values, the use of classes allows for more flexibility and incorporation of expert opinion. Such methods have been applied extensively in natural hazard risk assessment, for example, in Switzerland (Jaboyedoff et al. 2014). This approach also permits to visualize the effects and consequences of risk reduction measures and to give a framework to understand risk assessment. The system depends on the quality of the group of experts that are formed to identify the hazard scenarios and that carry out the hazard filtering and ranking in several substages characterized by frequency (probability) and impact classes and their corresponding limits (Haimes 2008).

Indicator-Based Approach

There are many situations where (semi)quantitative methods for risk mapping are not appropriate. This could be because some of the data are lacking, thus making it impossible to quantify the components, such as hazard frequency, intensity, and physical vulnerability, for instance, when the risk assessment is carried out over large areas or in areas with limited data. Another reason is that one would like to take into account a number of different components of vulnerability that are not incorporated in (semi)quantitative methods, such as social vulnerability, environmental vulnerability, and capacity. In those cases, it is common to follow an indicator-based approach to measure risk and vulnerability through selected comparative indicators in a quantitative manner in order to be able to compare different areas or communities. The process of disaster risk assessment is divided into a number of components, such as hazard, exposure, vulnerability, and capacity (see Fig. 2), through a so-called criteria tree, which list the subdivision into objectives, sub-objectives, and indicators. Data for each of these indicators are collected at a particular spatial level, for instance, by administrative units. These indicators are then standardized (e.g., by reclassifying them between 0 and 1) and weighted internally within a sub-objective, and then the various sub-objectives are also weighted among themselves. Although the individual indicators normally consist of quantitative data (e.g., population statistics), the resulting vulnerability, hazard, and risk results are scaled between 0 and 1. These relative data allows to comparison of the indicators for the various administrative units. These methods can be carried out at different levels, ranging from local communities (e.g., Bollin and Hidajat 2006) and cities (Greiving et al. 2006) to countries (Van Westen et al. 2012).

Conclusions

The four methods for risk assessment treated in this chapter all have certain advantages and disadvantages, which are summarized in Table 3. The quantitative risk assessment method is the best for evaluating several alternatives for risk reduction, through a comparative analysis of the risk before and after the implementation followed by a cost–benefit analysis. The event tree analysis is the best approach for analyzing complex chains of events and the associated probabilities. The risk matrix approach is often the most practical approach as basis for spatial planning, where the effect of risk reduction methods can be seen as changes in the classes within the risk matrix. The indicator-based approach, finally, is the best when

Risk Mapping, Table 3 Advantages and disadvantages of the four risk assessment methods discussed

Method	Advantages	Disadvantages
Quantitative risk assessment (QRA)	Provides quantitative risk information that can be used in cost-benefit analysis of risk reduction measures	Very data demanding. Difficult to quantify temporal probability, hazard intensity, and vulnerability
Event tree analysis	Allows modeling of a sequence of events and works well for domino effects	The probabilities for the different nodes are difficult to assess, and spatial implementation is very difficult due to the lack of data
Risk matrix approach	Allows expression of risk using classes instead of exact values and is a good basis for discussing risk reduction measures	The method does not give quantitative values that can be used in cost–benefit analysis of risk reduction measures. The assessment of impacts and frequencies is difficult, and one area might have different combinations of impacts and frequencies
Indicator-based approach	Only method that allows a holistic risk assessment, including social, economic, and environmental vulnerability and capacity	The resulting risk is relative and does not provide information on actual expected losses

there are insufficient data to carry out a quantitative analysis but also as a follow-up of a quantitative analysis as it allows one to take into account other aspects than just physical damage. Even though hazard and risk mapping may have taken place, real risk reduction will only happen when it leads to a reduction in either the hazard frequency and intensity or the number of exposed elements at risk and their vulnerability. This requires integration of risk analysis into a risk management framework, which includes the adoption of policy and regulations and interaction of geoscientists within this process (DeGraff 2012).

Cross-References

- ▶ Earthquake Magnitude
- ▶ Hazard Assessment
- ▶ Risk Assessment

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Rock Bolts

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Definition

Tension elements designed to resist tension forces in the rock, usually made of a steel bar installed in pre-drilled hole and grouted.

Characteristics

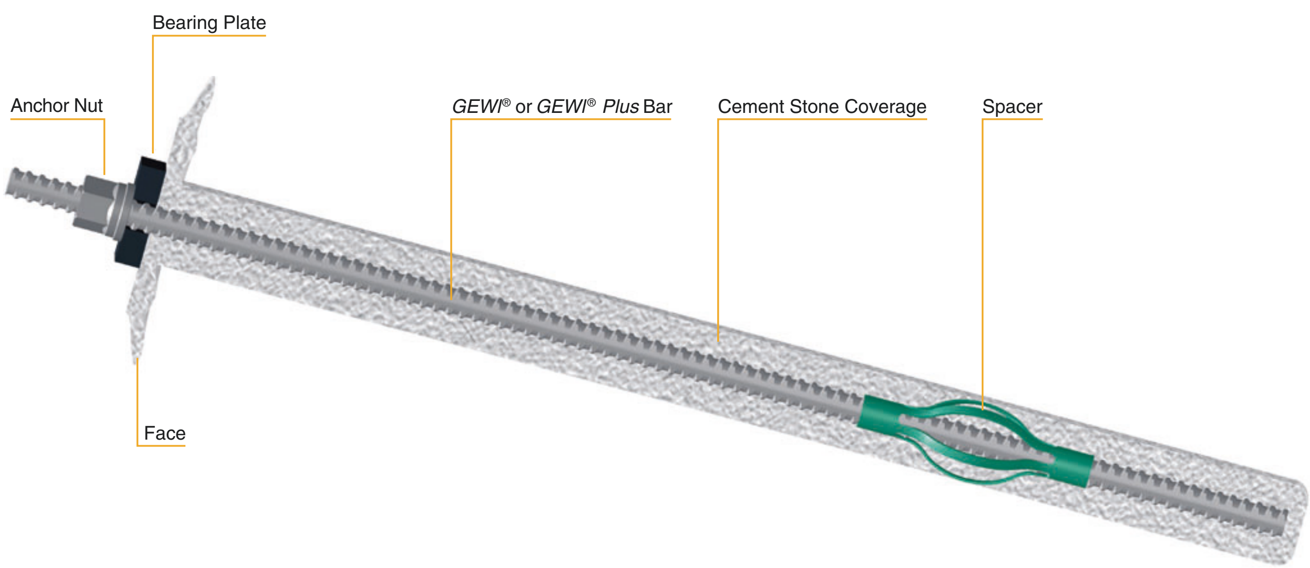
A rock bolt has the following elements:

- *Head* is the anchor end outside the ground, comprising a plate, a nut, and a bearing plate.
- *Bonded length* is the length that transmits tension forces to the ground.
- *Spacer* is a plastic device, not always used to keep the tendon centered in the drill hole (Fig. 1).

Ortigao and Brito (2004) give additional details of rock bolt characteristics.

Drilling

The most common drilling method is percussion and rotary drilling with pneumatic drill rigs and cutting tools.



Rock Bolts, Fig. 1 Rock bolt (By courtesy of Dywidag)

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Grouting

The most common grout is Portland cement grout. However, when rapid setting is needed, as is common in tunneling, resin grout is used.

Cross-References

- ▶ Grouting
- ▶ Ground Anchors

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Rock Coasts

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Definition

A rocky coast is one "*that is cliffed and yet composed of consolidated material irrespective of its hardness*" (Sunamura 1992, p. 2), that is, it ranges from very hard rock coasts (e.g., granite, basalt) to soft cohesive fluvial or glacial deposits, and they represent some 75% of the world's coastlines. Erosion on such coasts is irreversible and is a function of wave energy and rock strength (Fig. 1). They are usually high, and consequently there is an inherent danger for injury or even death to coastal visitors, so good signage is important (Fig. 1).

Rock Coasts, Fig. 1 The Glamorgan Heritage Coast, UK

Cliff Erosion

Erosion is becoming an acute problem within the coastal zone, and because of global warming/sea level change, *El Nino* effects and greater storminess are likely to increase the risk of cliff failure and reduce human utilization. The general processes responsible for rock coast cliff erosion are well known and have been extensively discussed elsewhere (Sunamura 1992; USACoE 2002; Woodroffe 2002; Trenhaile 1997). Quantifying erosion controls is however rather difficult (Van Jones et al. 2015).

An extensive body of literature supports the significance of mechanical action within the wave erosional system (cf. Sunamura 1992). Realistically, failure depends upon many additional factors, such as hydro-geological processes and ground water seepage through the cliff mass. The retreat rate of rock cliffs varies but depends upon the:

Strength of the Rock Material Forming the Cliff

Axiomatically the harder the rock, the slower is the erosion rate. Rock and joint strength parameters (Table 1) can be derived from field measurement, such as Schmidt hammer readings and tilt tests, and laboratory analyses, for instance, uniaxial compression/tensile strength, point loading tests with values incorporated into standard rock mechanics formulae, such as given below:

$$\mathrm{JRC} = (\alpha - \varphi_b)/\log_{10}[(\mathrm{JCS}/\sigma_n)] \tag{1}$$

$$\varphi_b = (\varphi_b - 20) + 20\,({}^{r}/_{R}) \tag{2}$$

$$\sigma_n = \gamma_h \cos^2\alpha \tag{3}$$

$$t_p = \sigma_n \tan\,[\varphi_b + \mathrm{JRC} \cdot \log_{10}(\mathrm{JCS}/\sigma_n)] \tag{4}$$

where JCS = Joint Compressive Strength, e.g. via Schmidt hammer, JRC = Joint Roughness Coefficient (varies from 0 - very rough to 20, α = joint tilt angle at failure, φ_b = residual angle of friction along the joint, σ_n = mean value of normal stress induced by the sliding block weight, r = uniaxial compressive strength for wet rock samples, R = uniaxial compressive strength for dry rock samples, h = block thickness, γ = bulk density, and t_p = peak shear strength.

Rock Coasts, Table 1 Geomorphic rock mass strength classification for limestone/shales in Lias age rocks, Glamorgan Heritage Coast, Wales (GHC), UK (after Selby 1980; Fig. 1)

Parameter	Limestone	Shale
Intact rock strength	18	5
Weathering	9	5
Joint spacing	21–28	15–21
Orientation	14	14
Width	6	2
Continuity	4	4
Totals	78–85	48–54

Basal Wave Energy

Coastal cliff erosion is a complex system and variations in the geological, erosional, and weathering environment are reflected in the general cliff erosion rates derived from field and laboratory observations. Many investigations have established the links between basal erosion, notching (Fig. 2), cliff instability, and recession (Sunamura 1992; Rosser et al. 2013). Cliff surface deterioration and cliff base erosion are related especially to the assailing force of waves. Quasi-periodic wave erosion is more efficient when high tides coincide with storms whereby basal erosion by waves produce notching, and a laterally extending cliff base hollow. They are clear indicators of cliff erosion.

Three main processes affect a cliff's base:

- Cumulative hydraulic action related to breaking waves, water spray, and high speed droplets. Hydraulic forces that include compression, tension, impact, and shearing actions, which combine to achieve wave quarrying; all of these result in repeated stress placement on rock surface layers.
- Turbulent water currents that lift boulders, pebbles, and sand from the shore platform and beach.
- Shell feeding algae and other organism living in the intertidal band.

Mathematically**,** the major factors of basal erosion are given by Sunamura's (1992) equation:

$$x = f(F_w, s_r, t) \tag{5}$$

where x is the basal cliff erosion distance, F_w is the wave induced force, s_r is cliff material resistance, and t represents time.

If F_W is ≤ 0 no erosion occurs; when $F_W > 0$ erosion takes place.

Amount of Abrasive Material Available at the Cliff Base

Almost by definition, the loose sediment associated with cliff erosion occurs in the pebble-boulder range, which, if in sufficient quantity, is the best beach that nature could envisage to retard erosion. Beach sediment, usually in the

Rock Coasts, Fig. 2 Golfo di Orosei, Sardinia: Active notch at the sea level and last interglacial fossil notch at approx. 8 m a.m.s.l. in a limestone cliff

pebble-boulder range, can accentuate erosion processes by either being an abrasive agent (boulders can smash into a cliff face) or can hinder it to form a protective beach. Increasingly strong wave action produces large hydraulic forces that are accentuated by the abrasive force of rock fragments hurled at the cliff base. The latter set up "*impact stresses on the rock surface, the stress increasing as the mass and/or angle of velocity of the impacting particles are increased"* (Sunamura 1992, p. 78). If the water depth in front of the cliff is higher than approximately half the wavelength, sediment is moved at the base and abrasion does not occur and the cliff becomes more stable (Plunging cliff: C type in Sunamura rock coast classification; Fig. 3).

Mass Movement

The coastal zone has a high frequency of cliff mass movement failures, which reflect the ability of high energy waves to exploit the well-jointed/interbedding nature of "very strong" and "moderately weak" rock materials. Wave undercutting is a critical control of many toppling and joint block detachment forms of failure. The Factor of Safety reduces as the ratio of undercutting depth to distance from the cliff face of tension fractures increase and as thrust forces within joint systems increase due to water infill by wave and tide factors, freeze-thaw and clay infill expansion and contraction.

The main forms of cliff falls are: Toppling (Fig. 4 left), Translation (Fig. 4 center), Buckling, and Falls rock (Fig. 4 right, debris and Earth), where the bulk of the mass falls as a free body; in soft rocks, flows can occur, where movement is faster towards the upper area of a moving body, that is, no block movement. The first two are usually the predominant mechanisms. Toppling occurs when an eccentricity develops

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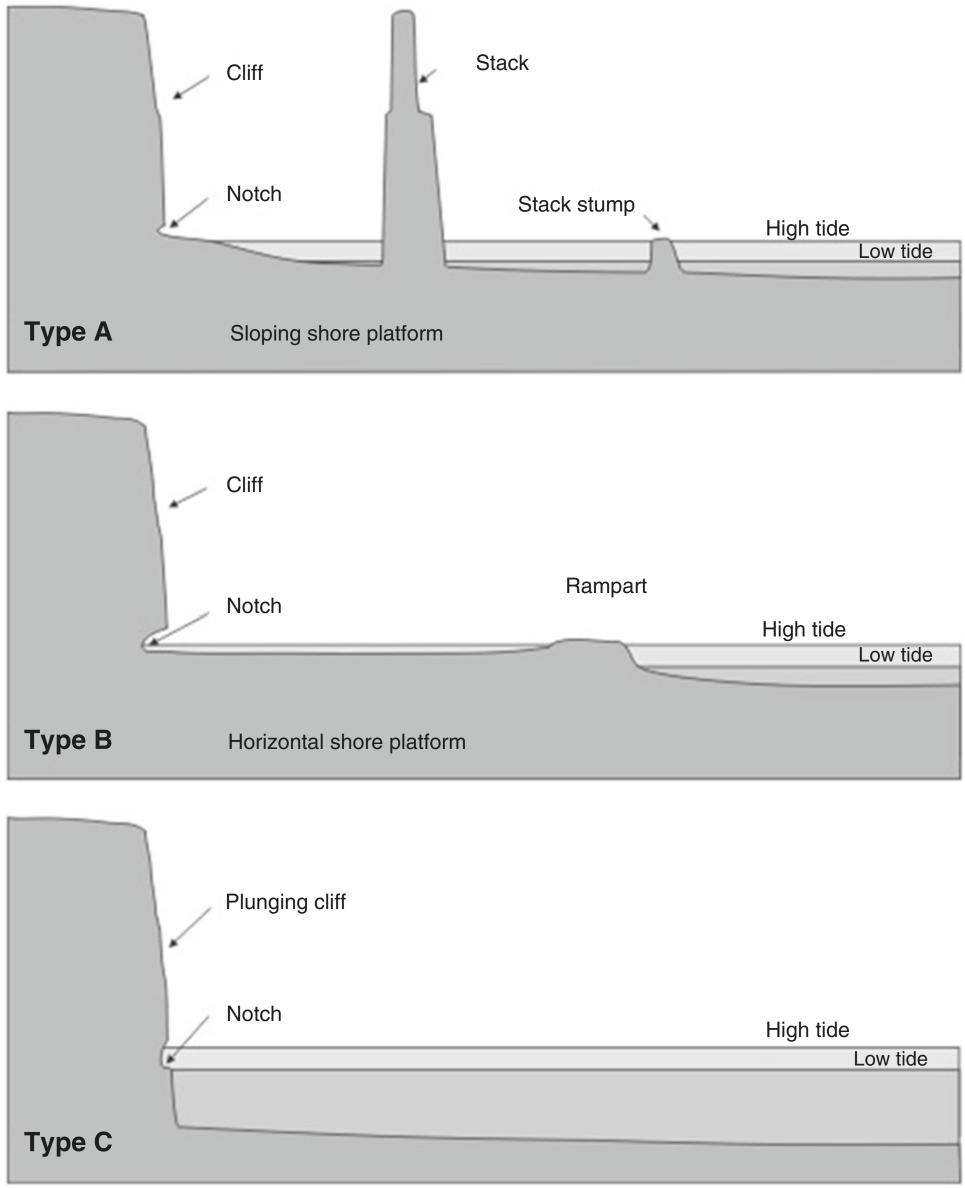
Rock Coasts, Fig. 3 Sunamura's rocky coast features classification: (**a**) sloping shore platform, (**b**) horizontal shore platform, (**c**) plunging cliff (from Pranzini 2004, modified)



so that overturning moments exceed resisting ones and little free fall movement occurs. A fulcrum is necessary along a hard rock band and it occurs when:

$$b/h < \tan ø \text{ and } \psi < ø \quad (6)$$

where b = block base dimensions; h = block height dimensions; ø = angle of frictional resistance; ψ = basal plane angle.

Sliding and toppling occur when

$$b/h < \tan \text{ and } \psi > ø \quad (7)$$

Shear resistance to toppling usually occurs along discontinuities orthogonal to the cliff strike. For translation to occur a hard rock band usually lies on top of a weaker rock unit and translation along a master joint is normal. The failure surface takes the path of least resistance through the rock mass, usually a curved surface.

Weathering

Weathering (chemical, hydrolysis, hydration, oxidation, and solution) and mechanical (frost, thermal stress, salt crystal growth, unloading, and swelling) decreases a rock's mechanical strength and coastal cliff deterioration due to temperature (nonuniform) variations mainly affecting surface rocks. Heat conductivity is a function of the rock's thermal properties, and discontinuous rocks are more vulnerable to thermal expansion/contraction. Rock composition differences cause

Rock Coasts, Fig. 4 (**a**) Toppling failure, Glamorgan Heritage Coast, UK (left); (**b**) Translation on glacial deposits, Poland (center); (**c**) Rock fall, Sardinia, Italy (right)

thermal conductivity anomalies resulting in large temperature gradients, which cause block cracking and displacement along joint/fault lines. Nivation processes by ice/water, salt crystallization, and so on reduce rock mass strength by entering rock discontinuities. Biological influences also can cause weathering. Plant growth can exert physical pressure on existing discontinuities. Marine organisms causing boring, for example, *Lithotrya, Lithophaga*, can also dislodge lithic material from a rock surface (Woodroffe 2002). Algae, fungi, and lichen can cause chemical alteration, especially in the tropics, as well as providing food for grazing organisms that can abrade the surface. This situation is estimated to reflect one-third of the mid-tidal zone at the Aldabra Atoll (Trudgill 1976). The importance of haloclasty (salt weathering) in cliff rock weathering is demonstrated by the ramparts that frequently border rock platforms on the seaward side, where the rock does not dry even at low tide due to constant exposure to spray. Salt does not precipitate and the process is not active leaving that ridge higher than the more sheltered platform (Fig. 5).

Rock Coasts, Fig. 5 Rock platform and rampart (Australia)

Parameter Interaction

Many models exist to show parameter interactions for soft coasts. Interaction of all the parameters discussed previously results in a rate of recession for the cliff mass as an erosion function, which can account for erosion forces and rock strength, as assumed by Sunamura (1992), Eq. (5), and also for the decrease of wave erosion intensity with cliff height. The variety of different mechanisms characterized by diverse time/space scales, process intensities, etc., in a general form can be expressed for the evolution of a 2-D cliff profile under marine influences (Williams et al. 1996; Belov et al. 1999) as:

$$(df/dt)^2 = k(z,t)\left\{(df/dy)^2 + (df/dz)^2\right\} \tag{8}$$

where $f = f(y,z,t)$ is the function defining the cliff profile, $k(z,t)$ is the erosion function – the vertical distribution of erosion intensity and also temporal variation of storm-tide periodicity which modulates impact amplitude.

This equation is eikonal, an uncommon approach for geological literature, reflecting long-term cliff profile change in 2-D Cartesian coordinates. Cliff erosion defined by the erosion function is dependent on the nondimensional erosion amplitude parameter v, which in turn is contingent upon breaking wave energy, storm/tide cyclicity, and cliff geometry. While the speed of cliff retreat is also influenced by cliff geology, strength, and geomorphology of the rock mass, it is implicitly subsumed in v, as a complex parameter.

Conclusions

Pre-sea level rise morphological differences, rock variability in petrography and structure, diversity in climate, tidal range, and wave energy interact to make each rock coast segment a different case as far as landscape, processes, and erosion rate. Sunamura (1992) lists some 220 case studies from the literature and gives cliff erosion rates: most of them are less than 1 cm/yr, with some cliffs almost stable for centuries, but sand, silt, clay, and generic glacial deposits can retreat as much as 10 m/yr. Values of coastal retreat for recently deposited pyroclastic material reach as high as 80 m/yr.

Since most of the processes are nonlinear with time, cliff failure events, or even single rock falls, are rarely forecasted, and this exposes people and values to a significant risk. Since cliffs are among the most attractive elements of coastal landscape monitoring, signage and restrictions are an integrant part of coastal management.

Cross-References

- ▶ Armor Stone
- ▶ Beach Replenishment
- ▶ Biological Weathering
- ▶ Boulders
- ▶ Chemical Weathering
- ▶ Classification of Rocks
- ▶ Climate Change
- ▶ Coast Defenses
- ▶ Erosion
- ▶ Factor of Safety
- ▶ Gabions
- ▶ Landforms
- ▶ Landslide
- ▶ Mass Movement
- ▶ Mechanical Properties
- ▶ Nearshore Structures
- ▶ Physical Weathering
- ▶ Risk Assessment
- ▶ Risk Mapping
- ▶ Rock Laboratory Tests
- ▶ Rock Mass Classification
- ▶ Rock Mechanics
- ▶ Rock Properties
- ▶ Sea Level
- ▶ Shear Stress
- ▶ Stabilization

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Rock Field Tests

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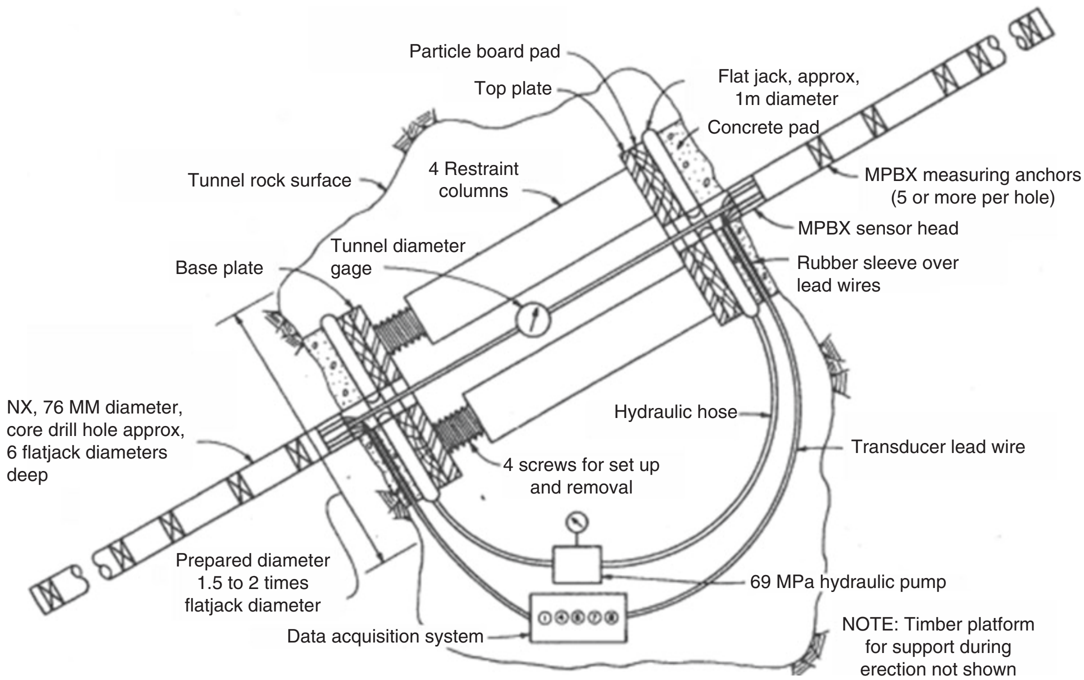
Definition

Rock field tests as discussed here do not refer to tests performed with portable devices that can be used in the field such as the Schmidt hammer or the point load test device. Rock field tests only refer to tests that have to be performed on the ground surface, underground openings, and boreholes to characterize deformability and *in situ* stress. The following is a summary of International Society for Rock Mechanics (ISRM)-suggested methods for rock field tests (Ulusay and Hudson 2007).

Introduction

Designing subsurface or on-surface civil engineering structures requires a sound knowledge of rock mass deformability and *in situ* stress of rock mass. Both deformability and *in situ* stress are controlled by discontinuities in rock and tectonic (paleo or active) stress at the site of proposed construction. Therefore, it is necessary to devise field-testing methods that can be performed inside underground openings, inside boreholes, and on the ground surface. Deformability of a rock mass is its response to change in stress due to loading by engineering/mining structures. It is estimated by applying stress using hydraulic jacks and monitoring displacements to quantify deformability. Examples of deformability tests inside underground openings or on ground surface include the plate test, radial jacking, and large flat jack, whereas the



Rock Field Tests, Fig. 1 Plate test inside a tunnel

downhole plate test and flexible/stiff dilatometers can be used inside a borehole. *In situ* stress is another important rock mass parameter that needs to be measured as well. Three methods for *in situ* stress determination, the flat jack, overcoming, and hydraulic fracturing, are also discussed below. The flat jack and hydraulic fracturing methods involve application of induced pressure, whereas the overcoring is performed by relieving stress by coring around a rock in which a probe is installed.

Deformability Tests

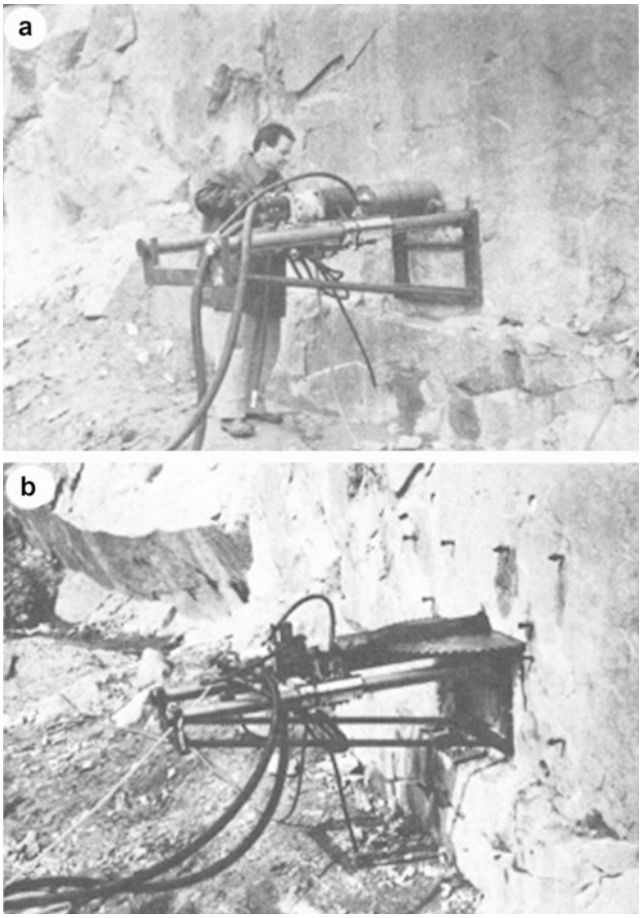
Deformability of rock is a measure of strain (deformation) of intact rock and rock masses in response to change in stress. Deformability tests can be performed on the ground surface, on tunnel walls, or in boreholes. The plate test, radial jacking, and large flat jack are performed on tunnel walls and on ground surface. Downhole tests include downhole plate test and flexible/stiff dilatometers.

Deformability: Plate Test

The plate test is performed by placing two flat jacks (1 m in diameter) on flat surfaces inside a tunnel that are diagonally opposite to each other (Coulson 1979). A flat jack is made up of metal sheets attached around their edges and can be inflated using hydraulic fluids. The flat jacks are secured against a tunnel wall by load-transferring restraint columns that resist motion of the flat jacks into the tunnel opening (Fig. 1). Fluid pressure is applied into the flat jacks simultaneously to apply stress onto the area under the flat jacks. Strain due to loading is measured in boreholes drilled perpendicularly behind the loading plates. Up to five strain gauges known as MPBXs (multiple-position borehole extensometers) are installed in both boreholes to monitor strain during loading (Fig. 1). Setup and testing methods suggested by Coulson (1979) for the plate test are summarized below.

Setup The tunnel area for testing should be cleaned off of loose rocks. Boreholes that are diagonally opposite from each other should be drilled as deep as 10 m. The drilled cores should be carefully logged. The MPBXs should be placed inside the boreholes to measure the anticipated deformation. The flat jacks will be pressurized to apply stress over the top of the boreholes (Fig. 1). The area between the boreholes and the flat jack should be covered with concrete. Wood or resin should be placed as filler between the flat jacks and the steel plates on top of the load-transferring restraint columns.

Testing Loading is applied cyclically. Each loading should be followed by a 24 h period of zero pressure. Deformation measurements from the installed MPBX instruments should be continuously recorded. The duration of loading, maximum test pressure, and number of loading increments are dependent on the type of project. Deformation is calculated based on distance between the flat jack and the depth of the displacement sensor, load, and radius of loaded area, Poisson's ratio, and Young's modulus. The deformation modulus (E_d)



Rock Field Tests, Fig. 2 Cutting a slot to insert a flat jack

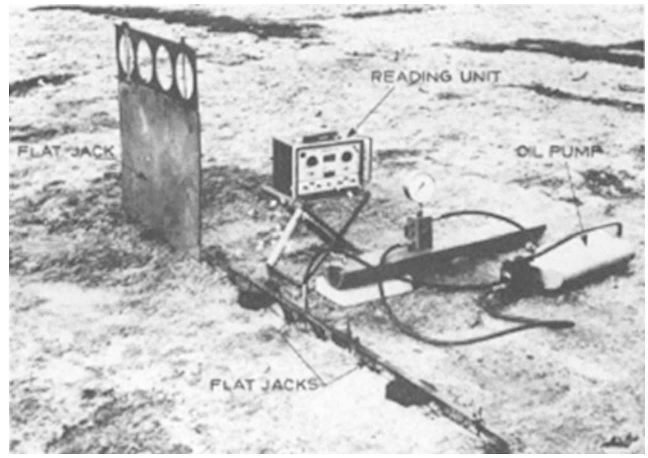
for the rock mass between two MPBXs at depths $z1$ and $z2$ behind the flat jacks is given by:

$$E_d = q\ (K_{z1} - K_{z2}/W_{z1} - W_{z2}),\ \ K_z = W_z(E/q)$$

where q is pressure applied, W_z is displacement in the direction of applied pressure, and E is Young's modulus.

Deformability: Large Flat Jack

This test is intended to measure *in situ* deformability of rock mass by inserting flat jacks into slots cut into rock using a rock saw or a series of overlapping drilled holes (Loureiro-Pinto 1986) (Figs. 2 and 3). The test can be performed on up to four coplanar slots simultaneously. Each flat jack consists of two steel sheets less than 1 mm thick, welded around the edges to be inflated by hydraulic fluid. Deformation is measured by measuring displacements at various places perpendicular to the slots. If the slot is made by line drilling, the semicircular gaps should be filled with concrete. Setup and testing methods suggested by Loureiro-Pinto (1986) for the test are summarized below.



Rock Field Tests, Fig. 3 Inserting a large flat jack

Setup Test locations should preferably be in zones which will be affected by the intended work with due consideration given to the direction of anticipated maximum compressive stress. At least two coplanar slots should be used. Flat surfaces perpendicular to the chosen jack positions should be prepared inside an underground structure such as an adit or tunnel or on surface. The flat surface may be lined by concrete to aid in the installation of the cutting machine that should carefully be operated in order to avoid deviations. The slot cut by a rotary saw or line drilling should be smooth (+/−5 mm) and have a width between 5 and 10 mm larger than the flat jack.

Testing The deformation gauges (deformeters) must be calibrated before testing. Three loading/unloading cycles should be used. The test pressure should be at a minimum of 0.2 MPa and maximum of 120–150% of the maximum loading due to the proposed structure. Each loading should be performed at constant increments to permit accurate plotting of pressure and deformation. The variation in applied pressure should not vary by more than 2%. The modulus of deformation (E) is calculated as follows:

$$E = k(1 - \nu 2)\ p/d$$

where $p =$ increment of applied pressure

$d =$ increment in slot opening corresponding to increment in pressure

$\nu =$ Poisson's ratio

$k =$ coefficient depending on stiffness, shape of flat jacks, location of measuring point, shape of the test chamber, and depth of crack that formed due to loading

Deformability: Radial Jacking Test

The radial jacking test is to measure deformability of rocks due to radial loading. The test is done in circular openings such as adits and tunnels. The load is uniformly distributed radially, and subsequent diametrical displacement within the ground opening is measured (Coulson 1979). Setup and testing methods suggested by Coulson (1979) for the test are summarized below.

Setup The test chamber is excavated to the required dimension and shotcreted. The geology, lithology and structural condition, and orientation of discontinuities should be documented. Holes to install extensometers should be drilled. Loading is done by placing flat jacks over the shotcrete surface. The flat jacks are placed on top of rigid steel rings that are attached to a frame of sufficient strength to resist movement into the opening. Wood planks are placed between the steel rings and the flat jacks. The setup allows the flat jacks exerting load only on both the steel rings and the shotcrete surface, but movement into the opening is resisted by the rigid rings. Deformation of the shotcrete surface can be measured by the extensometers anchored diametrically across the tunnel/adit opening. Multiple extensometers can be used, and the recorded displacement should be in reference to anchors placed well away from the zone of loading.

Testing Three loading and unloading cycles are recommended. For each cycle, pressure should be increased at the rate of 0.05 MPa/min. The displacement should be recorded until 80% of the anticipated displacement has been recorded. Each loading is followed by unloading to near zero pressure. The elastic moduli (E) and deformation moduli are given by:

$$E = p_2 r_2 / \Delta_e (m + 1/m)$$

$$V = p_2 r_2 / \Delta_t (m + 1/m)$$

where p_2 = pressure just below the shotcrete lining at radius of r_2

Δ_e = elastic displacement

Δ_t = total displacement

m = estimated Poisson's ratio

Deformability: Downhole Plate Test

This test is intended to measure *in situ* deformability of rock mass by applying perpendicular stress to a flattened borehole end and measuring displacements. The method allows measuring deformability at different depths with the primary loading axis coinciding with the borehole axis. The displacement due to loading should be measured. Setup and testing methods suggested by Coulson (1979) for the test are summarized below.

Setup The drilled hole for testing should at least have a diameter of 500 mm. The borehole end should be made flat (+−5 mm) and perpendicular to the drill axis (+−3 °). A circular loading plate of ~500 mm should be lowered to the borehole end. Casing may be necessary to stabilize the borehole as well as lowering the water table. A loading column to transmit force onto the loading plate should be assembled. The vertical displacement due to loading should be measured with respect to references placed on ground surface at greater than ten test borehole diameters.

Testing The range of loading is recommended to be within 0.3–1.5 q_0. q_0 is the stress due to the proposed structure. Three loading cycles with each loading increased equally over five increments are performed. The resulting displacement for each loading increment as a function of time should be recorded. If testing for deeper horizons is desired, the equipment should be removed, drilling should continue to the test level, and the borehole end should be prepared for another test. The deformation modulus is calculated as follows:

$$E = dq/d\rho \frac{\pi}{4} \, . D \left(1 - v^2\right) I_c$$

where q = applied pressure

ρ= settlement

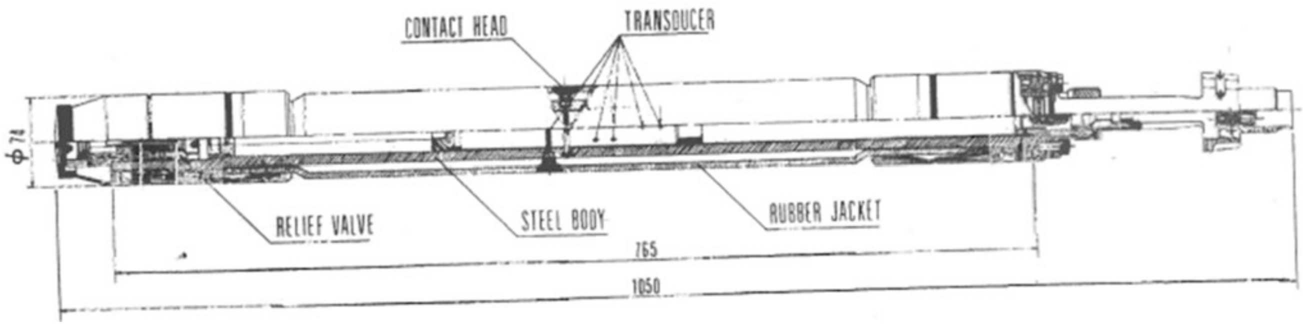
v = Poisson's ratio

I_c = depth correction factor

Deformability: Downhole Flexible Dilatometer

This test is a downhole measurement of deformability using an inflatable cylindrical flexible membrane placed within a borehole at desired depths (Ladanyi 1987) (Fig. 4). Deformation is measured as a function of volume change of the membrane when it is inflated and pushes against the borehole wall. The other method of measuring deformation is by direct radial displacement measurements using transducers installed inside the dilatometer. This method allows deformability measurement in any direction, thereby characterizing anisotropy. Deformability using the flexible dilatometer can be measured at different depths in a drill hole. The measurement is however limited to the horizontal axis or perpendicular to the drill axis. The volume of rock to be tested is usually very small compared to radial or flat jacking methods. Results should be size and orientation adjusted. Setup and testing methods suggested by Ladanyi (1987) for the test are summarized below.

Setup Rotary diamond coring is required to provide smooth walls for testing. Casing may be needed to stabilize the borehole outside the testing zone. The test section should be checked with a downhole camera or a diameter gauge if there is any obstruction for the dilatometer probe. Testing depth interval could be at regular spacings or at selected sites with certain geologic attributes. The stiffness of the system should be calibrated to correct pressure and volume measurement.



Rock Field Tests, Fig. 4 LNEC-type dilatometer

Testing The dilatometer probe is inserted at the test section. The probe is inflated until the membrane is in full contact with the borehole wall. Pressure is then to be increased incrementally to the maximum value. At the end of each increment, the dilation in terms of volume change (calculated in terms of pump fluid used) and the applied pressure should be recorded with respect to time for about 10 min. Up to three loading and unloading cycles are required. If the probe is equipped with radial displacement-measuring transducer, displacement can directly be measured at different orientations and recorded with respect to time. The deformation modulus (E_d) using just volume change is calculated as follows:

$$E_d = 2(1 + V_R)\, G_d, G_d = M_R\left(\pi L a^2/\alpha\right)\,[(1 + B_c(1 - 2v_R)(/(1 - B_c]$$

where G_d = deformation modulus
α = pump constant (fluid volume displaced per turn)
L = length of cell membrane
a = inside radius of cylinder
b = outside radius of cylinder
$B_c = (a/b)^2$
V_R = Poisson's ratio

The deformation modulus (E_d) using radial displacement measurements is given by:

$$E_d = (1 + V_R)\, D\, \Delta p/\Delta D$$

where D = drill hole diameter and $\Delta p/\Delta D$ is the slope of change in pressure with respect to radial displacement.

Deformability: Downhole Stiff Dilatometer

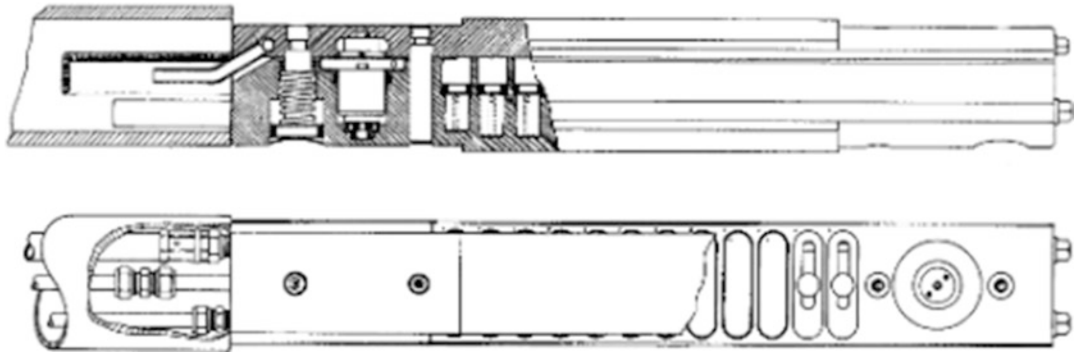
The stiff dilatometer measures deformability using curved loading platens that can exert pressure at different orientations on the borehole wall and measure displacement (Fig. 5) (Yow 1996). The probe can be lowered to any desired depth within the borehole. It can also be rotated to measure deformability at different orientations. Highly fractured or weak rock masses can be problematic for the test. The stiff dilatometer suffers the same drawbacks as the flexible dilatometers in regard to the very small volume of rock that can be tested. Setup and testing methods suggested by Yow (1996) for the test are summarized below.

Setup The dilatometer displacement-measuring devices, linearly variable differential transformers (LVDTs), should be calibrated so that displacement readings read zero displacement at the borehole-drilled diameter. The borehole to be tested should be logged and location/orientation for test should be specified. The borehole should be checked carefully for irregular wall surfaces and varying diameters.

Testing The dilatometer can be lowered to any desired depth for testing. After each test, the dilatometer can be retracted and moved to a different location. It is best to start the test from the lowest most location and continue testing upward to the collar of the borehole. This avoids rock failure obstacles during testing that might affect the movement of the dilatometer inside the borehole. Once the dilatometer reaches the desired depth, the pressure on the platens can be increased until they touch the borehole wall. The displacement reading should be recorded and ideally with zero displacement. Loading can then continue in equal increments and corresponding displacement readings be recorded. Once the maximum pressure has been reached, the pressure should be allowed to dissipate in decrements that correspond to the increments. Displacement values should be recorded during unloading as well. Multiple loading/unloading cycles can be performed. Time-dependent deformability can be tested by maintaining the maximum pressure for an extended time and measuring displacement regularly. The next testing location should at least be 30.5 cm away from a previous test location. The modulus of deformation (E) can be calculated as follows:

$$E = 0.86^{*}0.93^{*}\Delta Q_h (D/\Delta D)\, T$$

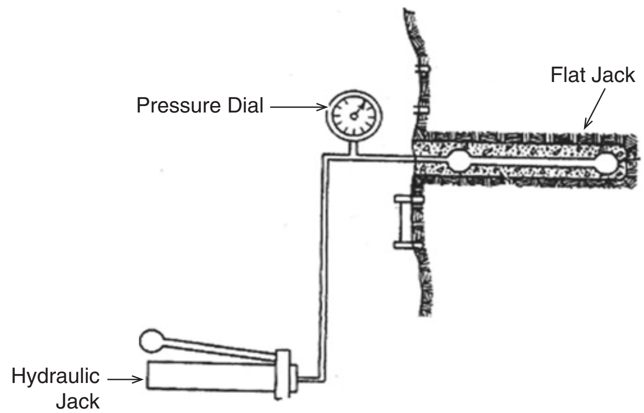
where D = borehole diameter
ΔD = change in borehole diameter
ΔQ_h = pressure increment
T = coefficient depending on Poisson's ratio



Rock Field Tests, Fig. 5 Line drawing showing a flexible dilatometer

In Situ Stress Test

Stress on underground rock mass generally increases with depth but is affected by geologic structures, tectonic forces, and residual stress from paleotectonics. The natural state of stress is termed an *in situ* stress, which can be much higher than the rock mass strength at a specific site causing problems to the stability of underground openings. Therefore, determining the *in situ* stress is essential for designing underground structures and foundations. The suggested methods for determining *in situ* stress include using the flat jack, overcoring, and hydraulic fracturing techniques.



Rock Field Tests, Fig. 6 Cross-sectional view of a flat jack placed in a slot

In Situ Stress Test: Flat Jack

The flat jack method involves inserting pressure-expandable steel sheets welded along their edges into a slot cut into a rock mass. It is ideally installed in underground openings that are wide enough to allow installation (Fig. 6) (Kim and Franklin 1987). The flat jack is connected to a hydraulic pump. Displacement perpendicular to the flat jack is measured with reference to pairs of pins grouted into the rock on either side of the flat jack. Each measurement determines the state of stress perpendicular to the flat jack, and therefore, multiple orientations of flat jacks may be used to get a complete picture of *in situ* stress (Fig. 7). Setup and testing methods suggested by Kim and Franklin (1987) for the test are summarized below.

Setup The flat jack which is at least 0.1 m^2 has one inlet for the pressurizing fluid and another for bleeding. A rotary rock saw is needed to create a slot to install the flat jack. A series of overlapping boreholes can also be used as slot, but the space between the flat jack and rock needs to be grouted. Displacement measuring pins anchored symmetrically on either side of the flat jack should be around 12 mm in diameter and 150 mm in length. A minimum of six setups at different orientations are required. The site of flat jack installation should be cleared off of loose materials and should be flat. The distance between test sites should be at least three times the length of the flat jack.

Testing Pressure- and displacement-measuring devices should be calibrated. Distance measurement between each pair of the pins should be taken before the slot is cut. Another set of distance measurements should be taken immediately after the slot is made to capture the amount of slot closure. The flat jack is then inserted and grouted so that it will be held in place. Pressure into the flat jack is increased until the separation between the pins is the same as it was before the slot was cut. This pressure is termed the cancelation pressure. Readings of the pin separation are recorded during the pressure increment stage. The *in situ* stress perpendicular to the flat jack is approximately within 5% of the cancelation pressure.

In Situ Stress Test: Overcoring

The method of overcoring is used to determine *in situ* stress in a borehole. The technique involves inserting a probe with

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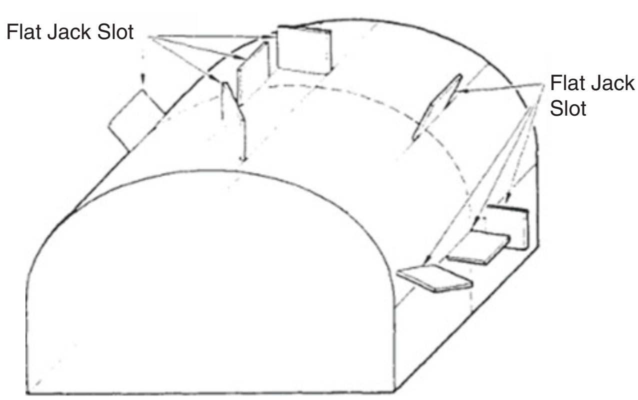
strain gauges bonded to the inside wall of a borehole called a pilot hole. The pilot hole is then drilled again (overcored) by a larger diameter borehole relieving the stress experienced by the probe inside (Fig. 8) (Sjöberg et al. 2003). As the overcoring advances and the pilot hole experiences a relief in stress, the strain gauges in the pilot hole respond by extending outward proportional to the *in situ* stress and elastic property of the rock. If the elastic properties of the rock are known from lab tests, the strain recovery (difference in strain before and after overcoring) as a result of overcoring can be related with *in situ* stress. Various types of probes, the Borre probe, CCBO, CSIR, and USBM, have been described by Sjöberg et al. (2003); Sugawara and Obara (1999); Kim and Franklin (1987).

Setup Before testing, calibration of strain gauges and cleaning of the borehole should be accomplished. The depth of testing, where the pilot hole should be located, is specified in advance. Drilling is advanced to the top of the specified zone of testing, and a smaller pilot hole (~50% of the original hole) is then drilled. The pilot hole core is analyzed for its homogeneity and the presence of open fractures. The ideal zone of testing should be where the rock is homogenous and free of open fractures. If the rock quality is unacceptable, drilling at the normal diameter should advance further. If the rock quality of the pilot hole core is found acceptable, the pilot hole should be cleaned by flushing water downhole before installing the probe.



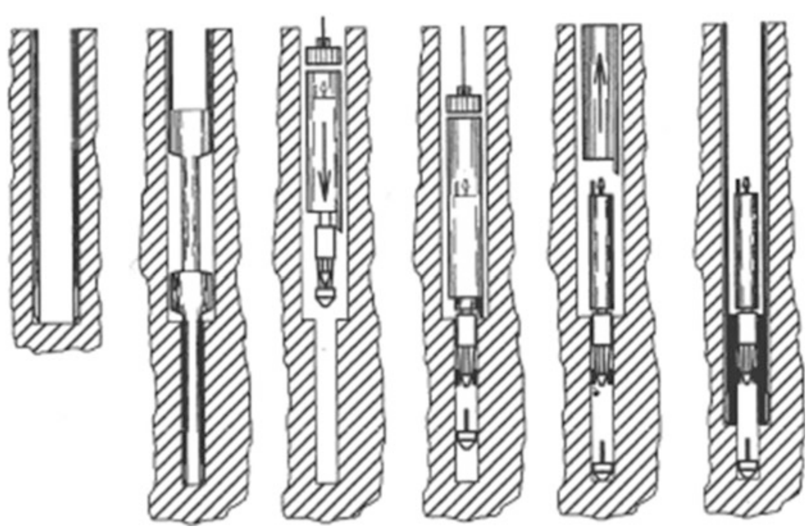
Rock Field Tests, Fig. 7 Layout of flat jack slots

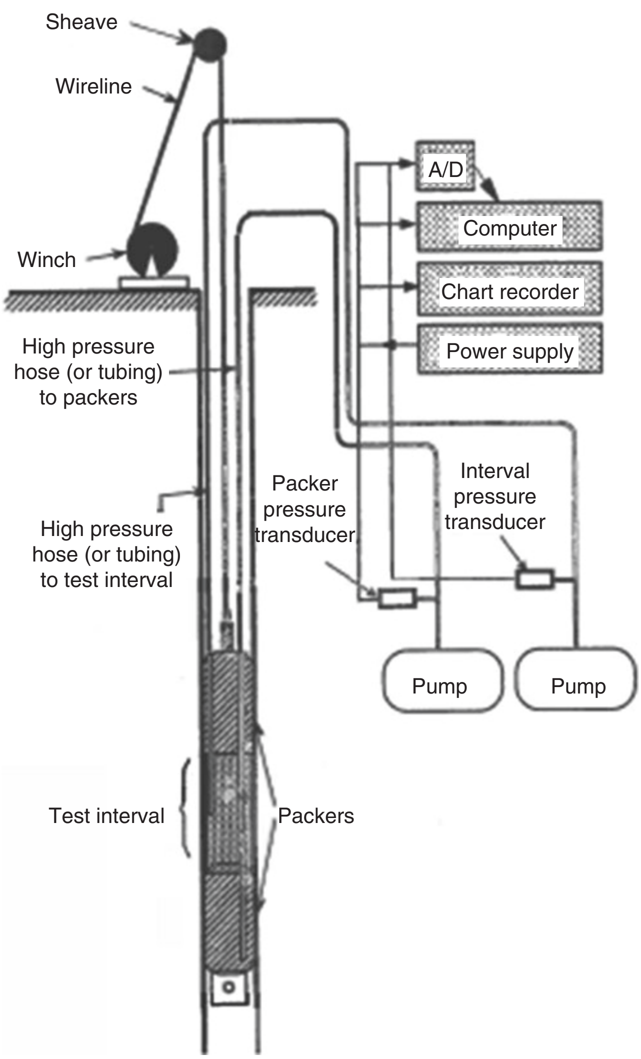
Testing For the testing to begin, a probe with strain gauges is installed inside the pilot hole. The strain gauges can be oriented perpendicular, parallel, and at 45 ° to the borehole axis and are glued to the pilot hole wall. Once the glue has hardened, overcoring can begin. The overcored section is broken off at the base and brought to surface to record length of sample, lithology, rock fabric, and uniformity of thickness. After the probe has been removed from the sample, the hollow core with a minimum length of 24 cm is subject to biaxial loading to determine Young's modulus and Poisson's ratio. A three-dimensional stress tensor is calculated based on multidirectional strain data from the strain gauges, orientation of the borehole, and elastic constants of the rock assuming that the rock is homogenous, isotropic, and linear elastic. For details of calculation, the reader is referred to Sjöberg et al. (2003).

In Situ Stress Test: Hydraulic Fracturing

The hydraulic fracturing method of determining *in situ* stress is based on the relationship between the fluid pressure needed to open new fractures or reopen existing fractures, rock property, and *in situ* stress. The zone to be tested is blocked from the rest of the borehole by placing inflated rubber packers on top and bottom to block vertical escape of the hydraulic fluid (Fig. 9) (Haimson and Cornet 2003). Once the packers are inflated and the test zone is securely

Rock Field Tests, Fig. 8 Installation procedure for overcoring starting with drilling to the test level (*leftmost*), drilling a pilot hole and installing the probe inside the pilot hole, and finally overcoring around the pilot hole (*rightmost*)





Rock Field Tests, Fig. 9 Hydraulic fracturing test equipment setup

sealed, fluid pressure is raised until a new fracture opens or a pre-existing fracture reopens. Pumping is stopped and pressure is allowed to decay. Cycles of raising pressure to the point of fracture reactivation and subsequent decay can be repeated. The fracture orientation before and after the test is captured using an impression packer or downhole geophysical methods.

Setup After selecting the zone of testing, the straddle packers are placed leaving a zone six times the borehole diameter in between. For tests relying on generating new fractures, the selected test zones should be devoid of fractures. The packers are inflated by a pump on surface or an attached pump controlled remotely. The hydraulic fluid is transferred from surface through high-pressure tubing to the test zone. Pressure gauges on surface are used to monitor real-time pressure change. Hydraulic pumps capable of generating up to 100 MPa at a flow rate of up to 10 liter/minute are needed. Oriented impression packer which has an outer layer of soft semi-cured rubber is inflated inside the zone of testing to capture the orientation of the fractures as imprints on its surface. Other geophysical tools such as borehole cameras or electrical imaging systems can be used to obtain fracture orientations from the test zone.

Testing Testing can be based on opening a new fracture or reopening existing fractures that have multiple orientations. The test zone's intrinsic permeability may be evaluated by an initial pressurized slug test. The pressure within the testing interval is raised by maintaining a constant flow rate until it reaches a point where a new fracture opened or pre-existing fractures reactivated. This pressure is termed the breakdown pressure. Pressurization can also be applied in a stepwise manner where flow rate varies and the maximum pressure for each flow rate is maintained for about 5 min. After the breakdown pressure is reached, pumping is stopped without venting the pump, and decay in pressure is monitored in real time until the fractures are closed reaching the shut-in pressure. The pump is vented after about 10 min of reaching the shut-in pressure. Cycles of repressurizing and decay may continue. The methods of *in situ* stress calculations vary depending on the test type, opening new fracture or reopening existing fractures. For cases where new fractures within 15 ° to the borehole axis, the least horizontal stress axis (σ_h) is equal to the shut-in pressure, and its direction is normal to the new hydraulically induced fracture. The maximum horizontal stress (σ_H) is given by:

$\sigma_H = T + 3(\sigma_h - P_0) - (P_b - P_0) - P_0$, where T is rock tensile strength, P_b is the breakdown pressure, and P_0 is pore pressure.

The direction of σ_h is perpendicular to σ_h.

If the test was performed on pre-existing fractures, the normal stress supported by the fracture ($\sigma_m{}^n$) is given by:

$$\sigma^m{}_n = \sigma\,(X_m)\,n_m n_m$$

where X_m is the location of the mth test, $\sigma^m{}_n$ is the measured normal stress supported by the fracture plane with normal n_m, and $\sigma\,(X_m)$ is the stress tensor at X_m. For details of calculation, the reader is referred to Haimson and Cornet (2003).

Summary

Deformability and *in situ* stress of rock masses are affected by discontinuities in rock and tectonic (paleo or active) stress. Therefore, it is necessary to devise field-testing methods that have been discussed above. Rock field tests can be performed inside underground openings, inside boreholes, and on the ground surface by applying stress using hydraulic jacks and monitoring displacements. Deformability tests inside underground openings or on ground surface include the plate test, radial jacking, and the large flat jack, whereas the downhole plate test and flexible/stiff dilatometers can be used inside a borehole.

Designing underground structures heavily relies on accurate determination of *in situ* stress in addition to rock deformability. *In situ* stress is mainly a function of depth, but tectonic forces (active or paleo) can also affect the state of stress. Three methods, the flat jack, overcoring, and hydraulic fracturing, are discussed above. The flat jack method is performed by relieving *in situ* stress by opening a slot into rock and applying induced stress to reopen the slot to its original width, thus estimating *in situ* stress. Overcoring is performed by installing a probe inside a drilled hole (pilot hole) and advancing a larger diameter core drilling around the pilot hole. As the larger hole advances, the pilot hole experiences a stress relief and deforms as a function of *in situ* stress and its mechanical properties. The principle of using hydraulic fracturing is to use the magnitude of fluid pressure injected into a section of a borehole to reopen existing fractures or create new ones as a proxy for determining *in situ* stress.

Cross-References

▶ Boreholes
▶ Deformation
▶ Extensometer
▶ Hydraulic Fracturing
▶ Jacking Test
▶ Modulus of Deformation
▶ Modulus of Elasticity
▶ Poisson's Ratio
▶ Pore Pressure
▶ Pressure
▶ Shear Modulus
▶ Young's Modulus

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Rock Laboratory Tests

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Definition

Tests performed in the laboratory to determine physical or mechanical properties of intact rocks and rock discontinuities.

Introduction

Rock Laboratory tests are carried out to determine rock properties. Since rock properties are a key input for rock mechanics design in civil, mining, and petroleum engineering, this entry is mainly based on documents by the International Society for Rock Mechanics (ISRM). The ISRM Commission on Testing Methods, since 1974, has published the ISRM Suggested Methods (SMs) covering different aspects of rock mechanics. The SMs are collected in books ("yellow" book, Brown 1981; "blue" book, Ulusay and Hudson 2007; and "orange" book, Ulusay 2015). A complete list of ISRM SMs is included in the "orange" book.

The scope of the SMs is to achieve some degree of standardization of testing procedures and to consider the result of tests, wherever obtained, as reliable as possible. Currently, the ISRM SMs are considered the fundamental documents on which all national standards for rock engineering applications, where they exist, are based.

Another important collection of testing standards, covering the same field of application as the ISRM and more, are produced by the ASTM International (American Society for Testing and Materials).

Rock properties are relevant not only in rock engineering, but also when rock is used as a construction material, such as masonry blocks of new and historical buildings, dimension stones, rock armors for coastal defense, rocks for Earth-structures, etc. Depending on the particular application, tests and related properties can be quite different from those proposed for rock engineering. In this field of application, standards have been formulated by the European Committee for Standardization (EN standards), ASTM International, etc. The description of these tests is beyond the scope of the present entry apart from exceptions of general interest.

Throughout, for each test, reference is made to the standard number if it exists (e.g., ASTM D854 or EN 13383-2) or to the author (e.g., ISRM 1974) and extensively reported in the reference list. In ASTM standards, the year of release is omitted. If two standards do not differ significantly, only one reference is given. Readers who are more acquainted with a different standard, can compare it with that referenced in the text. For the ASTM standards, the reader is addressed to the latest version of the ASTM annual book (ASTM 2015).

As laboratory tests are aimed to measure rock properties and some tests determine more than one property, this entry is divided into subentries, each referring to a rock property rather than a test. This has the added advantage of grouping some tests, so that a tedious list of tests is avoided.

The content of the present entry does not presume to provide detailed instructions on the test procedures or to be exhaustive of the rock laboratory tests. The reader is invited to refer to the above-mentioned ISRM documents. Moreover, not only are rock laboratory tests numerous but the work of the ISRM Commission on Testing Methods is always in progress and further SMs on different tests or updated procedures will be published.

Preparation and Identification of Specimens

Preparation of specimens requires great care to minimize damage and irregularities that introduce artefacts in test results. For most of the tests, cylindrical specimens (ASTM D4543) are required; they are prepared usually in the laboratory by overcoring cores or drilling block samples. Ends of specimens are then squared with a diamond saw and finished through a grinding machine. Some lithotypes, very sensitive to water (e.g., swelling or slaking marls and shales), require special care in specimen preparation.

The minimum size, diameter *D*, or height *H*, of cylindrical specimens for determining strength, deformability, sound velocity, and permeability is related to the size of the largest grain in the rock, whenever possible, by a ratio of at least 10:1, in order to consider the specimen homogeneous with respect to the measured property. The same criterion should apply to the ratio between specimen diameter and macropores.

For most tests, specimens can be tested dry or fully saturated or at their natural moisture content. Drying of specimens may be carried out using a desiccator or a low temperature oven. Particular care is required especially for crystalline rocks that should not reach temperatures over 50 °C, less than the recommended temperature of 105 °C (ISRM 1979a). Otherwise, microcracking could induce damage in specimens intended for mechanical testing. Specimens are saturated in water under vacuum for a given period and kept there up to testing. Detailed procedures are described in ISRM (1979a).

Specimens should be subjected to a preliminary petrographic examination and then grouped into sets of lithologically homogeneous specimens. The homogeneity of the statistical sample is relevant in interpreting the results of the mechanical tests.

For each set, a detailed petrographic description (see e.g., ISRM 1978a) is recommended prior to testing. Macroscopic analysis should regard rock texture (including grain size and anisotropy), approximate mineralogical composition, weathering grade and defects (pores, fissures). For many applications, a microscopy analysis with polarized-light can provide useful information on microtexture, mineralogical composition, microporosity, intimate weathering, etc. Moreover, a synthetic description of each specimen is required, including angle of foliation/lamination and presence of apparent defects, as their orientation with respect to load could influence the investigated rock property.

For joint specimens, a detailed description of filling thickness/type and joint surface conditions is to be provided. The latter includes mineralogical composition, if different from the surrounding rock, weathering and shear features such as polishing and striae.

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Physical Properties

Physical properties include density, porosity, water content, and water absorption of the rock material. Density can be referred to the solid phase (grain density ρ_s) and to the material at its natural water content (bulk density ρ), in dry conditions (dry density ρ_d), and in saturated conditions (saturated density ρ_{sat}).

Grain density ρ_s is calculated by dividing mass (M_s) and volume of powders, the latter being measured through the flask pycnometer (ISRM 1979a and ASTM D854) or, more conveniently, through a helium pycnometer (ASTM D5550). Measurement of grain density requires particular accuracy when the content of heavy minerals (i.e., mafic) is appreciable and variable.

Bulk and dry densities are calculated by dividing the measured total (M) or dry (M_s) mass of a cylindrical specimen, conforming to strict geometrical requirements ISRM (1979a), to its volume V, which is obtained from calliper measurements of height H and diameter D. For a good estimate of ρ_d, many specimens are required to be oven dried at low temperatures.

When cylindrical specimens cannot be easily prepared, especially in weak rocks, volumes of lumps can be measured through the buoyancy method (ISRM 1979a) or the displacement of mercury or microscopic particles (Webb 2001). The former procedure is suitable for rock materials that do not swell or slake when oven dried or immersed in water. The latter two techniques require that the rock material does not have macropores on its external surface volume.

Measurements of ρ_d and ρ_s are particularly important for the determination of porosity n, which can be calculated as $n = 1 - \rho_d/\rho_s$. Porosity could control strength and deformability of intact rocks, especially when high.

Saturated density ρ_{sat} requires measurement of the mass of the cylindrical specimen M_{sat}, saturated by de-aired water under vacuum, and is calculated as M_{sat}/V. Effective porosity (i.e., including only pores saturated by water) can thus be derived as $1 - [(\rho_{sat} - \rho_d)/\rho_w]$, where ρ_w is the density of the water.

Measurement of ρ_d, ρ, and ρ_{sat} is crucial in low-density rocks when dynamic stiffness is derived from sound velocity measurements (see section "Sound Velocity").

Water content w_n is measured as $(M - M_s)/M_s$, according to ISRM (1979a). It is worth noting that on site wax-coating of block samples yields a representative measurement of natural water content, whereas measurement on core samples is altered by water absorption during coring. It is recommended to measure natural water content only on rocks at *in-situ* conditions.

For many applications (e.g., quality assessment of aggregates for earth structures and rip-rap/armour-stone rocks), the absorption coefficient, that is, the mass of water retained by rock pores upon immersion in water divided by the dry mass of the sample, is determined (e.g., according to the EN 13383-2 standard).

Hardness and Abrasiveness

Abrasiveness is a rock property expressing the resistance to wear of the rock material. Hardness is a measure of the extent to which the surface of a solid resists a permanent change of shape obtained in different ways. Therefore, it is a "concept of material behavior" (ISRM 1979b) rather than a property.

Both abrasiveness and hardness depend on the type of minerals and their abundance, as well as on the bond strength between crystals/clasts.

Abrasiveness

The measure of abrasiveness is a major issue in such matters as:

(a) Mechanized tunnelling
(b) Rock drilling
(c) Performance of rock aggregates (from gravelly to blocky) used as armourstone or in highway/railway construction, rockfill dams, etc.

A number of SMs have been proposed to measure the effect of wear, most of which were reviewed by ISRM (1979b) but little has been done since then.

Wear of aggregate rock is usually evaluated through tests where revolving drums contain an assigned amount of rock fragments of prescribed size with or without a charge of steel balls. The former case includes the Los Angeles test (ISRM 1979b) and the Micro-Deval test (e.g., EN1097-1), whereas the latter includes the Deval and Mill Abrasion tests, no longer recommended for rock characterization. In these tests, abrasiveness is expressed as the loss in weight, that is, the weight of the material produced by abrasion passing through the 1.7-mm (or 1.6-mm) sieve, divided by the original weight.

For (a) and (b) applications, the problem can also be viewed as the ability of the rock to abrade cutting/drilling tools.

A different approach proposes to measure the wear of a steel tool simulating the wear of TBM (Tunnel Boring Machines) cutters. The Cerchar test, object of a SM by ISRM (Alber et al. 2014), measures the wear on the conical tip of a steel stylus (conforming to geometrical and hardness specifications) that moves on an irregular rock surface under a given normal force and displacement rate. The test yields the *CAI* index, which is the width of the flat wear surface in mm, multiplied by 10.

Hardness

Hardness tests included in the SMs by ISRM measure the rebound height of a standard "hammer" on a rock surface (Shore scleroscope SS, Schmidt hammer SH) or the indentation load and depth of a tool in the rock specimen. The SS and SH are nondestructive tests that differ for hammer dimension, weight, and drop mode (free fall for the SS and spring release for the SH). Rebound height can be correlated to uniaxial compressive strength (*UCS*) and Young modulus (the latter only with the SH test).

The SS (Altindag and Güney 2006) has a 5.94-mm-diameter hammer and is used on core/cubic specimens with a volume close to (and not less than) 80×10^3 mm^3. Even though many measurements on the whole specimen face are averaged, the reduced hammer diameter seems to restrict its use to rocks that are relatively homogeneous at this small scale.

The SH (Aydın 2009) has a hammer diameter of 15 mm and therefore is less sensitive to small-scale inhomogeneity. This is applicable to rocks in the *UCS* range of 20–150 MPa. Two types of SH instruments are easily available, with different impact energy: L-type (0.735 N·m) and N-type (2.207 N·m). Tests are potentially nondestructive for rocks having *UCS* > 80 MPa. Core and block samples have to conform to specific sizes and are to be clamped to a steel block of prescribed weight and shape (for core samples). The SH is also used on discontinuities to estimate compressive strength and weathering conditions of their surfaces, which affect shear strength of joints.

The indentation test (Szwedzicki 1998) is used to characterize/classify the rock with reference to hardness, especially with respect to drillability and cuttability and when only small rock lumps are available. Indentation hardness can also be correlated to *UCS* and tensile strength. The depth h of a crater produced by penetration of a conical tool, with specified geometry and steel properties, into the saw cut rock surface is recorded together with the applied load P. The indentation hardness index *IHI* [kN/mm] is calculated as P/h.

Durability

Rock durability is the resistance offered by a sample to weakening and disintegrating under repeated changes in environmental conditions. Durability is typically related to moisture and temperature changes (i.e., drying and wetting) but also to freezing, thawing and salt crystallization. This property is relevant for rockfill structures (e.g., dams, road embankments), rocks in marine environment, and building/dimension stones.

ISRM (1979a) edited a SM on the slake-durability test (Franklin and Chandra 1972), which is intended to evaluate the resistance of rock samples to slaking when subjected to cycles of drying and wetting.

The equipment consists of a drum made of a 2-mm square wire mesh which can resist temperatures up to 105 °C and can be immersed in water.

A representative sample of ten rounded (roughly spherical in shape) lumps with a mass in between 40 and 60 g is placed in the drum. After an initial drying in the oven until a constant mass is reached, the total mass of the drum plus rock lumps is measured.

The drum is then placed in rotation, through a motor drive at a constant revolution speed, partially submerged in a slaking fluid, for 10 min. The residual total mass is dried to a constant mass and then measured. A second cycle of wetting and drying is run and masses are measured. The drum mass is also measured to obtain the net mass of the rock lumps.

The second cycle slake-durability index is calculated as the net mass after the second cycle of wetting and drying over that after the initial drying. If successive cycles are operated, durability indexes can be calculated at the end of each cycle.

The slaking fluid can be tap or sea-water, or any fluid of interest for the final design.

Slake-durability is dependent on mineralogical composition. It is measured especially in rocks containing clay minerals. This test does not exclude the possibility of slaking in rocks with significant clay content subjected to prolonged wetting.

Strength of Intact Rock

Strength is one of the most significant mechanical properties of rock materials, as it is the capacity of sustaining loads, which is a major issue in all engineering applications. Rock materials fail under deviatoric stresses (uniaxial and triaxial compressive tests) and tensile stresses (direct and indirect tensile tests, point load test, beam bending tests). Weak rocks can yield under hydrostatic stresses (triaxial compressive tests), but their capacity of sustaining loads increases at increasing strains.

A common occurrence in the various tests is the decrease in strength as the size of the tested rock specimen increases. This is due primarily to the fact that the larger the specimen, the higher the probability of encountering defects, which initiate failure. As a result, special care is required in choosing specimen size.

Uniaxial Compressive Strength

The uniaxial compression test, the simplest and most common test among the laboratory tests, is mainly aimed to determine the uniaxial compressive strength (*UCS*).

The internationally recognized procedure for carrying out the test devoted to the measurement of uniaxial compressive strength and deformability is provided by ISRM (1979c). This SM was complemented by a successive SM (Fairhurst and Hudson 1999), which is devoted to determine the stress-strain behavior of intact rock in uniaxial compression. ASTM (ASTM D2938), too, recommends a standard procedure.

The measured uniaxial compressive strength is mainly intended to characterize the intact rock for engineering purposes. As *UCS* can be considered the most important index property, the test is also useful for strength classification (see "Rock Properties").

Specimens, in the shape of right circular cylinders, must conform to strict geometrical requirements.

In order to reduce the effect of restraint at the specimen ends, some precautions have been devised. Both specimen ends are confined by steel platens in the form of discs having a diameter not dissimilar to the diameter of the specimen. Test specimens are quite slender, having a height to diameter ratio (*H*/*D*) of 2.0–3.0; moreover, this high value allows the complete development of shear planes through the specimen volume.

To ensure uniform stress, specimen ends have to be flat. Moreover, a spherically seated upper loading platen is used to reduce the effect of oblique specimen ends on test results.

Unjacketed specimens are compressed, under a loading machine, parallel to their longitudinal axis in unconfined stress conditions. This loading condition leads specimen to failure under deviatoric stresses. The compressive strength is the peak stress, calculated as the maximum compressive force sustained by the specimen over its initial cross-sectional area.

As the compressive strength of rocks is quite variable, a minimum number of five specimens is recommended.

Uniaxial compressive strength is very sensitive to the water content of the specimen, similarly to other rock properties. Generally, rock strength in dry conditions is higher than that in saturated conditions. Thus, specimens have to be tested at the same water content, chosen as appropriate to the design for which the test data are required.

For a comprehensive review of the uniaxial compression test, see Hawkes and Mellor (1970).

Tensile Strength

Tensile strength is usually measured in uniaxial (direct) or diametral compression (indirect) tests on cylindrical specimens. Conceptual and experimental issues are reviewed by Mellor and Hawkes (1971) and Perras and Diederichs (2014). The testing procedure is reported in detail by ISRM (1978c).

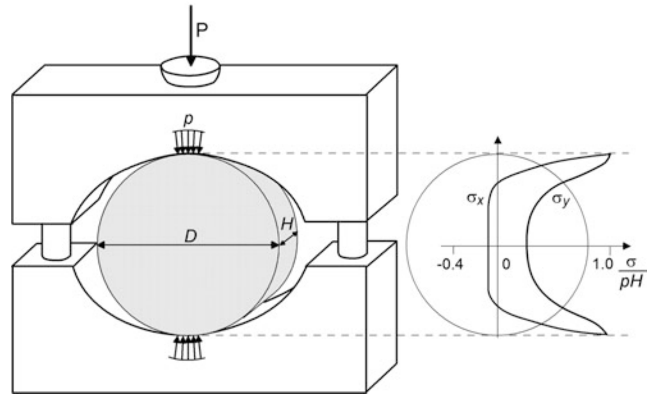
In direct tests, the specimen ends have a height to diameter ratio (*H/D*) not lower than 2.5. Their ends are either cemented to the load platens (ISRM 1978c) or clamped through opposite devices connected to the loading machine (e.g., Gorski 1993). The latter set-up is better suited to rocks of medium to high strength. The tensile strength *TS* is equal to the axial load at failure divided by the cross-sectional area of the specimen.

In the indirect test, also known as the Brazilian test, diametral load is applied through two curvilinear loading jaws that increase the contact area, thus reducing contact stresses and hence avoiding local ruptures. The ratio *H/D* of the specimen, according to ISRM (1978c), is set to 0.5.

On the diameter plane aligned with the compressive load, the state of stress at failure, calculated from the elasticity theory in plane strain conditions, is biaxial. It can be considered to be constant along a wide zone across the specimen center (Fig. 1) and equal to (tensile stresses are negative according to the Geotechnical Engineering convention):

$$\sigma_x = -\frac{2P_{\max}}{\pi DH}; \quad \sigma_y \sim -3\sigma_x \tag{1}$$

where $P_{\max}$ is the load at failure. The Brazilian tensile strength *BTS* is equal to the absolute value of the tensile stress σ_x at failure.



Rock Laboratory Tests, Fig. 1 Scheme of indirect (Brazilian) test. On the right, the distribution of horizontal σ_x and vertical σ_y stresses is shown, where *p* is the distributed load acting on the contact area

Since the state of stress acting in many engineering problems is multiaxial and the experimental set-up of the indirect test is much easier than that of the direct test (e.g., preparation of specimens for clamping or effectiveness of platen-specimen bonding), indirect tests are extensively performed in the practice.

On the other hand, tensile strength from indirect tests is often overestimated (Perras and Diederichs 2014) because failure develops along an imposed surface (not necessarily the weakest).

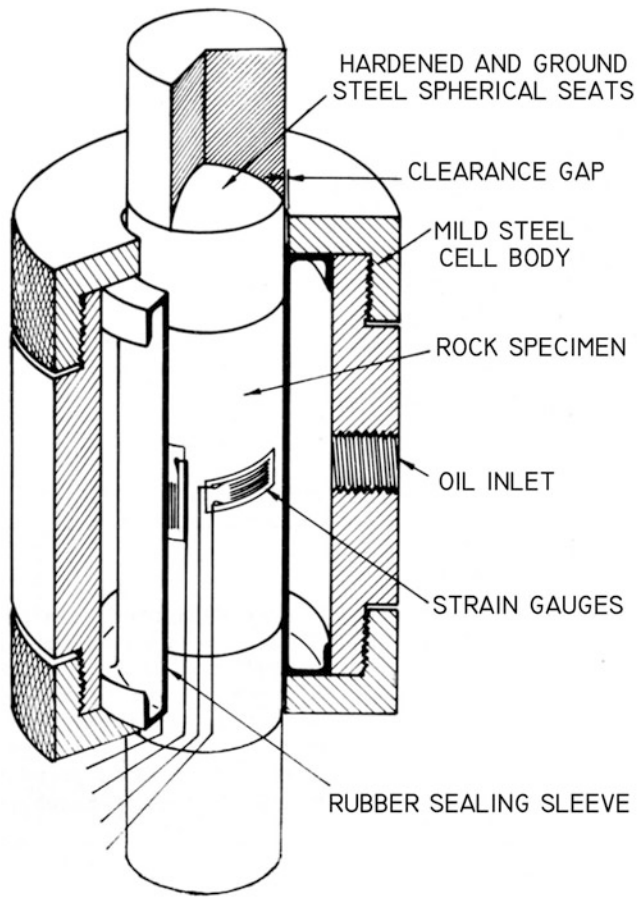
For dimension stones, a beam-bending test is commonly carried out. In the flexural strength test, the specimen, in the shape of a slab, is loaded according to a quarter-point loading configuration, along two lines, each 25% of the span from each support.

Triaxial Compressive Strength

Knowledge of the variation in strength of intact rocks under a confining state of stress is fundamental in many applications in rock engineering. The most common state of stress applied in triaxial tests is cylindrical, with the principal stresses under the condition $\sigma_1 > \sigma_2 = \sigma_3$. This is obtained through triaxial tests on cylindrical specimens, which provide data for determining the shape of the strength envelope and the parameters of the strength criterion of the intact rock. Detailed testing procedures are reported by ISRM (Kovari et al. 1983).

Specimens, which must conform to the same requirements of uniaxial tests, are tested in high-pressure cells. Figure 2 reports the schematic view of a common triaxial cell used in many laboratories (Hoek cell), where the axial and radial stress are applied independently.

The cell containing the specimen is placed under a loading machine. Typically a radial confining pressure $\sigma_r = \sigma_2 = \sigma_3$ is applied by pressurized oil acting on a thick rubber membrane surrounding the specimen (Franklin and Hoek 1970). The axial stress $\sigma_a = \sigma_1$ is equal to the axial load applied at the



Rock Laboratory Tests, Fig. 2 Triaxial test cell (Franklin and Hoek 1970, modified)

specimen ends divided by the cross sectional area of the specimen.

The most common test is the "individual test." The specimen is initially subjected to an isotropic stress state $\sigma_r = \sigma_a = p$. Successively, a deviatoric stress is applied by increasing σ_1 until failure, which determines the peak strength ($\sigma_{1,peak}$). When a stiff or a servo-controlled loading machine is used, after peak, axial strains are increased until σ_1 stabilizes; the related stress is the residual strength ($\sigma_{1,res}$).

Experimental values of $\sigma_{1,peak}$ (and $\sigma_{1,res}$ when available) obtained at different values of the radial stress p, together with *UCS* from uniaxial tests, are plotted on the plane of the principal stresses (Fig. 3). Strength data are then enveloped with different models according to the selected failure criterion.

Multiple or continuous-failure tests can also be performed (Kovari et al. 1983) to obtain further information on strength behavior from a single specimen of brittle rock. The procedure requires that the load is applied with a stiff or a servo-controlled testing machine and that axial strain is thoroughly measured. In multiple-failure tests, a single specimen is tested at different (increasing) isotropic stresses p_i, until peak strength $\sigma_{1,peak,i}$ for each value of p_i is reached. Damage due to successive failures is assumed to be significantly lower than that due to the increment in strength at the higher radial stresses. Successively, the strength envelope in the desired range of confining stresses is obtained by interpolating pairs $p_i - \sigma_{1,peak,i}$.

In hard rocks, strains are measured through electric strain gauges glued to the specimen (Fig. 2). The positions of the strain gauges must prevent damage to electric wires connecting strain gauges to the data logger. In soft rocks, external displacement transducers mounted on the load platens can be used for the measurement of the sole axial strains. Axial displacements at the platen-specimen joint and between the spherical seat halves have to be estimated and subtracted from measured values.

Testing procedures are designed for dry specimens. Moisture effects can be accounted for by preparing sets of specimens at different moisture content and running tests at different confining pressures on each set.

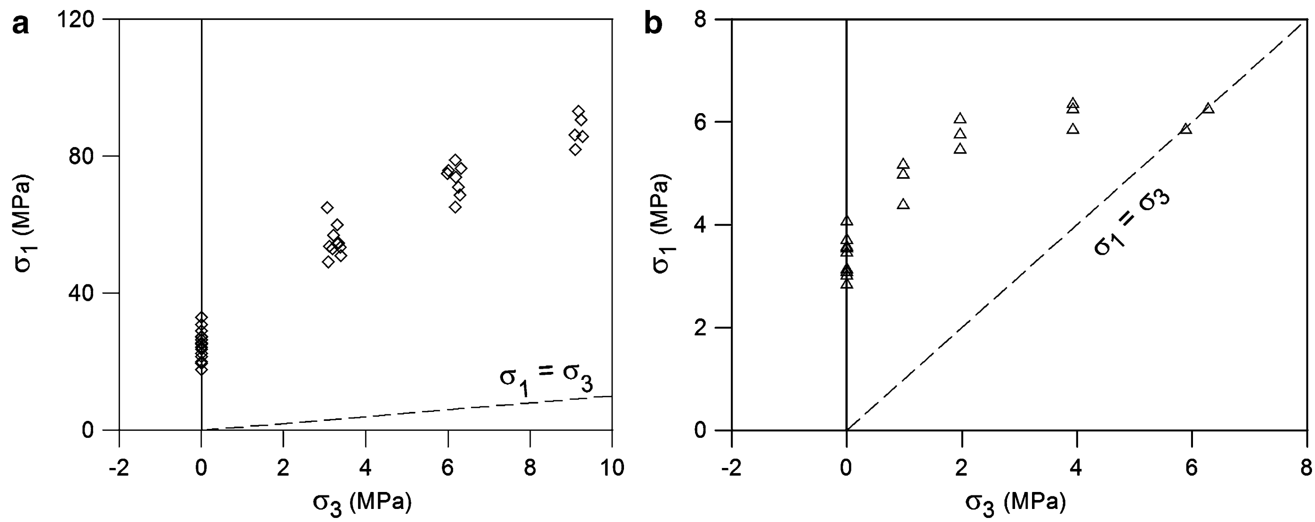
Point Load Strength

The point load strength test (Franklin 1985) measures the resistance of a specimen compressed between two conical platens yielding a strength index (*PLI*) of intact rocks. The *PLI* index is used for the characterization of rock materials and for the classification of rocks, similar to the uniaxial compressive strength. The test also measures a strength anisotropy index of the rock, which is the ratio of *PLI*s obtained in the directions of the greatest and the least values. The test is not recommended for weak rocks, where *UCS* is less than approximately some tenths of MPa.

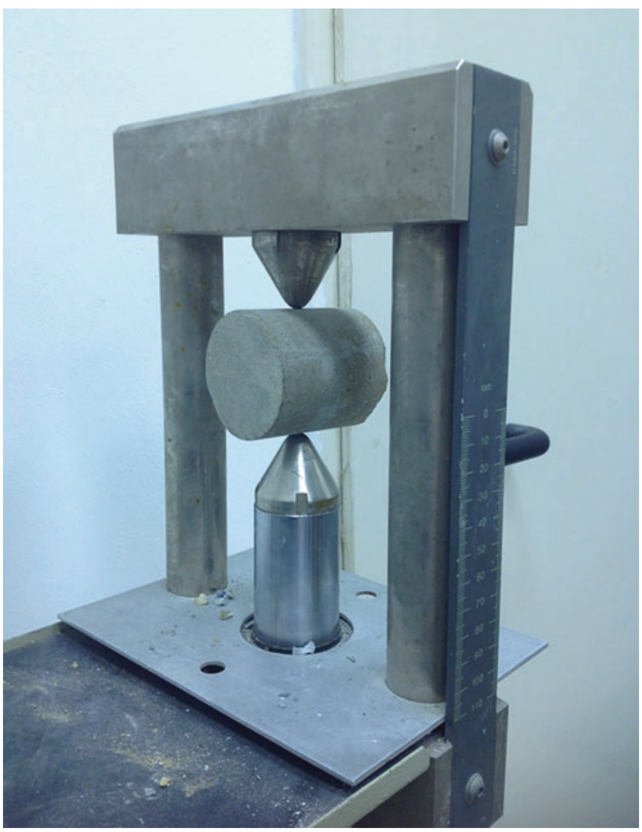
Similarly to the Brazilian test, the test induces tensile failure under the application of a compressive loading. The loading system is not required to have a high capacity, compared to that required to break a specimen under a compressive state of stress, and therefore the testing machine can be also portable (Fig. 4). This feature together with the little or no specimen preparation indicates that the test was originally intended to obtain a strength index for intact rocks directly on site (Broch and Franklin 1972).

The specimen, in the form of core, small block, or irregular lump, is broken by applying a compressive load through a pair of spherically truncated, conical platens, with specified shape and material. On core specimens, load is applied along a diameter (diametral test) or along the axis of the cylindrical specimen (axial test).

The point load strength index (*PLI*) is calculated as the maximum compressive force P_{max} over the squared "equivalent core diameter" D_e, which is the core diameter for a diametral test. For axial, block, and lump tests, the equivalent core diameter is derived from the minimum cross-sectional area of a plane through the platen contact points.



Rock Laboratory Tests, Fig. 3 Results of uniaxial and triaxial tests: (**a**) hard brittle rock; (**b**) soft rock. Note the different scales of the two plots



Rock Laboratory Tests, Fig. 4 Point load strength test (diametral test)

The influence of the specimen size on rock strength requires to correct the index *PLI* when the equivalent core diameter of the specimen is quite different from the conventional value of 50 mm, very similar to the widespread NX diameter (54 mm). The size-corrected strength index PLI_{50} can be calculated as:

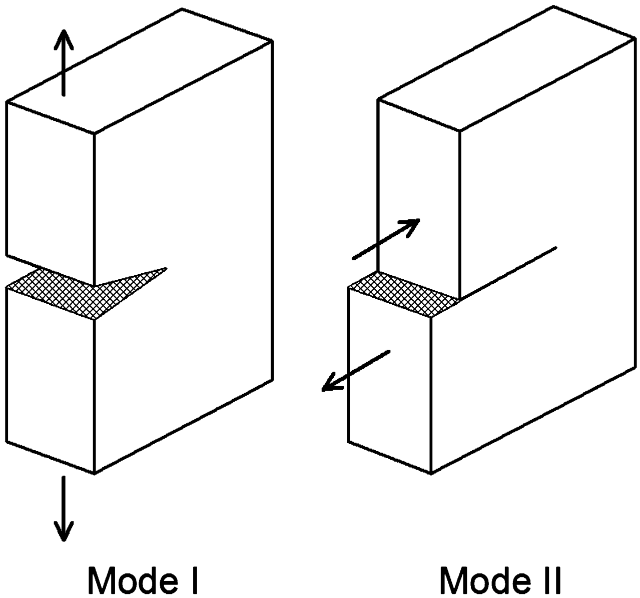
$$PLI_{50} = \left(\frac{D_{\mathrm{e}}\,[mm]}{50} \right)^{0.45} \frac{P_{\max}}{D_{\mathrm{e}}^2} \tag{2}$$

Due to the possibility of testing specimens of different shapes and of testing both in the laboratory and the field, researchers have proposed using this test to estimate *UCS*, although the failure mode is different in the two tests. Many empirical relations between *UCS* and *PLI*, for different rock types, have been obtained by researchers, so that this indirect test is now widely accepted for estimating *UCS* (Broch and Franklin 1972; Bieniawski 1975). It has been found, on average, that the uniaxial compressive strength *UCS* is about 20–25 times the index *PLI*. Tests on many different types of rock, however, show that the ratio between the two strengths can vary between 15 and 50, especially for anisotropic rocks. For low to medium-strength rocks, the ratio can be significantly less than 20 (Singh et al. 2012).

Fracture Toughness

The analysis of the stress distribution in the neighborhood of a crack tip considers three basic plane modes of distortion. The modes called I and II correspond to the two deformation conditions most typically measured. Modes I and II are plane strain distortions where the points on the crack surface are displaced normal and parallel, respectively, to the plane of the crack (Fig. 5). The intensity of loading at the crack tip, for the condition of crack propagation, is quantified by the stress intensity factors. These factors, depending on the type of material, correspond to the material property called fracture toughness.

Three ISRM Suggested Methods for determining mode I static fracture toughness K_{IC} have been presented. ISRM (Franklin et al. 1988) proposes to carry out a test on chevron



Rock Laboratory Tests, Fig. 5 Two deformation conditions of a crack

bend (CB) specimens and on short-rod (SR) specimens. Then ISRM (Fowell 1995) suggests performing the test on cracked chevron-notched Brazilian disk (CCNBD) specimens. The last SM from ISRM (Kuruppu et al. 2013) recommends that mode I static fracture toughness is determined under steady loading using semicircular bend (SCB) specimens.

The SCB specimen has the shape of a semicircular disk with a notch at the center of the planar surface along the thickness direction. Recommended values of the specimen geometry and notch length are given. The load is applied according to a three-point bend scheme through three cylindrical rollers offering no frictional resistance. Two supporting rollers are at the base of the planar surface along the thickness direction, whereas at the bottom, a loading roller is attached to the top loading plate transferring a compressive load. Mode I fracture toughness K_{IC} is determined using the observed peak load, together with some geometrical factors.

ISRM (Backers and Stephansson 2012) concerns the measurement at different confining pressures, through the Punch-Through Shear with Confining Pressure (PTS/CP) experiment, of the mode II plain strain fracture toughness K_{IIC}. The specimen is a right circular cylinder with two circular notches at both the end surfaces of the cylindrical specimen. The jacketed specimen is placed in a loading cell so that a confining pressure can be applied. An axial load is independently applied. The mode II fracture toughness may be evaluated, as a function of the confining pressure, from the peak load achieved during testing.

The measurement of the fracture toughness finds its field of application where the process of crack propagation is relevant: from the breaking ability of cutting tools to stability problems under the approach of the fracture mechanics.

Static and Dynamic Elastic Properties

Elasticity is the capacity to recover deformations when loads are applied to a body. Linearly, elastic bodies present linear relationships between applied stresses and resulting strains (Hooke's law), whose coefficients are the elastic constants. In intact rocks, these are related to the stiffness of the components (crystals or clasts) and to the presence of defects, such as pores and cracks, at the macroscopic and microscopic scale.

Behavior of intact rock is typically nonlinear when subjected to large stresses, and therefore, elastic properties are defined only for appropriately small stress ranges.

Most rocks behave as isotropic materials, whose response is independent of the orientation of the applied stress. Other rocks behave anisotropically. The most common type of anisotropy, affecting schistose and some types of sedimentary rocks, is transverse isotropy, corresponding to a full rotational symmetry around one axis. For isotropic and transversely isotropic materials, the number of independent elastic constants are 2 and 5, respectively. For other types of symmetry, the elastic constants are up to 21.

Elastic properties may be determined from static measurements of stress and strain or by dynamic methods. In the latter case, elastic wave speeds (seismic or ultrasonic velocities) are commonly measured.

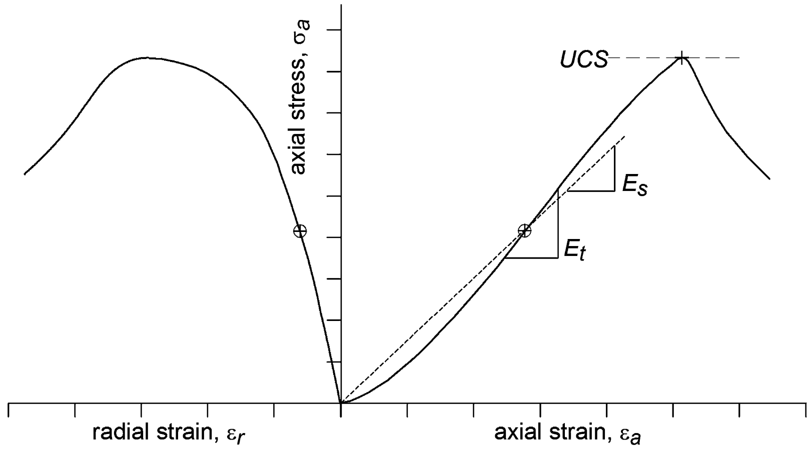
Stress-Strain Curve for Intact Rock

The stress-strain curve of an intact rock specimen is obtained in uniaxial and triaxial compression tests. For general purposes, the complete curve, including pre and post-peak behavior, is determined under an uniaxial compressive load (Fairhurst and Hudson 1999; ASTM D3148).

The complete curve for rocks was previously obtained using a stiff testing machine, whereas today servo-controlled testing machines are used. The control system operates in axial strain or radial strain, measuring the feedback signal at high loop-closure rates. When rock is expected to show a ductile behavior, the control variable is the axial strain. However, the choice between axial or radial strain as the control variable has to be considered when brittle behavior is expected. The most widely used control variable is the axial strain, whereas if a reduction of the axial strain is expected after the peak, only the radial strain provides the complete stress-strain curve.

According to ISRM (Fairhurst and Hudson 1999), a more stringent rule is required in preparing specimens, whose diameter is at least 20 times the largest grain or crystal in the rock.

To measure strains over a large volume, direct contact extensometers (two axial diametrically opposed and one circumferential) are recommended. If strain gauges are used, their length should be more than ten grain/crystal diameters.



Rock Laboratory Tests, Fig. 6 Complete stress-strain curve for a rock specimen showing the compressive strength *UCS*, the tangent E_t and secant E_s Young's moduli

Specimens are subjected to a uniaxial compressive load and the test is also suited to determine *UCS*.

Uniaxial stress conditions allow calculation of the engineering elastic constants. For an isotropic rock, the elastic properties are: Young's modulus E of the rock, defined as the ratio of the change in axial stress to the change in axial strain, and Poisson's ratio ν, calculated as the ratio of the change in radial strain to the change in axial strain.

As the stress-strain behavior is by no means elastic, conventional Young's moduli (Fig. 6) are defined: the tangent Young's modulus E_t, generally measured at a stress level equal to 50% of the *UCS*; the secant Young's modulus E_s, generally measured from zero stress to a stress level equal to 50% of the *UCS*.

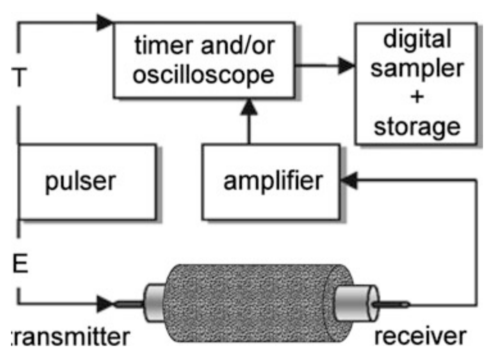
Tangent and secant of Poisson's ratios ν are calculated similarly to Young's moduli.

Sound Velocity

The ISRM SM for determining sound velocity (ISRM 1978b) proposes different approaches, which utilize waves generated at different frequency ranges.

An updated SM from ISRM (Aydın 2014) covers the high (100 kHz–2 MHz) and low (2–30 kHz) frequency ultrasonic pulse techniques. Two pulse techniques are proposed according to whether a single transducer (pulse-echo technique) or a pair of transducers (pitch-catch technique) is used. In the pitch-catch technique, the two transducers can be arranged in different positions; the most frequently used is the direct transmission configuration (transducers located at the ends of the specimen) (Fig. 7), where direction and length of the wavefront are known with greater certainty. This is also the set-up recommended by ASTM (D2845).

The test provides the velocities of compressional (longitudinal, P) and shear (transversal, S) waves in rock specimens of virtually infinite extent, compared to the wavelength of the pulse used.



Rock Laboratory Tests, Fig. 7 Layout of the direct transmission configuration and components of the ultrasonic apparatus (E transmitter excitation signal, T timer trigger signal) (Modified from Aydın 2014)

The simplest ultrasonic apparatus includes a signal generator, an arrival timer in the form of a threshold trigger and/or an oscilloscope for visual analysis of the waveform, amplifiers and filters for signal enhancement, and a data acquisition unit (Fig. 7). The oscilloscope displays both the direct pulse and the first arrival of the transmitted pulse, thus measuring travel time. Note that as the first transmitted arrival is the P-wave, its detection is relatively easy, but the S-wave arrival may be masked by the reflections of the P-wave.

Blocks or cylinders are the typical shapes of the specimens. In the pulse transmission technique, the receiver is positioned on a plane opposite to the plane to which the transmitter is pressed through a low stress (about 10 kPa) to assure a good contact. A thin layer of coupling medium, such as high-vacuum grease or glycerin, between specimen and transducer ensure also an efficient transmission of the signal. The velocities of either P or S-waves (V_P, V_S) are calculated by dividing the transmitter-receiver distance to the measured travel time.

In isotropic rocks, the relations between the velocities V_P, V_S, and the dynamic elastic moduli (E_{dyn}, G_{dyn}, ν_{dyn}), where ρ is the specimen density, are the following:

$$V_P = \sqrt{\frac{E_{dyn}}{\rho} \frac{1 - v_{dyn}}{(1 + v_{dyn})(1 - 2v_{dyn})}}$$
$$V_S = \sqrt{\frac{G_{dyn}}{\rho}} = \sqrt{\frac{1}{\rho} \frac{E_{dyn}}{2(1 + v_{dyn})}} \quad (3)$$

From these, two independent dynamic elastic moduli can then be calculated.

In transversely isotropic rocks, the measurements of velocities in the directions normal and parallel to the symmetry plane help determine four of the five elastic constants.

The dynamic elastic moduli are expected to differ from the static constants.

Dynamic elastic properties are affected by the microstructural characteristic, such as size and shape distribution of voids and grains and their relative arrangements. The influence of microfissures on the elastic properties is greater for the dynamic than for the static properties, and therefore, dynamic measurements are especially suited to study the effect of microfissures. Anisotropy is also investigated through these measurements.

Strength and Stiffness of Discontinuities

Especially, when a rock mass is modelled as a discontinuous medium, strength and stiffness of discontinuities are required. Currently, scale effects are only accounted for by *in situ* direct shear tests, but their high costs have made laboratory direct shear tests of relatively small discontinuity specimens a standard practice.

Both constant-normal-load (CNL) or constant-normal-stiffness (CNS) tests can be used. The former are adequate for near-surface problems, whereas the latter are preferred when joint behavior at relevant normal stress is to be analyzed and normal stresses do not remain constant during shearing. Testing procedures of both CNL and CNS tests are described in detail by the ISRM SM (Muralha et al. 2014). More details on CNS tests can be found in Indraratna and Haque (2000). The procedure for CNL tests is also the object of the D5607 ASTM standard. In the following sections, attention is focused on CNL tests, more extensively used also at construction sites. The ISRM SM mentions the possibility of determining stiffness parameters but does not provide detailed indications.

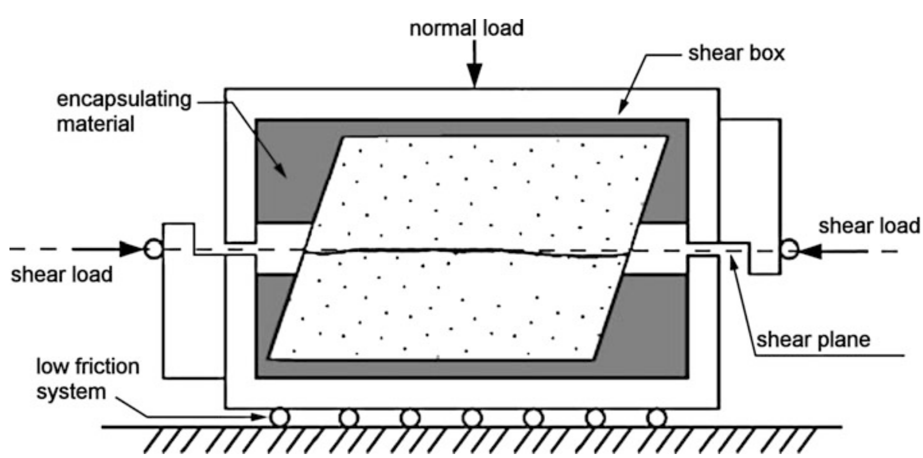
Direct shear devices on rock discontinuities are similar to those utilized for soil laboratory tests (Fig. 8). Specimen preparation requires great care in the position of the discontinuity, which must coincide with the shear plane.

The test is subdivided into two phases: a normal load is applied and maintained constant, a shear load is then applied. Normal σ_n and shear τ stresses, as well as normal u_n and shear u_s displacements, are measured. Stresses are the average values calculated as the force over the area of overlap of the two specimen halves.

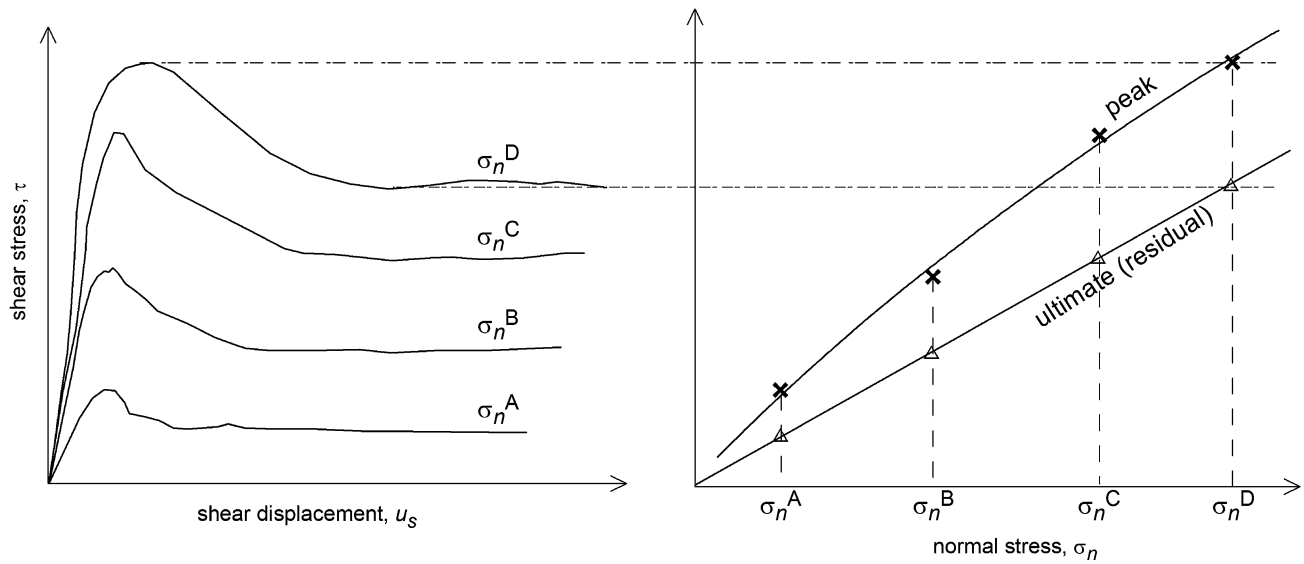
After peak strength is attained and the maximum shear displacement allowed by the apparatus is reached, further shearing cycles are performed in order to obtain ultimate shear strength (residual strength is usually reached at very large shear displacements). Direction of shearing should not be changed because strong asymmetry of asperity and formation of rock chips on the joint surface might alter original joint conditions. Typical results of shear tests on discontinuities are plotted in Fig. 9.

Usually, a single specimen gives peak strength at a unique value of the normal stress. Moreover, the ultimate strength at different values of normal stress, progressively higher, can be obtained.

Both the ISRM SM and the ASTM standard prescribe a device that applies shear displacements at a constant rate. However, a "portable" shear box is widely used, especially at construction sites, which applies shearing through load increments. When using this apparatus, an adjustable pressure maintainer for the normal load is preferable. The apparatus, specimen preparation, and testing procedure are described by Ross-Brown and Walton (1975).



Rock Laboratory Tests, Fig. 8 Typical scheme of a shear box accommodating a core rock specimen with a joint (Modified from Muralha et al. 2014)



Rock Laboratory Tests, Fig. 9 Shear stress vs. shear displacement and normal stress vs. shear stress curves for CNL tests (Muralha et al. 2014, modified)

Major uncertainties in portable-box tests derive from measurements of u_n during shearing and from reconstructing the $\tau - u_s$ behavior immediately after peak. Due to rotations of the upper half of the box, determination of dilation is affected by uncertainties that can be limited by averaging measures of four transducers mounted on top of the box corners. Evidence of post-peak behavior of joints with strain-softening behavior is actually difficult.

Regardless of the device, prior to testing, roughness of the joint surface should be measured (possibly also after the test) through a 2D or 3D profilometer (Muralha et al. 2014).

The 2014 ISRM SM is mainly intended to describe tests on clean discontinuities (without infilling) with negligible tensile strength. The SM recommends that, in the case of infilling, complete dissipation of excess pore pressures due to consolidation must occur before shearing. Conversely, the ASTM standard also includes clay-filled joints, noting that the test yields the undrained shear strength. This strength should be taken with caution because it also reflects partial saturation of the infilling. A realistic estimate of the shear strength of a clay-filled joint in drained conditions requires a shear box allowing immersion of the test joint in water and adoption of shear displacement rates that are sufficiently low to allow dissipation of excess pore pressures during shearing.

Since joint tests produce scattered results, a number of specimens with comparable surface conditions retrieved from the same joint or from joints of the same set is required (5 according to ISRM). Where a limited number of joint specimens is available, peak strength at different normal stresses can be obtained from the same joint specimen with a multistage procedure (Muralha et al. 2014).

Normal stiffness can be determined through *ad hoc* normal loading tests or during the normal loading stage preceding shearing. The normal stress is measured simultaneously to the normal displacement. The resulting $\sigma_n - u_n$ curve (Fig. 10a) is nonlinear and can be used to calculate secant or tangent normal stiffness at different values of σ_n.

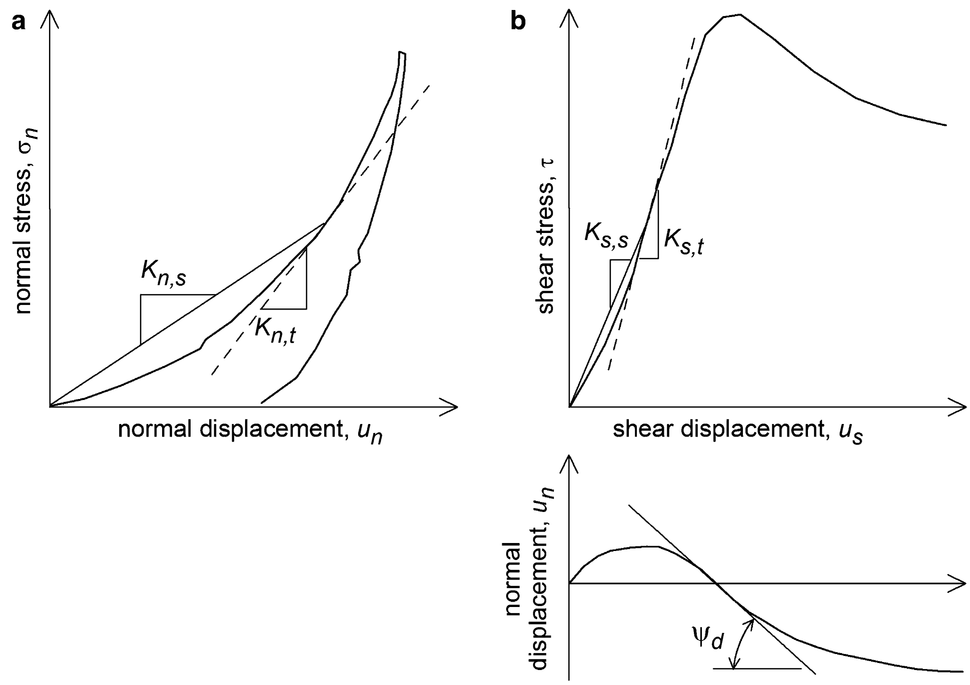
Similarly, tangent or secant shear stiffness and dilation can be calculated from the $\tau - u_s$ and $u_n - u_s$ curves, respectively (Fig. 10b). A detailed discussion on different aspects of joint deformability can be found in Bandis et al. (1983).

Permeability of Intact Rock

Permeability is a rock property necessary for analyzing hydro-mechanical problems. It describes the capacity of porous materials to be passed through by a fluid ("primary" permeability). In rock masses, water flow is mainly controlled by the aperture of open discontinuities. But flow through intact rock could be relevant when the *in situ* state of stress closes discontinuities, or when the primary permeability of the intact rock is high.

Primary permeability of rocks can be measured with various methods. A standard test method (ASTM D4525) is designed to measure the permeability to air of a small sample of rock, but the same procedures can be applied with a gas.

Permeability is measured by flowing air through the specimen. A permeameter arranges the cylindrical specimen, which is laterally isolated from flowing through a sleeve, allowing a directional flow from one end to the other. The end confining plugs of the permeameter could preferentially have a port for the flow of the air and another for a static



Rock Laboratory Tests, Fig. 10 Definition of normal secant $K_{n,s}$ and tangent $K_{n,t}$ stiffnesses (**a**), shear secant $K_{s,s}$ and tangent $K_{s,t}$ stiffnesses and angle of dilatancy ψ_d (**b**) of rock discontinuities

pressure line, thus permitting the specimen to be tested under loading.

The apparatus includes at least a pressure transducer for measuring the air pressure differential across the ends of the specimen and a microflowmeter for measuring the flow rate of the air.

By way of example, the Hoek cell can be used as a permeameter. It can be equipped with rigid end caps, screwed to the body of the cell; alternatively, two permeable steel platens can be used thus allowing application of a triaxial state of stress.

Three or more tests are performed on a specimen at different air pressure values, from the higher to the lower. At a fixed entrance pressure, the flow rate is measured, so that the coefficient of permeability is calculated as a function also of the air viscosity and of the geometry of the specimen.

Swelling Properties

Swelling affects both argillaceous rocks and rocks containing clay and sulfate minerals (anhydrite and gypsum). In clay-anhydrite rocks, two different swelling mechanisms are involved: the swelling of clay due to hydration of clay particles (the phenomenon lasts some days) and the swelling due to transformation of anhydrite into gypsum (long times are required). Therefore, it is important to determine the mineralogical composition of the rock in order to choose the correct testing procedure.

ISRM SM (Madsen 1999) suggests three different procedures according to their scope. All the tests should be carried out on specimens having the same density and water content as those at the time of sampling. Immediately after recovery, the samples should be carefully wrapped with a waterproof liner such as a thin plastic sheet, followed by an aluminum foil and sealed with paraffin or similar. The specimens, prepared in the laboratory avoiding the use of water, are circular discs whose thickness is 2–3 times shorter than the diameter.

The first test measures the time-dependent axial swelling stress of a radially confined rock specimen. The specimen is arranged in a rigid steel ring having the specimen diameter and is sandwiched between two porous steel plates. Then the specimen is located in a container. A loading plate, placed on top of the porous plate, transmits the load at the loading piston. The assembly is inserted in a rigid frame where the swell heave is resolved in an axial load and vice versa.

After a minimum axial stress is applied, the container is filled with water, thus covering the specimen. The axial stress and displacement are measured and recorded as a function of elapsed time. If rock contains only clay minerals, small amounts of strain can be compensated in a stepwise manner by increasing the axial force. If rock contains anhydrite minerals, strains due to the transformation of anhydrite into gypsum cannot be compensated. The test proceeds until the maximum axial force developed by the specimen can be determined or estimated.

The second test measures the axial and radial free swelling strain developed in unconfined stress conditions. The specimen is located in a container. It is confined by a thin flexible steel band, used to determine the radial swelling deformation, and is sandwiched between two porous steel plates.

After the assemblage, the container is filled with water to a level above the top of the specimen. Then the axial swelling displacement is recorded as a function of time elapsed, until a maximum (or constant) value is reached. The increase in circumference is measured at the end.

The third test measures the axial swelling strain necessary to reduce the axial swelling stress of a radially constrained rock specimen. The test is practicable only on purely argillaceous rocks. This test is quite similar to the first, but the specimen is initially loaded stepwise up to a desired axial stress (*in situ* stress conditions). The container is then filled with water to cover the top porous plate. A succession of heaves and axial load decrements are measured, until no displacement can be observed for the particular load decrement.

Summary

Laboratory testing to determine rock properties is important to various civil, mining, and petroleum engineering activities. Collected samples must be prepared and identified carefully prior to undertaking tests. Typical laboratory tests involve determination of physical properties (density, porosity, water content), hardness and abrasiveness, durability, strength (uniaxial compressive strength, tensile strength, triaxial compressive strength, point load strength, fracture toughness), static and dynamic elastic properties (stress-strain curve for intact rock, sound velocity), strength and stiffness of discontinuities, permeability of intact rock, and swelling properties.

Cross-References

▶ Abrasion
▶ Armor Stone
▶ Building Stone
▶ Clay
▶ Compression
▶ Consolidation
▶ Deformation
▶ Density
▶ Deviatoric Stress
▶ Dilatancy
▶ Durability
▶ Effective Stress
▶ Elasticity
▶ Failure Criteria
▶ Field Testing
▶ Geotechnical Engineering
▶ Hooke's Law
▶ International Society for Rock Mechanics (ISRM)
▶ Mechanical Properties
▶ Normal Stress
▶ Petrographic Analysis
▶ Poisson's Ratio
▶ Pore Pressure
▶ Rock Mechanics
▶ Rock Properties
▶ Saturation
▶ Sedimentary Rocks
▶ Shale
▶ Shear Strength
▶ Shear Stress
▶ Soil Laboratory Tests
▶ Strain
▶ Strength
▶ Stress
▶ Voids
▶ Young's Modulus

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Rock Mass Classification

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Synonyms

Ground classification; Rock strength classification

Definition

A classification system that captures all relevant information on the composition and characteristics of a rock mass to provide initial estimates of support requirements and to provide estimates of the strength and deformation properties of the rock mass.

Characteristics

One of the earliest classification systems for rock was developed by Terzaghi (1946). The classification system was developed as a method of classifying rock masses and evaluating rock loads based on qualitative assessments.

The rock-quality designation (RQD) developed by Dr. Don U. Deere (Deere and Deere 1988) is a method of logging sound drilled rock core to calculate and quantify the percentage of "good" rock in a core run. RQD is a quantitative method of evaluating rock quality and is widely used as one of the parameters in other more numerical rock classification systems.

$$\begin{aligned} \text{RQD} = {} & \text{Sum of length of core pieces 4 inches or greater} \\ & /\,\text{Total length of core run} \times 100\% \end{aligned}$$

The Rock Mass Rating (RMR) system developed by Bieniawski (1989) and the Quality Index (Q) updated by Barton (2002) provide overall comprehensive indices of rock mass quality for the design and construction of excavations in rock.

The RMR system incorporates rock mass data regarding rock strength, RQD, discontinuity spacing, discontinuity condition, groundwater, and an adjustment for discontinuity orientation with respect to the excavation. These parameters are assigned numeric values based on their conditions and the summation of the numeric values for all the parameters is the rating of the rock mass.

The Quality Index (Q) uses parameters similar to the RMR system to evaluate the stability that can be expected for excavation within the rock mass. One of the differences between RMR and Q lies in the assessment of the *in situ* stress state in the Q system by use of the "Stress Reduction Factor." The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1000 and is estimated from the following expression:

$$Q = (RQD/Jn) \times (Jr/Ja) \times (Jw/SRF),$$

where

Jn = joint set number
Jr = joint roughness number
Ja = joint alteration number
Jw = joint water reduction factor
SRF = stress reduction factor

The general relationship between Q and rock quality is provided in Table 1 below.

A new classification system, termed the Geotechnical Strength Index or GSI, Marinos et al. (2006), captures variability in geologic materials associated with faulting and extreme deformation associated with tunnels in rock. It is meant to provide reliable input data related to rock-mass properties required as input for numerical analysis or closed form solutions for designing tunnels.

Rock Mass Classification, Table 1 Relationship between Q and rock quality values

Q	Rock quality
400–1000	Exceptionally good
100–400	Extremely good
40–100	Very good
10–40	Good
4.0–10	Fair
1.0–4.0	Poor
0.1–1.0	Very poor
0.01–0.1	Extremely poor
0.001–0.01	Exceptionally poor

Cross-References

▶ Engineering Geological Maps
▶ Site Investigation

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Rock Mechanics

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Definition

Rock mechanics is a subdiscipline within applied geology, geological engineering, and mining engineering focused on the physical mechanics of rock with applications in both dynamic structural geology and in engineering.

An understanding of rock behavior requires proficiency in engineering mechanics, material properties and physics, and engineering geology, including structural geology. Rock mechanics may be considered with soil mechanics as end members of geomechanics.

Rock is a natural material with substantial ranges in material properties: strong or weak, stiff or highly deformable, ductile or brittle, and durable to easily disaggregated and weathered. In the lower end of the strength range, rock will approach soil-like behavior. Physical strength is highly variable across rock types, ranging from soil-like values (less than 1 MPa for uniaxial strength) for weak mudstones to uniaxial compression strength up to 500 MPa for fine-grained basalt, for example (Brady and Brown 2005; Hoek 2006). Likewise, rock stiffness varies across at least two orders of magnitude (up to 100 GPa for poly-minerallic rocks and higher for single-mineral crystals or glass).

Rock Mechanics, Fig. 1 Silty limestone (micrite) rock mass exposed along a highway road cut, including intact rock blocks separated by discontinuities (subhorizontal bedding planes, two steeply dipping joint sets, and a few additional randomly oriented joints) (Photo by D. Jean Hutchinson)

Rock generally has limited void space between the mineral particles, and therefore can be of high unit weight and very low porosity and permeability, or it may have voids created at the time of formation of the rock (particularly for clastic sedimentary or clay-rich rocks, or vesicular basalt or scoria) or created subsequently by preferential weathering of various mineral components. Permeability in most rock masses is created by the presence of discontinuities separating blocks or fragments of intact rock (Fig. 1). The highest permeability rocks are generally those where water flow has dissolved mineral constituents creating macroscale, interconnected pathways, such as in karstic limestone.

The range of engineering properties depends on the origin of the rock material, whether sedimentary, igneous, or metamorphic, and upon the environment in which the rocks have existed. Engineering analysis of rock depends upon whether it will be used as a construction material or be used *in situ* for excavations at the ground surface and underground, or for foundation support. In some cases, whether due to application of high stress to rock *in situ* or the presence of weak rock, excavations may be subject to creep closure or violent rockbursting behavior.

In situ rock behavior is controlled by the solid material and the presence and orientations of discontinuities, including joints, shear zones, and faults – together these components make up the rockmass. The expected strength of the rockmass depends upon the spacing (frequency) and strength and stiffness characteristics of the discontinuities, the strength of the rock between the discontinuities, and the magnitude and orientation of applied stress. The response of the rockmass to excavation or foundation loading depends on the scale of the engineering work relative to the frequency of the discontinuities – a larger excavation in a given rockmass will be subject to more substantial deformation and potential for failure than a smaller excavation.

Rock mechanics, as part of dynamic structural geology, includes the physics of brittle fracture, strain weakening and ductile yield, as well as the rheology of "flow" and continuum deformation (in a geological context). It includes consideration of the relationship between stress and strain across all yield modes.

Rock mechanics, as part of geotechnical engineering and rock engineering, includes the considerations described above, and also involves acquiring and interpreting field, instrumentation, and laboratory data on soil, rock, and groundwater conditions; evaluating distribution of stresses, including the response of the rockmass within foundations to the pressure imposed; analyzing seepage and drainage, and slope stability and stabilization measures; and understanding the effect of creating underground excavations on the rockmass, including the generation of rockmass damage depending on the excavation method and rate, and the use of rock support systems. For underground excavations, engineering analysis is required to control the behavior of the rockmass. This may require limiting deformation (for example, in water supply tunnels, or large open pit slopes), preventing rockmass loss or failure into the excavation (for example, in caverns for hydropower generation or mining stopes), or minimizing the damage to control rockmass permeability (for nuclear waste disposal). Important considerations in design with rock are the expected design life for the excavation, whether public access will be allowed, whether enhanced drainage may improve stability (for large natural rock slides; Clague and Stead 2012), and whether rockmass support will be subject to deterioration by corrosion, creep, water inflow, and/or high stress levels. Long-term design considerations include review of support and drainage-system performance and rehabilitation requirements, the potential for eventual excavation collapse to induce subsidence that may lead to distress in the ground surface or damage to adjacent infrastructure.

In addition to theoretical aspects of strength and deformation resulting from load combinations, rock mechanics also involves knowledge of construction methods, geology, and hydrogeology. Rock strength can be tested in the laboratory considering both the strength of the intact rock over a range of confining stresses and the strength of the discontinuities. *In situ* testing, considering both the influence of the intact rock and discontinuities on the rockmass response, can be completed, but generally such testing tends to be restricted to a relatively small volume of the rockmass due to the scale of most engineering works in rock. The influence on rock mass behavior of the discontinuities can be estimated with rockmass classification systems, based on empirical data collected from excavations or slopes, providing estimates of stand-up times for unsupported tunnels and requirements for effective tunnel support. Analysis of expected rock response is increasingly supported by numerical simulations, which are initially developed considering site investigation data and the type and shape of opening to be excavated, and calibrated using monitoring data as the excavation proceeds. In this case, the observations of rock response in early stages of engineering projects in rock should be used to guide subsequent excavations, whether at the same time or on future projects.

The geological origins and history of rock formations are essential considerations, because materials with substantially different engineering characteristics are often found within larger-scale excavations, particularly tunnels or large slopes. Data may be collected from previous experience with construction in these materials, and from mapping and analysis of surface exposures and drill hole core of the same or similar rock types with similar types of joints and faults.

Rock mechanics provides the analytical tools to evaluate stresses, strains, and deformations in rock materials, depending upon the excavation method and support systems installed. It is a subdiscipline of geological engineering, mining engineering, and geotechnical engineering within civil engineering that typically requires collaboration with professionals with expertise in geology, engineering geology, and hydrogeology.

Cross-References

- ▶ Borehole Investigations
- ▶ Classification of Rocks
- ▶ Dewatering
- ▶ Effective Stress
- ▶ Excavation
- ▶ Foundations
- ▶ Geotechnical Engineering
- ▶ Groundwater
- ▶ Instrumentation
- ▶ Mass Movement
- ▶ Rock Laboratory Tests
- ▶ Rock Properties
- ▶ Shear Strength
- ▶ Shear Stress
- ▶ Site Investigation
- ▶ Subsurface Exploration

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Rock Properties

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Definition

Engineering properties of rocks are the indices used for comparing the engineering behavior of rocks tested under similar conditions, following standardized procedures.

Introduction

Rocks are significant in engineering construction because (West 1995):

1. They are important building materials with numerous applications in engineering construction.
2. Many engineering structures are built directly on rock, and their stability depends on the stability and quality of the foundation rock.

The engineering properties of rocks determine their behavior as construction materials and as structural foundations. There are two classes of rock properties: (a) intact rock properties and (b) rock mass properties. An intact rock contains no visible discontinuities (joints, bedding, foliation planes, etc.) whereas a rock mass is interrupted by discontinuities. Properties of intact rock are measured on small samples in the laboratory, whereas rock mass properties, being controlled by planes of weakness in the rock, are evaluated by studying large outcrops in the field.

Intact Rock Properties

Properties used for characterizing intact rock as a building material include: specific gravity, absorption, porosity, degree of saturation, unit weight (density), unconfined compressive strength, tensile strength, shear strength, Young's modulus, Poisson's ratio, and durability (Johnson and De Graff 1988; West 1995).

Specific Gravity, Absorption, Porosity, Degree of Saturation, and Unit Weight

Specific gravity is the ratio of the weight in air of a given volume of rock to the weight of an equal volume of water. In order to account for the presence of pores in a rock, the American Society for Testing and Materials (ASTM) (ASTM 2013) recommends using three different types of specific gravity in engineering practice. The laboratory test for determining specific gravity and absorption [ASTM D6473-10 (ASTM 2013); International Society for Rock Mechanics (ISRM) 2007)] requires that the rock specimen be weighed in air in a dry condition, weighed in air in a saturated condition, and weighed in water in a saturated condition. From these data, specific gravity and absorption values are obtained as follows:

$$\text{Bulk specific gravity } (\text{Sp } G_d) = A/B - C \tag{1}$$

$$\text{Bulk specific gravity } (\text{Sp } G_s) = B/B - C \tag{2}$$

(saturated, surface dried)

$$\text{Apparent specific gravity } (\text{Sp } G_a) = A/A - C \tag{3}$$

$$\text{Absorption} = \{(B - A)/A\}(100) \tag{4}$$

where:

A = mass of rock in air, oven dried for 24 h
B = mass of rock in air, saturated, surface dried
C = mass of rock in water, saturated

Porosity is the ratio of the volume of voids (Vv) to the total volume (Vt) of a rock, expressed as a percentage. It can be determined by using phase relations, as described in most soil mechanics textbooks. Porosity can range from 0.1% for dense rocks like diabase and quartzite to 5–25% for sandstone, and even higher for volcanic rocks like tuff (Gonzalez de Vallejo and Ferrer 2011).

Degree of Saturation is the ratio of the volume of water (Vw) to the volume of voids in a rock, expressed as a percentage. It can also be determined by using phase relations and ranges from 0% for completely dry rock to 100% for completely saturated rock.

The unit weight, or density, of rock is defined as the mass per unit volume and can be obtained by multiplying the bulk specific gravity by the density of water (1 g/cm^3; 1 Mg/m^3) or by dividing the mass by volume of a core sample. The general range of unit weight is 20–30 kN/m^3 (Gonzalez de Vallejo and Ferrer 2011).

Density, absorption, and degree of saturation show strong correlations with compressive strength (Shakoor and Bonelli 1991; Shakoor and Barefield 2009). Rocks with higher specific gravity and density and lower percent absorption, porosity, and degree of saturation have more desirable engineering properties.

Rock Strength

Depending upon the nature of applied stresses, rock strength can be described as unconfined compressive strength, tensile strength, and shear strength.

Unconfined Compressive Strength

The unconfined or uniaxial compressive strength is one of the most commonly used properties of rock (Bieniawski 1989). Either ASTM method D7012-13 (ASTM 2013) or ISRM method (ISRM 2007) is used to determine unconfined compressive strength. These test methods involve failing an NX-size (54 mm) core sample, with a length to diameter ratio of 2.0–2.5, under the application of vertical load. The strength is obtained by:

$$\sigma_c = P/A \tag{5}$$

where:

σ_c = unconfined compressive strength
P = failure load
A = cross-sectional area

Unconfined compressive strength of intact rock ranges from less than 1 MPa for weak rocks (shales, claystones, mudstones, etc.) to more than 350 MPa for rocks like granite, basalt, and quartzite (Johnson and De Graff 1988; West 1995; Gonzales de Vallejo and Ferrer 2011).

Table 1 shows the typical ranges of compressive strength for selected rock types. The large variation in strength within the same rock type is due to variation in petrographic characteristics. Compressive strength is greatly influenced by the texture, mineral composition, type and amount of cement, and degree of weathering (Johnson and De Graff 1988; Shakoor and Bonelli 1991). Among the igneous rocks, basalt and diabase exhibit higher average values of compressive strength than do granites because of their finer grain size and greater degree of grain interlocking. Also, the higher strength of quartzite can be attributed to a higher degree of grain interlocking. The high strength sandstone is characterized by a smaller percentage of straight grain-to-grain contacts (Shakoor and Bonelli 1991) and a higher percentage of siliceous cement.

Rock Properties, Table 1 Typical ranges of compressive and tensile strength values for selected rock types

Rock type	Compressive strength (MPa)	Tensile strength (MPa)
Granite	75–300	10–25
Diabase	100–350	15–55
Basalt	100–300	10–30
Quartzite	175–350	10–30
Sandstone	20–235	5–25
Shale/claystone/ mudstone	5–125	1–20
Limestone	50–250	5–30
Marble	100–200	10–20

Source: Farmer 1983; Johnson and De Graff 1988; West 1995; Gonzales de Vallejo and Ferrer 2011

The ASTM method D7012-13 for measuring compressive strength is time consuming and core samples required for the test are not always available. For this reason, several empirical tests for estimating compressive strength have been developed, of which point load and Schmidt hammer tests are the most frequently used. The point load test consists of placing an unprepared core sample or an irregular lump of rock between two conical platens and applying compressive load until the sample fails in tension (Broch and Franklin 1972; ASTM method D5731-08 (ASTM 2013); ISRM 2007). The three variations of the point load test include: (i) testing an irregular sample, (ii) testing a core sample axially, and (iii) testing a core sample diametrically. From the failure load P, and platen separation D, as indicated by the apparatus, the point load index I_s is determined as follows:

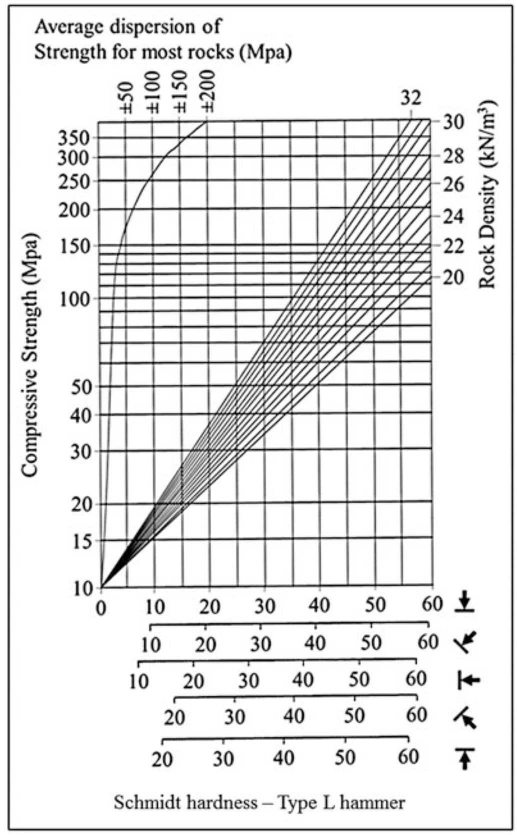
$$I_s = P/D^2 \tag{6}$$

Unconfined compressive strength of a rock is related linearly to point load index by the following equation:

$$\sigma_c = k(I_s) \tag{7}$$

The value of k depends on core diameter. For NX-size (54 mm) samples of most hard rocks, k is approximately 24 (Broch and Franklin 1972; Bieniawski 1989; Cargill and Shakoor 1990). For weaker rocks (shale, claystone, mudstone), the k values are significantly less (11–16). For irregular samples, Broch and Franklin (1972) have developed correction charts that can be used to normalize I_s values to 50 mm standard size.

The Schmidt hammer (Type L) is a portable device that can be used to estimate compressive strength in both the laboratory and the field. The hammer is pressed against the rock and a rebound number (N) is noted from the scale provided on the hammer sleeve. The rebound number has been correlated previously with unconfined compressive strength as shown in Fig. 1. The Schmidt hammer is considered to be a less reliable estimator of compressive strength than the point load test (Johnson and De Graff 1988; Cargill and Shakoor 1990).



Rock Properties, Fig. 1 Relationship between Schmidt hammer rebound number and unconfined compressive strength (After Deere and Miller 1966)

Other indices of compressive strength include shore hardness, indentation hardness, and block punch strength index. Test procedures for determining these indices can be found in ISRM suggested methods (ISRM 2007).

Tensile Strength

The tensile strength of rocks is important in the design of roof spans for underground excavations or in situations where rocks are subjected to bending stresses. On average, tensile strength of rocks is approximately 10% of their compressive strength (West 1995), the range being 5–15%. Table 1 shows the ranges of tensile strength for some common rocks. The tensile strength can be determined either directly by applying a tensile load on a core sample, referred to as the direct pull test (ASTM D 2936-08 (ASTM 2013); ISRM 2007), or indirectly by applying a compressive stress on a disk-shaped sample and failing it in tension, called the Brazilian test (ASTM D3967-08 (ASTM 2013); ISRM 2007). Tensile strength is influenced by the same geologic parameters as compressive strength.

Shear Strength

Shear strength of rocks is evaluated by determining the shear strength parameters (c and φ). This is accomplished by establishing the Mohr envelope by either performing a direct shear test (ASTM D5607-08; ASTM 2013) or a triaxial test (ASTM D 2664-08; ASTM 2013; ISRM 2007). The cohesion value for rocks can range from less than 1 MPa for some weak argillaceous rocks (Hajdarwish et al. 2013) to as high as 48 MPa for stronger rocks like granite (West 1995), whereas friction angle can range from 10° for weak argillaceous rocks (Hajdarwish et al. 2013) to 70° or more for strong quartz-rich rocks (West 1995). Shear strength parameters are controlled by the same textural and mineralogical characteristics that influence compressive and tensile strengths, such as grain size, grain shape, degree of grain interlocking, type and amount of cement, percentage of clay size material, percentage of quartz, etc.

Elastic Properties

Elastic properties indicate deformational behavior of rocks. A cylindrical sample subjected to axial compression will decrease in length and increase in diameter. Upon removal of compressive force, some, but not all, of the deformation may be recovered. The recoverable deformation is the elastic deformation and the nonrecoverable deformation is the plastic deformation. In engineering, deformation is expressed as strain, the ratio of the change in dimension or volume to the original dimension or volume, expressed as a percentage. Figure 2 shows a typical stress-strain curve for rocks and elastic and plastic deformations. The two elastic properties that are used most frequently to evaluate the deformational behavior of rocks are Young's modulus and Poisson's ratio. Methods for determining these two properties have been standardized by ASTM (ASTM D7012-13; ASTM 2013) and ISRM (2007)

Young's Modulus

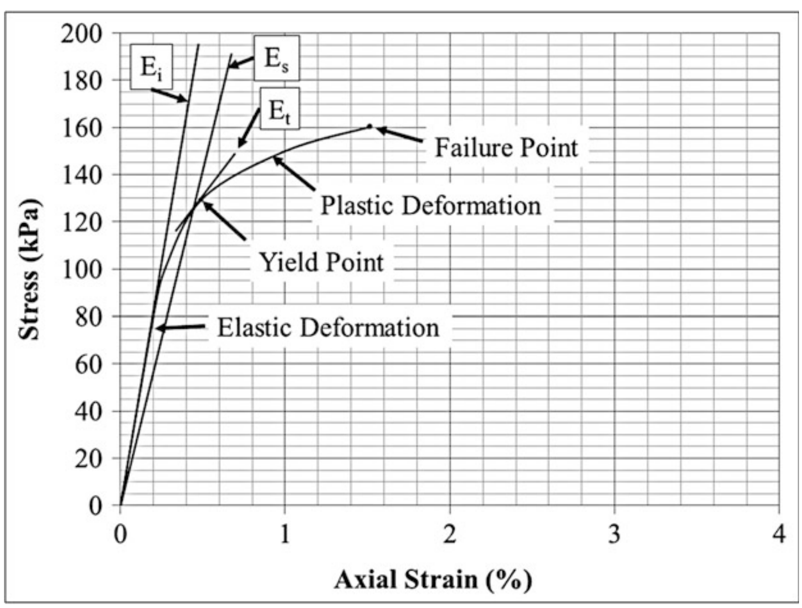
Young's modulus or modulus of elasticity (E) is the ratio of stress to strain:

$$E = \sigma/\varepsilon = (P/A)/(\Delta L/L) \qquad (8)$$

where:

σ = stress
ε = strain
P = applied load in kg or Newtons
A = cross sectional are in cm^2
ΔL = change in length in cm
L = initial length in cm

E is the slope of the stress-strain curve shown in Fig. 2. Since the slope is variable, three different E values are shown in Fig. 2. These include the initial tangent modulus (E_i), the secant modulus at any point selected on the curve (E_s), and tangent modulus at any point selected on the curve (E_t). In engineering practice, E_{t50} and E_{s50}, tangent and secant modulus at 50% of the failure load, respectively, are frequently used. Young's modulus is a very valuable property for estimating the anticipated deformation under given loading conditions. In general, rocks with higher compressive strength also exhibit higher E values (Shakoor and Bonelli 1991) because both properties are controlled by the same petrographic characteristics. Average E values can range from 13.7 GPa for shales and claystones to 79.9 GPa for quartzites (Johnson and De Graff 1988; West 1995).



Rock Properties, Fig. 2
A typical stress-strain curve for rocks showing elastic versus plastic deformation and the three types of moduli of elasticity

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Poisson's Ratio

Poisson's ratio (ν) compares lateral extension to vertical compression.

$$\nu = \text{lateral strain}/\text{vertical strain} = (\Delta D/\mathrm{D})/(\Delta L/\mathrm{L}) \quad (9)$$

where:

ΔD = change in diameter or lateral dimension in cm
D = initial diameter or lateral dimension in cm
ΔL = change in length in cm
L = initial length in cm

A perfectly elastic material has a Poisson's ratio of 0.33. The Poisson's ratio for rocks can range from 0.1 to 0.5 with values for most rocks falling between 0.15 and 0.25 (Johnson and De Graff 1988; West 1995).

Durability

Durability is the resistance of a rock to climatic changes such as heating and cooling, wetting and drying, and freezing and thawing, that is, to weathering and disintegration. Shale, especially clay shale, claystone, and mudstone frequently exhibit nondurable behavior upon wetting and drying. Several durability evaluation tests have been developed since the early 1970s, the most important of these being the slake durability index test developed by Franklin and Chandra (1972). Both ASTM (D4644-08; ASTM 2013) and ISRM (2007) have standardized the procedure for slake durability testing. The test consists of placing an oven-dried sample, consisting of 10–12 pieces, each weighing 40–60 g with a total weight of 450–500 g, in a 2 mm-meshed drum and rotating the drum through water for 10 min at a fixed speed. The sample that remains in the drum is oven-dried and weighed. The slake durability index (Id) is calculated as the ratio of the weight of the remaining sample to the initial weight, multiplied by 100. Repeating the test on the remaining sample provides the second-cycle slake durability index (Id_2). Id_2 can range from 0% for some claystone to nearly 100% for some silty shale or siltstone. Id_2 is frequently used as the standard for classification purposes.

Engineering Classification of Intact Rock

Classifications of intact rock, based on compressive strength and modulus ratio (Young's modulus/compressive strength), developed by Deere and Miller (1966), are given in Tables 2 and 3, respectively. The very high strength category in Table 2 includes rocks like basalt, diabase, and quartzite. Other igneous rocks, limestone, dolomite, and well-cemented sandstone are included in the high strength category, whereas schist and silty shale belong to the medium strength category. Clay shale, claystone, and mudstone fall in the low to very low strength categories. Since modulus ratio takes into account both the compressive strength and elastic modulus, it is considered to be more reflective of the engineering behavior of rocks than compressive strength alone. Marble has a distinctly high modulus ratio and that explains the historical use of marble as an excellent building stone. Granite, diabase, limestone, and dolomite mostly have medium values of modulus ratio, whereas foliated rocks can have modulus ratios ranging from low to high depending upon the direction of compression with respect to foliation.

Rock Properties, Table 2 Engineering classification of intact rock on the basis of unconfined compressive strength (After Deere and Miller 1966)

Class	Description	Uniaxial compressive strength (MPa)
A	Very high strength	Over 220
B	High strength	110–220
C	Medium strength	55–110
D	Low strength	27.5–55
E	Very low strength	Less than 27.5

Rock Properties, Table 3 Engineering classification[a] of intact rock on the basis of modulus ratio (E_t/σ_c) (After Deere and Miller 1966)

Class	Description	Modulus ratio[b]
H	High modulus ratio	Over 500
M	Medium modulus ratio	200–500
L	Low modulus ratio	Less than 200

[a]Rocks are classified by both strength and modulus ratio such as AM, BL, BH, CM, etc.
[b]Modulus ratio = E_t/σ_c
E_t = tangent modulus at 50% ultimate strength
σ_c = unconfined compressive strength

Rock Properties, Table 4 Durability classification based on second-cycle slake durability index (After ISRM 1979)

Second-cycle slake durability (Id_2)	Classification
0–30	Very low
30–60	Low
60–85	Medium
85–95	Medium high
95–98	High
98–100	Very high

Numerous durability classifications for clay-bearing rocks have been proposed by various researchers. Table 4 shows the ISRM (1979) classification based on Id_2.

Rock Mass Properties

The design and stability of large engineering structures such as dams, tunnels, highway cuts, and surface and underground mines depend on the properties of rock masses that are controlled by the presence of discontinuities such as bedding planes, joints, foliation, faults, and shear zones. Also, rock masses are significantly more anisotropic than intact rock.

There are seven aspects of discontinuities that are significant with respect to the stability of rock masses. These include geometry, continuity, spacing, surface irregularities, physical properties of adjacent rock, nature of infilling material, and groundwater (West 1995; Wyllie and Mah 2004).

Geometry deals with the orientation of the discontinuities and plays a fundamental role in the stability of rock slopes (Wyllie and Mah 2004) and roofs of underground openings (Hoek and Brown 1980).

Continuity indicates the persistence of the discontinuities. The more continuous the discontinuities, the weaker the rock mass.

Spacing represents the frequency of discontinuities, with spacing and continuity being interrelated. Table 5 shows a classification of discontinuities based on spacing by Deere (1964). Closely spaced discontinuities represent a weaker rock mass with greater potential for slope failure and deformation.

Surface irregularities contribute to increased resistance against failure by either overriding the irregularities or shearing through them (Patton 1966; West 1995; Wyllie and Mah 2004). When a discontinuity separates two different rock types, such as a bedding plane between sandstone and shale units, the properties of the weaker rock unit will control the shear strength along the discontinuity.

Infilling includes all soil-like material filling the discontinuities. The properties and thickness of the infilling material influence the resistance against shearing significantly (West 1995; Wyllie and Mah 2004).

Groundwater decreases the shear strength of a rock mass through buildup of pore pressure (Wyllie and Mah 2004; Gonzalez de Vallejo and Ferrer 2011).

Engineering Classification of Rock Mass

The following sections discuss briefly the various indices and classification schemes that describe the quality of rock mass and quantify its engineering behavior.

Percent Core Recovery

Percent core recovery is the ratio of the length of the core obtained to the length drilled, expressed as a percentage. It indicates both the quality of drilling and the soundness of the rock. A core recovery of 90% indicates a sound, homogeneous rock, a 50% recovery suggests rock with seams of weak, weathered material, and very low or no recovery means the rock is highly decomposed.

Rock Quality Designation (RQD)

Rock Quality Designation, developed by Deere (1964), is one of the most important and universally used indices of rock mass quality. It is defined as the ratio of the sum of NX-size core pieces that are equal to or greater than 10 cm to the total length drilled, expressed as a percentage. Table 6 shows the rock mass quality bands based on RQD. The RQD has been used to estimate Young's modulus (Coon and Merritt 1970), loads on tunnel support systems (Cording et al. 1975), and bearing capacity of foundation rock (Peck et al. 1974). However, while using RQD, one should keep in mind that: (1) RQD depends on the driller's experience; (2) schistose rocks may have a high RQD value but still contain many planes of failure; and (3) joints filled with clay seams may be widely spaced but can still result in failure.

Fracture Index

Fracture index or fracture frequency is the number of fractures per meter length of core (Farmer 1983). The higher the fracture index, the poorer is the quality of the rock mass.

Velocity Index

Comparing the square of the seismic wave velocity through a rock mass in the field (VF)2 to the square of seismic wave velocity through an intact rock sample in the laboratory (VL)2 is known as the velocity index or velocity ratio (Onedera 1963; Farmer 1983; Gonzalez de Vallejo and Ferrer 2011). As the fracture frequency in rock mass increases, the velocity index decreases. Conversely, a decrease in fracture frequency will result in an increase in velocity index. Table 7 (Farmer 1983) shows the relationship between rock mass quality, RQD, fracture frequency, and velocity index. For a given direction, the correlation between velocity index and RQD is 1:1 (Gonzalez de Vallejo and Ferrer 2011).

Rock Properties, Table 5 Descriptive classification of discontinuity spacing (After Deere 1964)

Bedding	Spacing	Joints
Very thin	< 5 cm	Very close
Thin	5–30 cm	Close
Medium	30 cm–1 m	Moderately close
Thick	1–3 m	Wide
Very thick	> 3 m	Very wide

Rock Properties, Table 6 RQD quality bands (After Deere and Miller 1966)

RQD (%)	Description
0–25	Very poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Very good

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Rock Properties, Table 7 Relationship between RQD, fracture frequency, and velocity index (After Farmer 1983)

Quality classification[a]	RQD[a] (%)	Fracture frequency (per meter)	Velocity index $(v_F^2)/v_L^2)^b$
Very poor	0–25	>15	0–0.2
Poor	25–50	15–8	0.2–0.4
Fair	50–75	8–5	0.4–0.6
Good	75–90	5–1	0.6–0.8
Excellent	90–100	1	0.8–1.0

[a]Deere and Miller (1966)
[b]V_F is the wave velocity in the field; V_L is the velocity in the laboratory

Rock Mass Classification Systems

One of the earliest rock mass classifications for estimating tunnel supports was developed by Terzaghi (1946) who divided rock mass into categories such as intact rock, stratified rock, moderately jointed rock, blocky and seamy rock, squeezing rock, and swelling rock, based on discontinuity spacing and degree of weathering. However, the more frequently used quantitative classification systems that take into account a number of parameters include the Rock Structure Rating (RSR) developed by Wickham et al. (1972), the Geomechanics Classification or Rock Mass Rating (RMR) developed by Bieniawski (1973), and Rock Mass Quality or Q-system developed by Barton et al. (1974). The parameters considered in developing these classification systems include discontinuity spacing, discontinuity orientation, discontinuity surface properties, intact rock strength, and groundwater conditions. These parameters are assigned varying scores, based on the conditions they represent, which are then added or multiplied to obtain the final rating index.

The RSR system is based on three parameters designated A, D, and C that represent the general geology (rock type and structure), joint pattern (joint spacing and orientation), groundwater, and joint condition, respectively. The system is used specifically for designing support systems for mines and tunnels. The details of this system and its applications can be found in Wickham et al. (1972), Farmer (1983), and Bieniawski (1989).

The RMR classification divides rock mass into five classes (very good, good, fair, poor, and very poor) on the basis of RQD, intact rock strength, joint spacing, joint separation, joint continuity, joint orientation, and groundwater inflow. The RMR has been related to modulus of deformation (Bieniawski 1989) as well as cohesion and friction parameters (Hoek and Brown 1980). Complete details of RMR system are provided in Hoek and Brown (1980), Farmer (1983), and Bieniawski (1989). High RMR scores indicate very good to good quality rock mass, and low RMR scores represent poor to very poor quality rock mass.

The Q-system of the Norwegian Geotechnical Institute (NGI), developed specifically to evaluate tunnel roof stability and design of support system, uses six parameters to obtain the Q value as follows:

$$Q = (RQD/J_n)\,(J_r/J_a)\,(J_w/SRF) \qquad (10)$$

where:

RQD = rock quality designation
Jn = number of joint sets
Jr = joint roughness
Ja = joint alteration
Jw = water inflow in joints
SRF = stress reduction factor

In the equation for Q value, RQD/J_n represents the block size, J_r/J_a the inter-block shear strength, and J_w/SRF the active state of stress (loosening load during excavation, squeezing load in incompetent rock, residual stress relief in competent rock). The higher the Q value, the better the quality of rock mass with respect to tunneling. Tables for assigning scores to various parameters comprising the Q-system can be found in Hoek and Brown (1980), Farmer (1983), and Bieniawski (1989).

Summary

There are two classes of rock properties: (i) intact rock properties and (ii) rock mass properties. Intact rock properties include specific gravity, absorption, porosity, degree of saturation, unit weight, unconfined compressive strength, tensile strength, shear strength, Young's modulus, Poisson's ratio, and durability. These properties are determined in the laboratory, and they are controlled by the petrographic characteristics of the rock. Properties of intact rock are used to evaluate the suitability of a rock for use as construction material. Rock mass properties, controlled by discontinuities, include percent core recovery, rock quality designation (RQD), fracture index, and velocity index. Rock mass properties are measured in the field on rock outcrops, and they are used to evaluate the quality of a rock mass for structures such as dams, tunnels, mine openings, and building foundations. Classification systems, based on intact rock properties and rock mass properties, have been developed to classify intact rock and rock mass into categories ranging from very good quality rock or rock mass to very poor quality rock or rock mass. These quantitative classifications provide the basis for evaluating the quality of rock as building material and for designing engineering structures on or inside the rock mass.

Cross-References

► Angle of Internal Friction
► Building Stone

▶ Dams
▶ Deformation
▶ Density
▶ Durability
▶ Engineering Properties
▶ Mechanical Properties
▶ Modulus of Deformation
▶ Modulus of Elasticity
▶ Poisson's Ratio
▶ Shear Strength
▶ Velocity Ratio
▶ Young's Modulus

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Run-Off

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Definition

Run-off, also referred to as runoff or surface runoff, is the term used to refer to that part of precipitation in the water cycle that moves on the ground surface downslope or downstream away from the location where it first accumulated as rain or snow. Surface water run-off is part of the water cycle that also includes evaporation, infiltration, and storage (USGS 2016). Run-off is estimated from rainfall based on an empirical approach that is known as the rational method (Goyen et al. 2014)

$$q = C\,A\,p \tag{1}$$

where q is the peak unit discharge in m^3/s from a drainage basin at a point of interest, C is a dimensionless coefficient that represents the amount of run-off as a decimal fraction of precipitation, A is the drainage basin area in m^2 above the point of interest, and p is representative rainfall intensity in mm/hr for a meaningful duration and a desired return period. The duration for the rainfall intensity may be the time of

R

concentration (the time required for water to flow from the most distant point in the drainage basin to the outlet defined as the point of interest). The unit discharge for several precipitation return periods (2 y, 5 y, 10 y, 25 y, 50 y, 100 y, 200 y) would be of interest for flood routing and flood hazard studies.

$$Q = C\,A\,P \tag{2}$$

where Q is total discharge volume in m^3 for a duration of interest, often one year, and P is total precipitation that falls during the period of interest. Thus, P might be the annual precipitation averaged to represent the drainage basin area. The total discharge would be of interest for water resources information.

The value of the C coefficient depends on the ground conditions in the drainage basin (vegetation, bare soil, exposed bedrock, urbanized pavement, and rooftops) that tends to vary with time and may be seasonal. The value of C also depends on the duration of the precipitation; C in a particular drainage basin would be lower for a rainfall event associated with a 2-year return period than for a 50-year rainfall event.

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S

Sabkha

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Synonyms

Salina; Salt flat; Salt pan

Definition

This is an Arabic term for salt flats found in the deserts and coastal areas of the Arabian Peninsula and North Africa. The type locality is the Sabkha Matti, at the boundary of the United Arab Emirates and Saudi Arabia. Similar environmental conditions are recognized in other arid regions where evaporite minerals are deposited.

Characteristics

The sabkha landscape includes salt flats, salty soils (halosols), scoured sand plains, and sand dunes. Wind erosion commonly forms a sabkha plain, known as a Stokes surface (Fryberger et al. 2006), where the strength of cohesion from water or crystalline salt is sufficient to bind sand grains in place. Windblown sand deposits may contain significant portions of salt-cemented silt-sand pellets and crystalline salt. Lag deposits of deflated soil may contain larger salt crystals, including clustered gypsum crystals known as desert roses. Below stable sabkha surfaces, salts may accumulate within the upper 0.5 m layer to form a salt-cemented soil, such as gypcrete, that can provide limited strength to sabkha soils.

Aragonite, gypsum, and halite are common in coastal sabkhas of the Red Sea (Banat et al. 2005), typical of seawater-fed sabkhas. Coastal sabkhas in the Arabian Gulf are associated with dolomitization of aragonite and the deposition of gypsum in shallow sediments (Patterson and Kinsman 1982).

Coastal sabkhas form in the supratidal zone where intermittent flooding by seawater saturates soils and deposits evaporate salts (Glennie 1998). Seasonal tides can alter the water table and cause local flooding of back beach areas. Soluble salts that add strength to sabkha soils may dissolve, changing soil properties abruptly. Weak, saturated soils can extend several meters below the surface.

Sabkha soils can be difficult to manage for engineering purposes. Seasonal saturation can make soils susceptible to liquefaction or rapid loss of bearing capacity, as shown on Fig. 1. Reduced durability of concrete from attack by salt is

Sabkha, Fig. 1 Front end loader stuck while attempting to recover a truck and trailer mired on an unimproved road in sabkha terrain in the UAE. The road had been passable for the previous 3 months (Photograph, M. McMackin 2010)

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common in the Arabian Peninsula, with chloride being more problematic in hot-humid coastal environments (Haque et al. 2006). The deposition or dissolution salts in the soil can cause heaving or subsidence. Phase changes between gypsum and anhydrite in sabkha soils can cause volumetric change of as much 40% resulting in heaving or subsidence (Azam 2007).

Cross-References

- ▶ Coastal Environments
- ▶ Desert Environments
- ▶ Evaporites
- ▶ Marine Environments
- ▶ Saline Soils

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Saline Soils

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Definition

Soils with high concentrations of halite (NaCl) or other soluble evaporites such as additional chlorides, sulfates, or nitrates.

Saline soils may occur naturally in coastal mudflats (sabkhas), desiccated lakes and pans, and ephemeral river environments and are particularly common in desert environments such as the coastal Gulf States, the Middle East, Northern Africa, Asian interior, arid and some semiarid parts of South and North America, Australia, Southern Africa, and Spain. They have also been identified in the Arctic permafrost (Brouchkov 2003) and reclaimed land. The sources of salt may be seawater or dissolved salts from sedimentary bedrock which are concentrated in areas of shallow water tables (Salama et al. 1999) and high evaporation rates. Saline soils are a particularly common by-product of dryland irrigation but can also be the result of seawater intrusion and even atmospheric sea spray inputs.

Saline soils may depict a white surface crust and altered physical characteristics as well as water stressed plants exhibiting leaf burn and other drought symptoms; however, salt tolerance for plants is highly variable (Qadir et al. 2000). Soil salinity may also accelerate decay and corrosion of infrastructure including, roads, buildings, and pipelines. Salt

Saline Soils, Fig. 1 (**a**) Saline sabkha surface in Walvis Bay, Namibia, producing evaporitic salts. Note plastic membrane on top of lime rich soil used in the building foundation. (**b**) Saline damp rising in newly constructed building in Dubai, in close proximity of the saline creek. This is likely to result in weakening of plaster and cement decay and rising damp can be associated with a wide variety of building materials and climates (Charola 2000). Salinity can be measured using electrical conductivity (EC) which is expressed in decisiemens per meter (dS/m) or by measuring the total soluble salts (TSS) expressed in parts per million or milligrams per liter (ppm). Given the spectral characteristics of salts, salinity may also be detectable in satellite imagery (Metternicht and Zinck 2003).

Saline soils can be treated by applying lime and gypsum (calcium sulfate, CaSO4·2H2O), by washing, draining, and leaching with salt-free water or by lowering the ground water table. To safeguard buildings, adding layers of broken stone, lime, and gypsum or an impermeable membrane may eliminate salt damage to infrastructure (Fig. 1).

Cross-References

- ▶ Desert Environments
- ▶ Desiccation
- ▶ Evaporites
- ▶ Permafrost
- ▶ Sabkha

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Sand

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Definition

Sand is a loose granular substance resulting from the erosion of rocks and minerals. The word has two meanings: sand is defined as a size class if used in terms of a single grain, or as a textural class when used in terms of an aggregate of grains. As a general rule, sand feels gritty when rubbed between the fingers. Silt, by comparison, feels like flour.

The two most widely used size classification schemes to define sand are the Unified Soil Classification System (USCS) and the Krumbein phi scale. The Unified Soil Classification System defines sand as particles with a diameter between 0.074 and 4.75 mm. The Krumbein phi scale defines sand as particles 1/16th mm to 2 mm, or 0.0625 mm to 2 mm. There are other less common or older classification systems as well.

Sand is found in rivers, beaches, dunes, and shallow shelf seas. The most important commercial sources of sand have been glacial deposits, river channels, and river flood plains. Whereas some countries routinely mine offshore deposits of sand for onshore construction projects, this is not permitted in some western countries due to environmental concerns.

The demand for sand comes from a wide range of economic sectors. Its largest use is in construction, and second largest in land reclamation. In lesser volumes, sand is used in the production of glass and electronics. Its use has been growing rapidly for use in hydraulic fracturing. The quantities and properties of sand are important in engineering geology investigations for geological hazard assessments, geotechnical properties and slope stability, erosion, land reclamation and shoreline nourishment, groundwater seepage, and geological mapping.

In construction, sand is a principal component of concrete. It is mixed with clay to form bricks, and is used in mortar and in plaster. Sand is the one of the most widely consumed natural materials in the world. In fact, over twice as much sand (40 billion tons) is mined, than is created in a year. The trend for sand extraction can be estimated from the records for cement production, which has increased 437% in China and 59% in the rest of the World according to 2014 statistics.

The most common sand is silica sand (SiO_2). It is formed by preferential weathering of weaker materials in rock, most commonly granite, and secondarily gneiss. Feldspar and other minerals in the rock are more prone to chemical decay, leaving behind small crystals of quartz. These are often transported by flowing water or wind. There will be varying amounts of feldspar and other minerals in a sand deposit depending on how quickly preferential weathering has taken place and how far it is from source to deposit.

In addition to weathering of igneous rock, sand can be formed by precipitation of chemicals, especially calcium carbonate, the disintegration of shells, pelletization of finer grained material, and even by the deposition of feces by certain living organisms (e.g., parrot fish).

The degree of roundness in sand indicates its history. In engineering, sharp sand is preferred for concrete and other

Sand, Table 1 Sand classification according the Unified Soil Classification System (ASTM Standard D2487 (2000)

			Soil classification	
Sand 50% or more of coarse fraction passes No.4 sieve (4.75 mm)	Clean sand Less than 5% fines	$C_U >= 6$ and $1 <= C_C <= 3$	SW	Well-graded sand
		$C_U < 6$ and/or $Cc < 1$ or $C_C > 3$	SP	Poorly graded sand
	Sand with fines More than 12% fines	Fines classify as ML or MH	SM	Silty sand
		Fines classify as CL or CH	SC	Clayey sand

Sand, Table 2 Sand sizes according the Krumbein phi scale

Type	Φ	Size in mm
Very fine sand	−1–0	1/16–1/8
Fine sand	0–1	1/8–1/4
Medium sand	1–2	1/4–1/2
Coarse sand	2–3	1/2–1
Very coarse sand	3–4	1–2

building purposes. Sand (soil) classification according to the USCS is first based on grain size, followed by its Coefficient of Uniformity (C_U) and Coefficient of Curvature (C_C), given by

$$C_u = \frac{D_{60}}{D_{10}}$$

where D_{60} is the grain diameter at 60% passing and D_{10} is the grain diameter at 10% passing.

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

where D_{60} is the grain diameter at 60% passing, D_{30} is the grain diameter at 30% passing, and D_{10} is the grain diameter at 10% passing through (Table 1).

On the Krumbein phi scale, the phi value for grain size is given by

$$\Phi = -\log 2D$$

where D is the particle size in millimeters.

For sand, the value of Φ varies from −1 to +4, with the divisions between categories at sand at whole numbers (Table 2).

Cross-References

▶ Aeolian Processes
▶ Aggregate
▶ Aggregate Tests
▶ Alkali-Silica Reaction
▶ Aquifer
▶ Beach Replenishment
▶ Characterization of Soils
▶ Classification of Soils
▶ Clay
▶ Coastal Environments
▶ Current Action
▶ Desert Environments
▶ Engineering Properties
▶ Erosion
▶ Floods
▶ Fluvial Environments
▶ Glacier Environments
▶ Groundwater
▶ Hydrocompaction
▶ Hydrogeology
▶ Igneous Rocks
▶ Infiltration
▶ Landforms
▶ Noncohesive Soils
▶ Quicksand
▶ Sedimentary Rocks
▶ Sediments
▶ Silt
▶ Soil Field Tests
▶ Voids

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Saturation

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Synonyms

Maximum soil water content

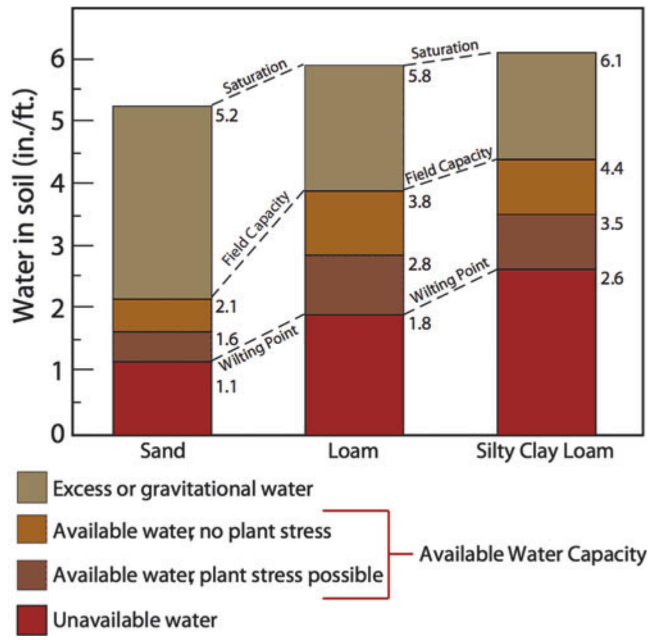
Definition

The condition in which all available pore space in soil or rock is occupied by water (or, in some circumstances, by another liquid).

Soil moisture beneath the ground surface occurs in two zones: the unsaturated and saturated zones. The unsaturated zone has pores between soil grains and cavities that are either partly or not filled by water. The underlying saturated (or phreatic) zone has spaces that are completely filled with water. The interface between the two zones is the groundwater table.

Precipitation or melt water enters the ground and percolates downwards under the influence of gravity until it reaches either an impermeable layer or a pre-existing saturated layer leading to a rise in the water table. In the saturated zone, the water then moves laterally with the groundwater. Movement in sandy or gravelly soils may be of the order of millimeters per day but in clay soils movement may be slower (Alley et al. 1999).

Soil water patterns are influenced by topography, soil properties, vegetation, meteorological conditions, and water routing processes; these differ between humid, semiarid and arid environments. In humid areas, variations in precipitation/melt water and evapotranspiration cause are seasonal changes in saturation (Gómez Plaza et al. 2001).



Saturation, Fig. 1 Available water capacity in three types of soil. At saturation, all pores are filled with water immediately after rainfall. At field capacity, the moisture content is that remaining after gravity has removed all water that it can. At the wilting point, the remaining soil moisture is insufficient to promote continued plant growth.

The degree of saturation is defined as the fraction of porosity that is occupied by water, thus a saturated soil is one that contains the maximum soil water content. This is expressed in volume/volume percent or by saturation units. The total volume of pore spaces (n) varies depending on soil coarseness and texture between approximately 0.25 and 0.75. In the unsaturated zone, the pores are occupied by air and some water (although, in some circumstances, other gases may be present). The volume occupied by water is measured by the volumetric soil moisture content, defined as the total volume of water (θ), hence $0 \leq \theta \leq n$. The soil moisture content equals the porosity when the soil is saturated. Soil moisture content is also sometimes characterized by degree of saturation which is defined as $S_d = \theta/n$. The degree of saturation varies between 0 and 1 (Tarboten 2003). Water can be held more tightly in small pores than in large ones. Therefore a soil with a high proportion of silt grade particles can hold more water than coarse soils because of the higher combined surface areas of the smaller particles (Ball 2001).

The concept of saturation also extends to the proportions of gas, oil, or other fluids distributed within rocks (Snyder 2008) which also depend on porosity. Saturation may also occur with fluid pollutants entering the ground (UK Groundwater Forum 1998).

Saturation is of significance to engineering geology and associated activities in a number of ways:

- Saturation of soils, to the extent that saturation reaches the ground surface leads, to all additional precipitation being diverted as surface run-off and, if there is sufficient upward pressure, leads to groundwater flooding.
- Saturated or nearly saturated soils lead to conditions where heavy machinery cannot be used on site without causing compaction or machinery becoming bogged down (BIO Intelligence Service 2014).
- The level of saturation below ground, which depends on the type of soil, determines the available water capacity that supports plant growth and survival – depending on the depth of penetration of roots, plants may be fully supplied, suffer stress, wilt, or die depending on the type of soil and depth of the saturated zone (Fig. 1). This has implications for reduction of vegetation cover, which has implications for soil erosion, slope stability, and remediation of damaged land through re-vegetation.

Cross-References

► Dewatering
► Erosion
► Excavation
► Floods
► Groundwater
► Infiltration
► Soil Properties

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Sea Level

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Definition

Predictions of sea level change are usually given in terms of the "eustatic level" which can be defined as the mean geodetic level of the sea surface, but the term has to be used with a degree of caution. Firstly, there are many factors influencing local sea level resulting in a wide geographical variation from a mean value as shown in Fig. 1. Whereas most of the world will show positive increases, areas undergoing uplift resulting from reduction in past ice loading will show decreases of sea level. Secondly, modern measurements of sea level use satellite altimetry which makes it easy to determine an average global change, but this technique has only been available since 1993 (Fig. 2). Previously an average had to be calculated from tide gauge records, but these have a distribution related to well-populated areas, and hence the global average was biased, but nevertheless, making appropriate allowances, such records are very useful for studies of historical changes. Sea level is subject to the influence of many factors, and a good review with chapters discussing various problems is given by Church et al. (2010). A brief but comprehensive review is provided below.

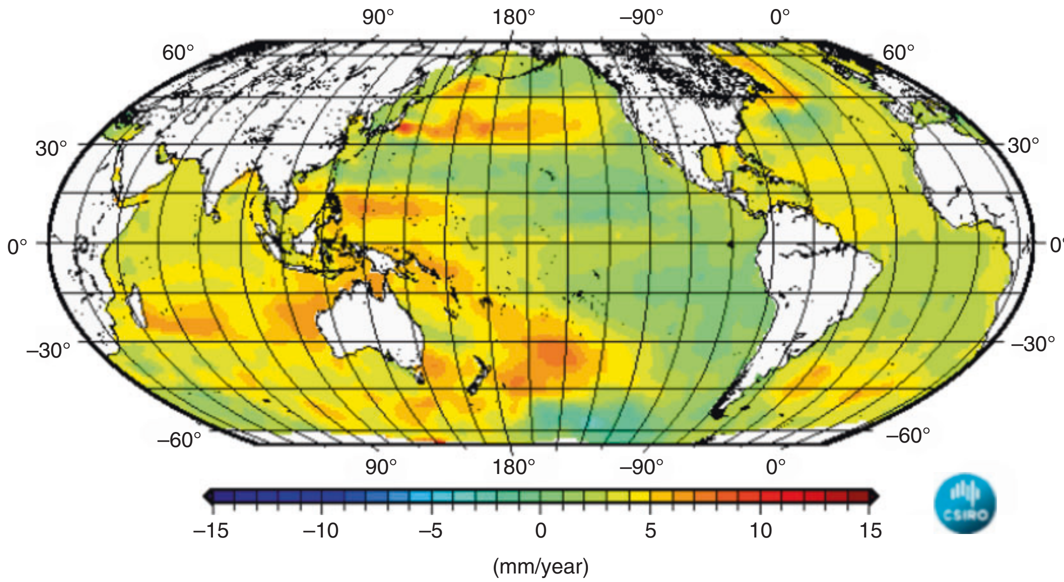
Factors Influencing Sea Level

(i) *Tidal effects* due to the gravitational pull of the moon and the sun with a short-term range between spring and neap tides plus the longer 18.6-year periodicity due to the precession of the moon's elliptical orbit.

(ii) *Meteorological effects* will include the inverted barometric pressure effect but becomes of particular significance with storm surges.

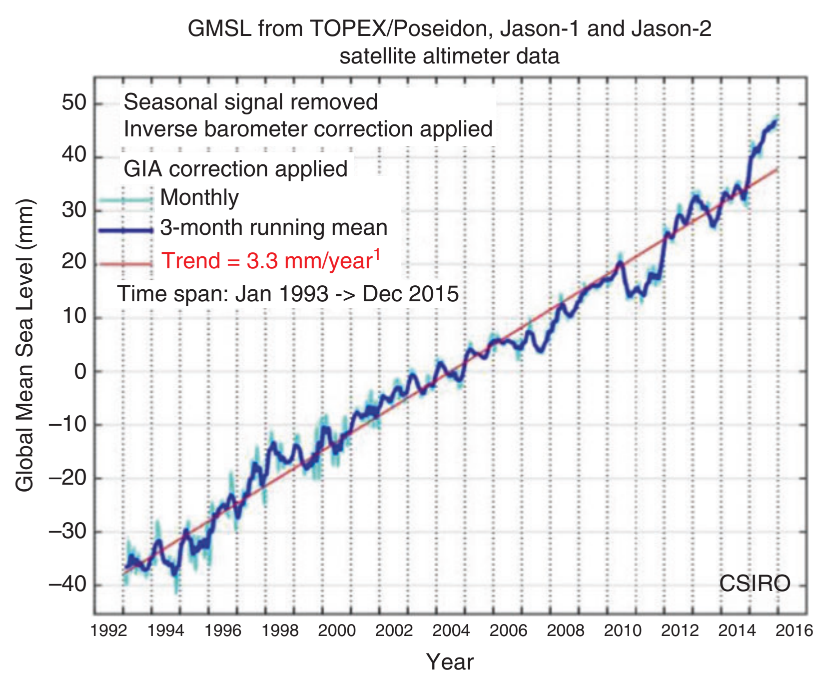
(iii) *Glacio-isostasy* involves the viscous deformation of the Earth's mantle under the load of an ice sheet, the effect not being confined to the area of the sheet itself but also transmitted over a wider area in the form of a "forebulge": the response of the mantle beneath the ice sheet being of an opposite sign to the area of the forebulge.

(iv) *Hydro-isostasy* involves the response of the mantle to the alteration in water loading as a consequence of its extraction and subsequent discharge from the ice sheet. [We should note in passing that the Earth's crust can be considered as an elastic solid so that the elastic strain can be treated as immediate response in comparison to the viscous behavior of the mantle.]



Sea Level, Fig. 1 Sea level trends from 1993 to 2015 as measured by satellite altimetry. The global variability strongly reflects the influence of dynamic and thermo-steric effects: the larger tends in the Western Pacific being the result of the dominant westward trade winds associated with the La Niña years. https://www.cmar.csiro.au/sealevel/sl_hist_last_decades.html (CSIRO 2016)

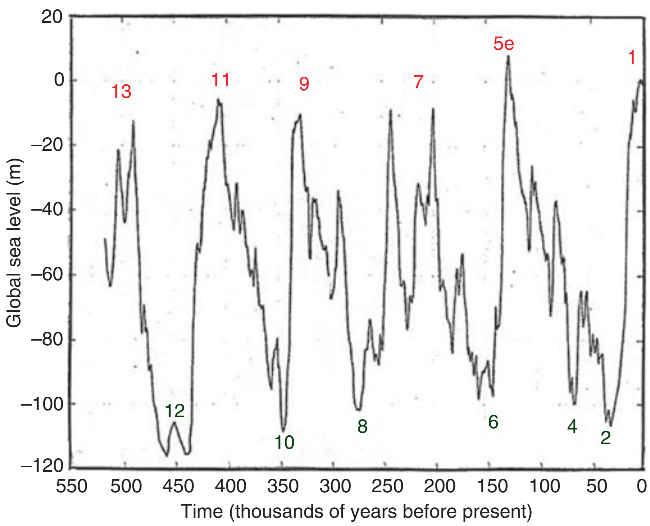
Sea Level, Fig. 2 Global sea levels obtained by satellite altimetry plotted as 3-month running mean from January 1993 to December 2015. The red trend gives an average of 3.3 mm/year. http://www.cmar.csiro.au/sealevel/sl_hist_last_decades.html (CSIRO 2016)



(v) *Continental levering* is a term used for the strain applied to a continent margin by the isostatic stress changes and can be manifest as a tilting of the continental shelf and adjacent coast.

(vi) *Ocean siphoning* refers to the transfer of water from an ocean basin to the area of a subsiding forebulge and also toward the area of reduced elevation caused by continental levering.

(vii) *Gravitational* refers to the attraction of ocean water to the mass of an ice sheet raising the sea level in its vicinity: subsequent decay of the ice sheet allows this water to be released back to the oceans with the result that if we are referring to the Greenland ice sheet, then the subsequent rise in sea level resulting from its decay becomes mainly transmitted to the southern hemisphere and vice versa for the Antarctic.



Sea Level, Fig. 3 Sea level changes during the last five glacial cycles of the Quaternary with identification given by the marine isotope stages (MIS) (Adapted from Rohling et al. 2009 and Church et al. 2010)

(viii) *Rotational* refers to the deviation of the earth's rotational axis relative to the crust and includes the Chandler wobble which has a period close to 436 days and is believed to be generated by atmospheric and/or ocean processes. It is recorded as giving rise to a tidal amplitude of more than 30 mm in the Gulf of Bothnia, but elsewhere the amplitudes are much smaller. More significant deviations of the rotational axis over a longer geological time period occurs due to changes in ice sheet loading and movements generated within the Earth's mantle and core connected with plate tectonics, but the values are very sensitive to viscosity and are the subject of research (Mitrovica et al. 2010).

(ix) *Steric factors.* These include *thermo-steric effects* resulting from ocean warming with the expansion taking place slowly owing to the slow rate at which temperatures are distributed through the ocean water column, *halo-steric effects* resulting from changes in salinity which alter water density, and *dynamic effects* induced by ocean currents, especially where these introduce waters of different temperatures, acting to alter sea levels.

(x) *Construction of dams and reservoirs* transfers water volume from the oceans to the land, and over the twentieth century, the effect has been of significance for reducing eustatic sea level (Milly et al. 2010).

(xi) *Growth and decay of the major ice sheets during the Quaternary* produced sea level changes which are many orders of magnitude larger than the other changes listed above. The magnitude of the changes wrought by the last five glacial stages is shown in Fig. 3 with the maximum reduction in sea level during the last glacial stage (the Devensian) being in the order of -120 m and the highest level reached being estimated as +5 m during the last interglacial, marine isotope stage 5e known as the Eemian (Rohling et al. 2009). It is estimated that if the total volume of water currently locked in the ice sheets was to be released, it would amount to a eustatic sea level rise of 70 m.

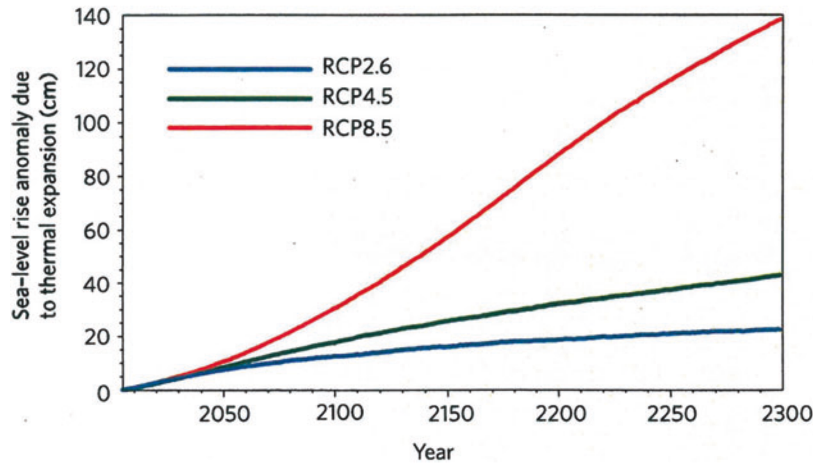
(xii) *Tectonic effects* including earthquakes, tsunamis, and local crustal subsidence, with the former two potentially larger but of short-term significance and the last slower but longer term.

(xiii) *Local subsidence* includes human action such as groundwater lowering and depletion of aquifers for freshwater but which in deltas lowers the ground level by accelerating the consolidation of soft sediments; for example, in Bangkok, ground level has been reduced by 2 m and in parts of Tokyo by up to 5 m.

Note that factors (xii) and (xiii) produce a *relative change* of sea level as opposed to the global changes produced by factors (i) to (xi). We need to note also the *relative magnitudes of the* sea level changes which the various factors can produce. Thus, the changes in level produced by ice sheet melting and accumulation during the Quaternary are many orders of magnitude larger than the other changes. With the decay of the ice sheets following the last glaciation, rapid sea level rise took place during the late Devensian and early Holocene. The late Holocene prior to the onset of global warming, due to the rapid exploitation of fossil fuels, was a period with relative slight changes in ice sheet volumes, although the changes due to items (iii) to (viii) will still have taken place. Although the latter effects are small with magnitudes no more than a few mm/year or less, they are swamped by the short-term tidal and meteorological effects, but nevertheless being effects which are continuous over a geological time scale, their eventual significance outweighs the more ephemeral changes. As from the onset of the industrial revolution, sea level rise gradually increased to the value of 3.3 mm/year as recorded by the CSIRO (Fig. 2).

Predicting Future Sea Level Rise

Predictions of future sea level rise under the influence of global warming, given by the IPCC Fifth Assessment Report (Church et al. 2013), are heavily dependent upon the future economic scenario with the latter controlling the volume of greenhouse gas emissions and their concentration in the atmosphere. Thus, it is convenient to represent the economic scenario in terms of its *representative concentration pathway* (the RCP value, van Vuuen 2011). Sea level rise due to the thermo-steric response of the oceans to the warming which has already taken place is now underway and is calculated to proceed for the next few centuries (Meehl et al. 2012) with the amount dependent upon the RCP value (Fig. 4). Geological evidence for the long-term relationship between the



Sea Level, Fig. 4 Predicted sea level rise over the next three centuries due to the thermal expansion of the oceans in response to the global warming resulting from three possible economic scenarios identified in terms of the relative concentration pathway (RCP value) (Data from Meehl et al. 2012)

concentration of CO_2 and the volume of the ice sheets indicates that the current concentration has reached a level out of equilibrium with the long-term stability of the ice sheets. With the slow operation of natural processes over geological time, it can be expected that eventually sea level will rise to the value shown by the geological evidence which is considered to be more than 9 m above the current level (Foster & Rohling 2013). Calculations from modeling suggest that equilibrium to a CO_2 concentration of 400 to 450 ppm (assuming international agreement controls emissions to this value) will take at least several centuries. Rohling et al. (2013) consider that the rate at which this rise will occur is thus slow enough to allow appropriate action to be taken to minimize the human consequences although other authors suggest that further study of ice sheet dynamics and analysis of the patterns currently being shown could suggest that acceleration of ice sheet decay and concomitant sea level rise over a short time scale remains a distinct possibility (Golledge et al. 2012).

Engineering Aspects

The rate of rise for any local coast is supplied by the observations made locally, and for short-term measures to combat erosion and safeguard against flooding, those observations are paramount. Difficulty arises from the uncertainty associated with the long-term predicted sea levels, and whether or not these justify expenditure on protection rather than managed coastal retreat, problems discussed by Nicholls 2010 and Nicholls et al. 2011. Predictions of sea level rise concentrate on the value likely by the end of the current century, but as noted above, the values for the next few centuries will create serious problems and, if the emissions are not strictly controlled, will be dire. Emission control by itself cannot be sufficient, so effort should be directed to sequestrate CO_2 from the atmosphere, the most effective process for which remains photosynthesis such as the schemes to promote algal growth (Walsh et al. 2015).

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Sedimentary Rocks

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Definition

Rocks formed from the products of physical erosion or chemical and biological processes at the surface of the Earth, both on land and under water.

Sedimentary rocks are formed in four ways:

- Debris from mechanical erosion of soils and rocks then transported and deposited as sediments
- Chemical and biochemical processes
- Particulate material from volcanic eruptions
- Accumulation of organic material mainly from algae and plants

Sedimentary rocks need to be understood in the context of the Earth's dynamic processes. The movement of tectonic plates give rise to volcanic activity and mountain building, which are eroded and the resulting debris are carried to, and deposited in, depressions in the Earth's surface. Other sediments are precipitated directly from water or arise from accumulations of organic material and other deposited from debris originated during volcanic activity. Whereas sedimentary rocks are less abundant in the Earth's crust than igneous and metamorphic rocks, they do cover most of the Earth's surface and, so, are of importance to engineering geology and provide the main mineral resources for building and construction.

Many types of sedimentary rocks contain the remains of organisms either in the form of intact fossils or broken debris. These may be scarce or scattered but in some types of limestone may constitute the major part of the rock.

The complexity of sedimentary rocks has led to several systems of classification, as is possible to understand in the classical book of Pettijohn (1949) and in a consolidated way in Folk (1968), who took account of the descriptive-genetic classification by A.W. Grabau and descriptive classification by P.D. Krynine. A recent comprehensive review of sedimentary rocks is that of Tucker (2008).

Clastic (Epiclastic) Sedimentary Rocks

Epiclastic sedimentary rocks (also known as terrigenous or siliciclastic rocks) include those that originated from geological processes at the surface of the Earth through erosion, transport of debris, and deposition of these as sediments. These consist mainly of quartz accompanied by varying quantities of other minerals, notably feldspar and mica.

A classification based on grain size distinguishes between clay/mudrock (finest), siltstone, sandstone, and conglomerate/breccia (coarsest).

Sandstone (arenites) is the most extensively studied epiclastic rocks, because of the importance of porosity and permeability characteristics. Some sandstone is sufficiently porous to have 20% of their volume filled by oil, gas, or water which characterizes them as good reservoir rocks.

The mineral compositions of sandstone vary depending on the proportions of quartz, feldspar, or lithic fragments. In accordance with the amounts of these three different components, a sandstone can be a quartzarenite, a feldspathic arenite (arkose), or a litharenite.

Angularity of debris also influences classification; thus, a conglomerate is a sedimentary rock composed by thick coarse well-rounded material (pebbles and blocks) with interstitial sand, whereas a breccia has a similar range of particle size but the debris is angular. When there are large quantities of sand and mud forming the matrix between pebbles and blocks, the rock is called diamictite.

The fine grained clastic sedimentary rocks are plastic clay or, if fissile, shale. These mainly consist of clay minerals and some are important for use in brick making, ceramics, creation of impermeable structures, and other industrial uses.

Sediments that are a mixture of clay and silt are referred to as mud. The equivalent rocks are mudrock/mudstone.

Chemical and Biochemical Sedimentary Rocks

Many sedimentary rock are formed at the site of deposition by direct precipitation, chemical or biochemical, from the water and are known as authigenic.

It is difficult to determine if the genesis of the some are purely chemical, without direct or indirect biological influence, such as some limestone and ironstone, but others are purely chemical such as evaporites, (salt deposits), deposited from super-saturated sea or lake water.

Some ironstone exhibits rhythmic alternations between iron minerals, mainly hematite, and pure silica. Several interpretations exist to explain this alternation, some relating to the alternation of climate conditions with proliferation of micro-organisms that promoted hematite precipitation and periods less favourable to life, when the silica precipitated. This rock is known as banded iron formation (BIF). The majority of these rocks, which are often important ores, are extremely old, around 2.5 billion years, a time related to an increase of oxygen in the ocean and atmosphere, due to the evolution of life and appearance and proliferation of cyanobacteria: the first organisms to develop photosynthesis. Around 1.8 billion years ago, the precipitation of ironstone stopped, and in the geological record, the next unique event of iron precipitation occurred at the end of the Precambrian, around 0.7 billion years ago, associated with the "Snowball Earth" when the Earth was completely frozen over (Hoffman and Schrag 2002). Ironstone that occurs in Phanerozoic strata are less abundant and are associated with biological and/or diagenetic processes.

Another kind of chemical sedimentary rock is chert or flint (consisting of micro or crypto-crystalline silica) that sometimes probably originated from precipitation in hot water in springs but, in other cases, during diagenesis. Chert and flint were important to humanity in the past because they were a favored rock for making stone tools.

An unusual sedimentary process, related to biochemical processes, is the formation of calcretes - a type of limestone - in soils in arid and desert areas.

Organic Sedimentary Rocks

Other sedimentary rocks are almost completely formed from accumulations of organic substances. Those consisting of plant debris in the form of unconsolidated peat, and lignite to coal depending on the degree of carbonization due to burial, compression, and heating which drives off water and volatile organic compounds.

In some circumstances, organic compounds, mainly from plankton, are present in significant quantities in pores and, during diagenesis, become oil. Such deposits are known as oil shale. The oil can be extracted by hydraulic fracturing of these rocks.

Both types are important sources of energy but emit large quantities of carbon dioxide during combustion.

Pyroclastic Sedimentary Rocks

Pyroclastic sediments are produced during volcanic eruptions. Molten magma is expelled and rapidly solidifies to form ash and coarser debris. Although of igneous origin, these can be considered as sediments because they are deposited in layers at the Earth's surface and are best classified by grain size: bombs (greater than 64 mm), lapilli (between 64 and 2 mm), coarse ash (between 2 and 0.06 mm) and fine ash (less than 0.06 mm) (Tucker 2008). Rocks consisting of ash are called tuffs. Those consisting of coarse angular debris are volcanic breccias.

Rocks Resulting from a Mixture of Processes

Some sedimentary rocks result from a mixture of processes, for example:

- Epiclastic sediments mixed with authigenic sediments, such as marl, which is a mixture of fine grained limestone and clay minerals; and
- Rocks in which volcanic ash falls into waters where other sediments are being deposited.

Summary and Conclusion

There are four main processes that form sedimentary rocks – (1) epiclastic (terrigenous) by erosion, transportation and deposition of clastic sediments, (2) chemical and biochemical, through direct precipitation of the sediment from water, (3) pyroclastic, from the explosive eruption of volcanoes, and (4) by accumulation of organic material. After deposition, the sediments are transformed into rock by diagenetic processes with the strength of the rock reflecting the degree of compaction and cementation.

Cross-References

▶ Classification of Rocks
▶ Clay
▶ Coal
▶ Diagenesis
▶ Erosion
▶ Evaporites
▶ Hydraulic Fracturing
▶ Limestone
▶ Organic Soils and Peats
▶ Sediments
▶ Shale
▶ Silt
▶ Volcanic Environments

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Sediments

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Synonyms

Accumulation; Deposit

Definition

Sediments are solid, fragmented particulate matter (silt, sand, and gravel), which are transported and deposited by physical processes such as water, wind, and ice. Sediments consist of loose, solid particles originating from the weathering and erosion of rocks, or chemical precipitation from solution, including secretion by organisms in water.

Sediment Characteristics

Sediments vary in terms of particle size, shape (round, oblong, and angular), density, and composition. For example, the most common mineral in river sediment is normally quartz (Williams 2012). Over time sediments become sedimentary rocks through lithification. Sedimentary rocks are basically clastic, chemical, or organic in nature. Common sedimentary structures include bedding (cross-bedding, graded bedding), mud cracks, ripple marks, and fossils.

Sediment Classification

Sediment classification is based on particle size (texture), location (grain deposition), source, and chemistry (Table 1).

Sediments, Table 1 Sediment (Grain) size classification

Descriptive name		Diameter (mm)
Gravel	Boulder	>256
	Cobble	64–256
	Pebble	4–64
	Granule	2–4
Sand	Very coarse	1–2
	Coarse	0.5–1
	Medium	0.25–0.5
	Fine	0.125–0.25
	Very fine	0.0625–0.125
Mud	Silt	0.0039–0.0625
	Clay	<0.0039

Textural classification in the USA conforms to the Wentworth or Krumbein scales. During transportation and deposition, sediments sort into grain size ranges (sorting and grading). Sorting and grading relate to engineering properties, such as compressibility, shear strength, and hydraulic conductivity.

Sedimentation Processes

To dislodge and carry a particle in a stream environment, the current speed must exceed 20 cm/s. When the current speed falls below 1 cm/s, the particles settle out. Sediments in transport are divided into dissolved, suspended, and bed loads. Sediment load is an indicator of stream capacity and competence. On average, 18.3 million tons of sediment is eroded per annum from the surface of continents and washed into the seas and oceans via rivers (Garry 2005). Effective factors on the sediment load rate include type, intensity, and spatio-temporal distribution of precipitation; upper watershed features including soil type, type and situation of vegetation cover; land use and morphological features such as slope, topography, and field area. Factors contributing to sedimentation include climate, geology, soil, hydrology, physiography, and human activities (Miyab et al. 2017).

Engineering Geology Aspects of Sedimentation

In engineering geology, the interpretation of landforms and Earth processes for potential geologic and related man-made hazards that may have an impact on civil structures and human development is very significant. Sediment in transport depends on the relationship between the upwards velocity of the particle (drag and lift forces) and the settling velocity of the particle based on the mass balance as defined through the Exner equation (Paola and Voller 2005). Specifically, if the upward velocity is approximately equal to the settling velocity, sediment will be transported downstream entirely as suspended load. On the other hand, if the upward velocity is much less than the settling velocity, but still high enough for the sediment to move, it will move along the bed as bed load by rolling, sliding, and saltating, whereas if the upward velocity is higher than the settling velocity, the sediment will be transported high in the flow as a suspension load.

Sediments play a crucial role in influencing the suitability of sites for engineering works. Moreover, variability in the nature, extent, and context of sediments provides challenges to engineering geologists in their planning, assessment, and development efforts associated with a wide variety of features in the built environment.

Cross-References

▶ Aggregate
▶ Boulders
▶ Clay
▶ Current Action
▶ Fluvial Environments
▶ Hydraulic Action
▶ Sand
▶ Silt

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Sequence Stratigraphy

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Definition

The study of genetically related bedrock units, surficial deposits and soils bounded by significant correlative erosional, nondepositional, or conformable chronostratigraphic surfaces.

The Earth's crust is composed of layers (strata) of igneous, metamorphic and sedimentary rocks, unconsolidated surficial deposits, and soils. Stratal layers are a geological record of changes in environmental energy (i.e., base level), eustatic sea level, tectonic subsidence and uplift, igneous and volcanic activity, climate change, glaciations, sediment supply, basin physiography, and biotic activity (including humans) through deep time (Table 1). Base level is an abstract plane that represents a surface of dynamic equilibrium between erosion and deposition. It is the upper limit of sedimentation and the lower limit of erosion. Base level change is the relative movement between base level (e.g., relative sea level) and a datum below the sea floor and above crystalline crustal basement. Tectonic processes and glaciations drive uplift and subsidence of the datum, whereas climate change and glaciations drive eustatic sea level changes. Falling base level (and relative sea level) is accompanied by an increase in environmental energy and erosion. A decrease in environmental energy and deposition accompany rising base level (and relative sea level). Deep time refers to the multibillion-year history of the Earth (and solar system) preserved in the geological record.

Successions of genetically related strata with distinctive geological characteristics and engineering properties, bound at their top and bottom by erosional or nondepositional surfaces, can be mapped and correlated locally within drainage basins, regionally across continents, or globally (Vail et al. 1977; Helland-Hansen and Gjelberg 1994; Salvador 1994; Embry 2009; SEPM 2017).

Sequence stratigraphy examines the chronological order in which environmentally related successions of strata were deposited and eroded. This knowledge helps engineers and geologists analyse and account for how bedrock, surficial deposits and soils acquired their petrology, lithological characteristics, bedding relationships, geochemistry, mechanical properties, other geotechnical attributes and natural resource potential. Forward numerical sequence stratigraphy models are designed to account for and predict the architecture of bedrock, surficial Earth materials and soil sequences as they appear in outcrop, geophysical data, borehole logs, or cross-sections. Predictive modelling uses accommodation space, relative sea level change, shelf width, and sediment transport as important spatial and temporal controls on stratigraphic sequences and hierarchies of bounding surfaces (Kamp and Naish 1998; Burgess et al. 2006; Warrlich et al. 2008; Huang et al. 2015). Accommodation space for deposition lies between base level and crustal basement. It is the space generated for accumulation and storage of sediments by changes in relative sea level.

Bounding surfaces are used in sequence stratigraphy for correlation and as unit boundaries. Unconformities are bounding surfaces that represent major erosion events and a significant time gap. Diastems are scoured or abrupt contacts

Sequence Stratigraphy, Table 1 Simplified global chronostratigraphic chart (adapted from Cohen et al. 2013). Strata preserved in the geological record are divided chronologically into eons, eras, periods, epochs, and finer intervals of deep time

EON	ERA	PERIOD	EPOCH
Phanerozoic *present day - 541 Ma*	**Cenozoic** *present day -66 Ma*	Quaternary *present day – 2.6 Ma*	Anthropocene *present day*
			Holocene *present day - 11.7 Ka*
			Pleistocene *11.7 Ka – 2.6 Ma*
		Neogene *2.6-23 Ma*	Pliocene *2.6-5.3 Ma*
			Miocene *5.3-23 Ma*
		Paleogene *23-66 Ma*	Oligocene *23-34 Ma*
			Eocene *34-56 Ma*
			Paleocene *56-66 Ma*
	Mesozoic *66-252 Ma*	Cretaceous *66-145 Ma*	Upper *66-100 Ma*
			Lower *100-145 Ma*
		Jurassic *145-201 Ma*	Upper *145-163 Ma*
			Middle *163-174 Ma*
			Lower *174-201 Ma*
		Triassic *201-252 Ma*	Upper *201-237 Ma*
			Middle *237-247 Ma*
			Lower *247-252 Ma*
	Paleozoic 252-541 Ma	Permian 252-299 Ma	Lopingian *252-259 Ma*
			Guadalupian *259-272 Ma*
			Cisuralian *272-299 Ma*
		Carboniferous 299-359 Ma	Pennsylvanian *299-323 Ma*
			Mississippian *323-359 Ma*
		Devonian *359-419 Ma*	Upper *359-382 Ma*
			Middle *382-393 Ma*
			Lower *393-419 Ma*
		Silurian *419-444 Ma*	Pridoli *419-423 Ma*
			Ludlow *423-427 Ma*
			Wenlock *427-433 Ma*
			Llandovery *433-444 Ma*
		Ordovician *444-485 Ma*	Upper *434-458 Ma*
			Middle *458-470 Ma*
			Lower *470-485 Ma*
		Cambrian 485-541 Ma	Furongian *485-494 Ma*
			Series 3 *494-509 Ma*
			Series 2 *509-521 Ma*
			Terreneuvian *521-541 Ma*
Proterozoic *541 Ma-2.5 Ga*	**Neo-proterozoic** *541 Ma-1 Ga*	Ediacaran *541-635 Ma*	
		Cryogenian *635-720 Ma*	
		Torian *720 Ma – 1 Ga*	
	Meso-proterozoic *1-1.6 Ga*	Stenian *1-1.2 Ga*	
		Ectasian *1.2-1.4 Ga*	
		Calymmian *1.4-1.6 Ga*	
	Paleo-proterozoic *1.6-2.5 Ga*	Statherian *1.6-1.8 Ga*	
		Orosirian *1.8-2 Ga*	
		Rhyacian 2-2.3 Ga	
		Siderian 2.3-2.5 Ga	
Archean 2.5-4 Ga	Neoarchean *2.5-2.8 Ga*		
	Mesoarchean *2.8-3.2 Ga*		
	Paleoarchean *3.2-3.6 Ga*		
	Eoarchean *3.6-4 Ga*		
Hadian *4-4.6 Ga*			

representing a short erosion event or depositional hiatus. Surfaces can also be conformable if there is no time gap between units, whereas diachronous surfaces develop over extended periods of time. When strata below a diastem or unconformity are older than the units above, the boundary is termed a time barrier. First and second-order boundaries penetrate far into the basin and mark noticeable changes in sedimentary regime. Surface uniformities indicate large amounts of erosion in addition to significant changes in eustatic sea level, tectonic setting, and sedimentary regimes. Third-order boundaries and higher have noticeable changes in sedimentary regime, but the amount of erosion and basin penetration of the surface unconformity are less. These sequence boundaries are also easily correlated.

Applications of Sequence Stratigraphy

Although developed by the energy and mining resource industries as a predictive tool in exploration and production, this discipline also has numerous civil, geological, and environmental engineering applications.

Sequence stratigraphy incorporates the disciplines of sedimentology, stratigraphy, geophysics, geochemistry, structural geology, and basin analysis. Basin analysis is the study of the petrological and lithological composition of facies, sedimentary and deformation structures to explain the geological history of a sedimentary environment (e.g., ocean or lake basin).

Understanding the controls on deposition and erosion, and forward predicting stratigraphic patterns has important engineering applications in many fields. Academic researchers focus on genetic interpretations of stratal sequences, environmental reconstructions, and understanding sedimentary basin architecture. Government agencies synthesize this broad information base for thematic mapping and correlation purposes. These data sources are then used by industry project engineers and geologists to guide and minimize risks associated with exploration, production, reclamation of hydrocarbon plays, coal fields, aquifers, and mined ore bodies or in the construction and maintenance of major infrastructure projects.

Hydrocarbon and Groundwater Exploration and Production

The hydrocarbon industry pioneered the application of sequence stratigraphy for subsurface oil and gas exploration and to predict and define significant conventional hydrocarbon plays in Paleozoic, Mesozoic, and Cenozoic strata (e.g., Vail et al. 1977; Dietrich et al. 1989; Embry 2009; Kumar et al. 2009; Huang et al. 2015). Sequence analysis helps engineers and geologists model the geometry of fracture permeable and porous reservoir units through chronostratigraphic correlations and analysis of relationships between geological structures, facies, and bounding surfaces (Fig. 1a). Sequence analysts in the oil and gas industry are particularly interested in subaerial unconformities, regressive erosion surfaces, ravinements, maximum regressive surfaces, maximum flooding surfaces, and slope onlap surfaces.

Subaerial unconformities are significant gaps in the stratigraphic record that are defined by erosive surfaces in underlying strata, often with a weathering zone containing paleosols and karst features, overlain by nonmarine fluvial, lacustrine, glacial, brackish, and marine strata or volcanic sequences. Subaerial unconformities are time barriers. Regressive erosion surfaces are sharp, scoured diachronous surfaces that bound upward coarsening marine sequences. These bounding surfaces form during intervals of falling base level, marine regression, and glacial advance when fine-grained marine deposits are eroded and then overlain by coarser terrestrial or shoreface sediments migrating basinward.

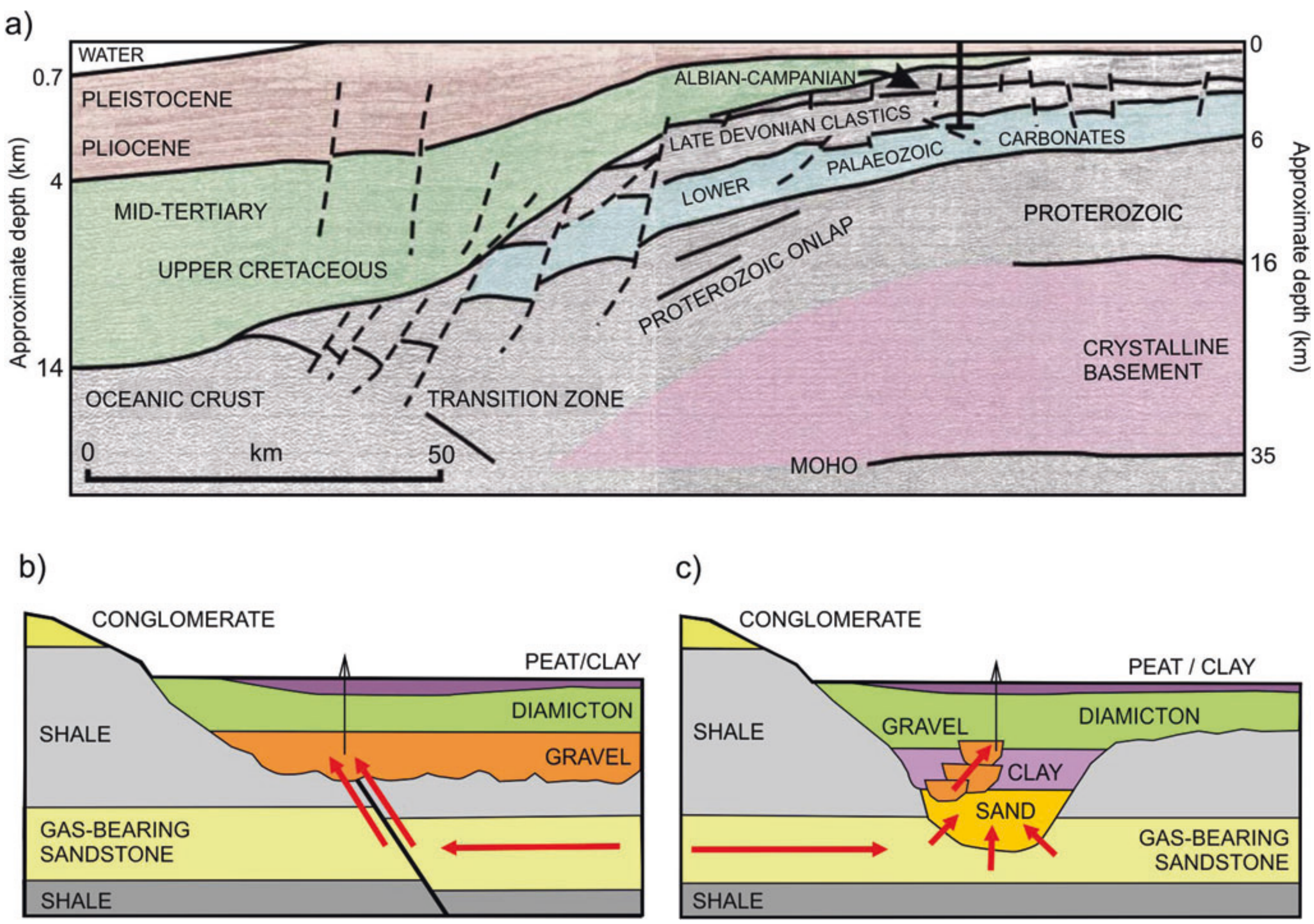
Ravinements are abrupt, scoured diastems, or unconformable surfaces in underlying nonmarine or marine strata, formed by fluvial, mass-wasting, wave, and tidal processes. These surfaces represent a change from deposition to nondeposition. In former coastal areas, brackish or marine strata overlie these bounding surfaces, indicating deposition during intervals of rising base level during marine transgression, lake formation, or glacial advance.

Maximum regressive surfaces transition laterally from shoreline ravinements and are conformable horizons or diastems in marine or lacustrine strata marking a change from upward coarsening shallow-water facies to upward fining deep-water facies. Forming at the end of marine regression or glaciation when the rate of base level rise exceeds that of sediment supply to shorelines, these surfaces are useful sequence boundaries for regional correlation in clastic sedimentary and carbonate rocks, and unconsolidated surficial deposits.

Maximum flooding surfaces are marked by a change from upward fining to upward coarsening strata, that indicate the end of marine transgression and change from deepening to shallowing water. Nearshore, these bounding surfaces are diastems or conformities, whereas offshore, they are unconformities developed by minor scouring and sediment starvation. In glacial environments, the change from upward fining basal tills to upward coarsening meltout tills and outwash mark the onset of ice retreat from maximum extent.

Slope onlap surfaces are prominent unconformable surfaces developed on marine shelves and in lake basins during intervals of falling base level, marking an interval of sediment starvation and nondeposition, or erosion by scour and mass-wasting. Slope onlap surfaces are unconformities and time barriers, with strata below older than above. As such, onlap surfaces are also useful for correlation and chronostratigraphic analysis.

Reservoir analyses and resource assessments of hydrocarbon plays, salt deposits, and groundwater aquifers are defined by their mapped field relationships, petrographic, lithological, biochemical, and geomechanical properties of hand specimens and core samples and interpretation of geophysical borehole logs and chronostratigraphic cross-sections. In stratigraphic and seismic sections, stratabound salt deposits are an indicator of maximum marine regression. Salt diapirs extruded through younger strata in response to tectonic stresses and isostatic adjustment cross-cut stratigraphic and geochronological boundaries. Together with anticlines and faults, salt deposits are important features to identify in cross-sections since they provide fluid migration pathways and seals that trap hydrocarbons and groundwater. Isostatic adjustment is the transient (10^2–10^4 years) or long-term ($>10^5$ years) nonelastic response of the lithosphere to loading



Sequence Stratigraphy, Fig. 1 (**a**) Sequence stratigraphy of Geological Survey of Canada deep crustal profile FGP87-1 of the Arctic continental shelf, showing the extensional character of the continental margin and hydrocarbon-bearing rocks of Paleozoic age (modified from Dietrich et al. 1989). (**b**) Stratigraphic model for the emplacement of very shallow natural gas in a Quaternary-hosted reservoir where an incised paleovalley intersects a gas-bearing unit that allows gas to migrate into buried channel gravel. (**c**) Stratigraphic model for the emplacement of very shallow natural gas where bedrock provides a conduit for natural gas to migrate from gas-bearing units through the seal and trapped in basal gravels of the Quaternary sequence (modified from Hickin 2009). Shallow gas plays and artesian aquifers are restricted to stratigraphic sequences of buried subaerial fluvial and glaciofluvial units confined to paleochannels incised into Cenozoic subaerial erosion surfaces

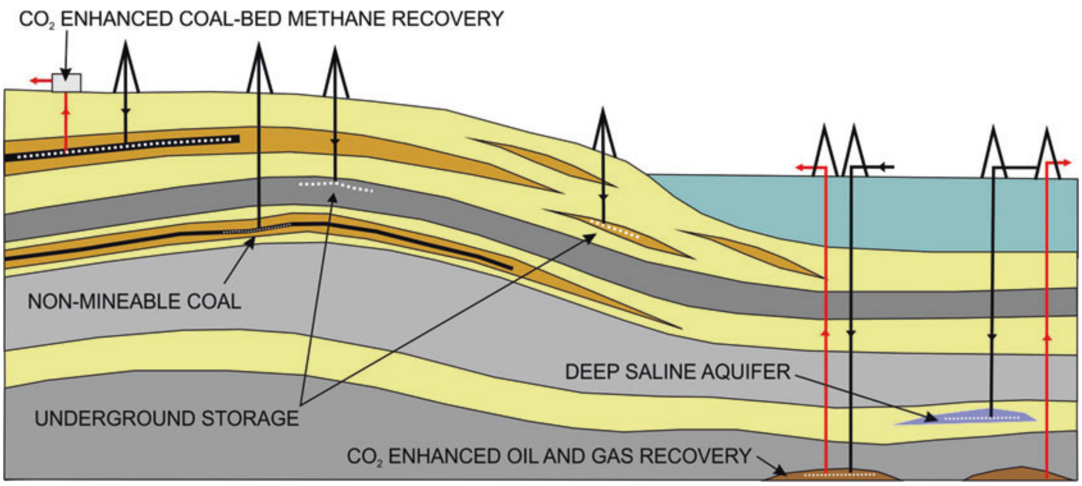
and unloading due to erosion, deposition, water loading, desiccation, and glaciation.

For well-site engineers and geologists, sequence stratigraphic principals are also used to predict facies changes when drilling into reservoirs in deep sedimentary basins with complex tectonic and sedimentary boundaries, or in drift-covered regions once covered by continental ice sheets.

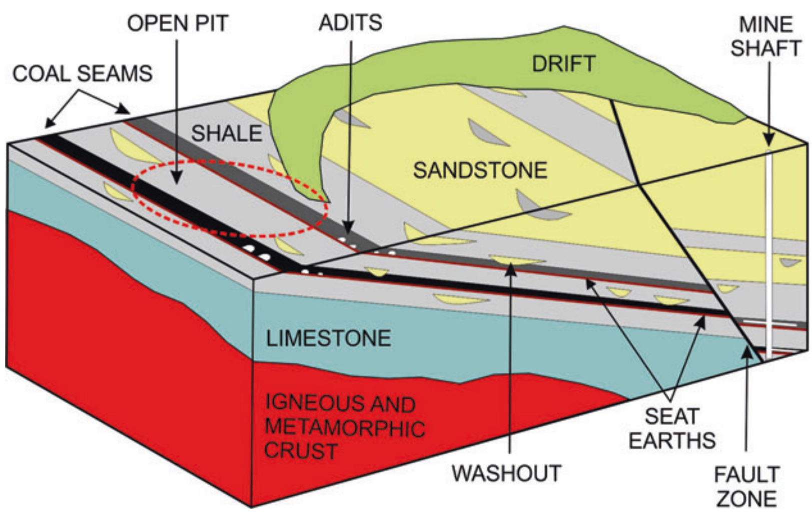
In unconventional shale gas reservoirs, engineers and geologists generally rely on geophysical and well-log data to delineate bounding surfaces between ductile clay mineral-rich and brittle silica/carbonate-rich units, and to identify groundwater sources for hydraulic fracturing (e.g., Hickin 2009; Baye et al. 2016; Nadeau et al. 2017). Reference cross-sections and chronostratigraphic analyses help optimize production and minimize operational costs by predicting variations in rock strength, fracture toughness, other geomechanical and petrophysical properties, and reservoir response to fluid injection during hydraulic fracturing, stimulation treatment, and gas withdrawal. Minor very shallow gas and groundwater reservoirs have been unexpectedly encountered, predicted, and successfully targeted in unconsolidated Quaternary deposits through sequence analysis. Shallow gas plays and artesian aquifers are restricted to stratigraphic sequences of buried subaerial fluvial and glaciofluvial units confined to paleochannels incised into Cenozoic subaerial erosion surfaces (Fig. 1b, c). Groundwater resource assessments, protection measures, and sustainable development plans are based on stratigraphic sequence models and hydrogeological maps that delineate the architecture, extent, and volume of regional aquifers and aquitards.

Geological CO_2 Sequestration

Underground storage of carbon dioxide is considered a long-term solution to reducing anthropogenic greenhouse gases



Sequence Stratigraphy, Fig. 2 Stratigraphic settings for geological sequestration of CO_2 in relatively undeformed sedimentary rocks; long-term storage options include coal seams, sandstones, and deep saline aquifers. Injection of CO_2 improves recovery of oil and gas from deep plays and coal-bed methane



Sequence Stratigraphy, Fig. 3 Lithostratigraphic factors influencing coal mining (modified from Open Learn 2018). Mechanized longwall mining is best suited for seams greater than a meter thick. Dip angle is an influence on adit and open pit design. Seat earths will cause heavy machinery to gouge seam floors, reducing quality of coal. Unstable roof conditions and groundwater inflow can occur where sandstone-filled paleochannels form washouts. Faults displace mineable seams and impose constraints on mechanized mine workings. Deformation, igneous activity, and metamorphism also adversely affect the quality of coal

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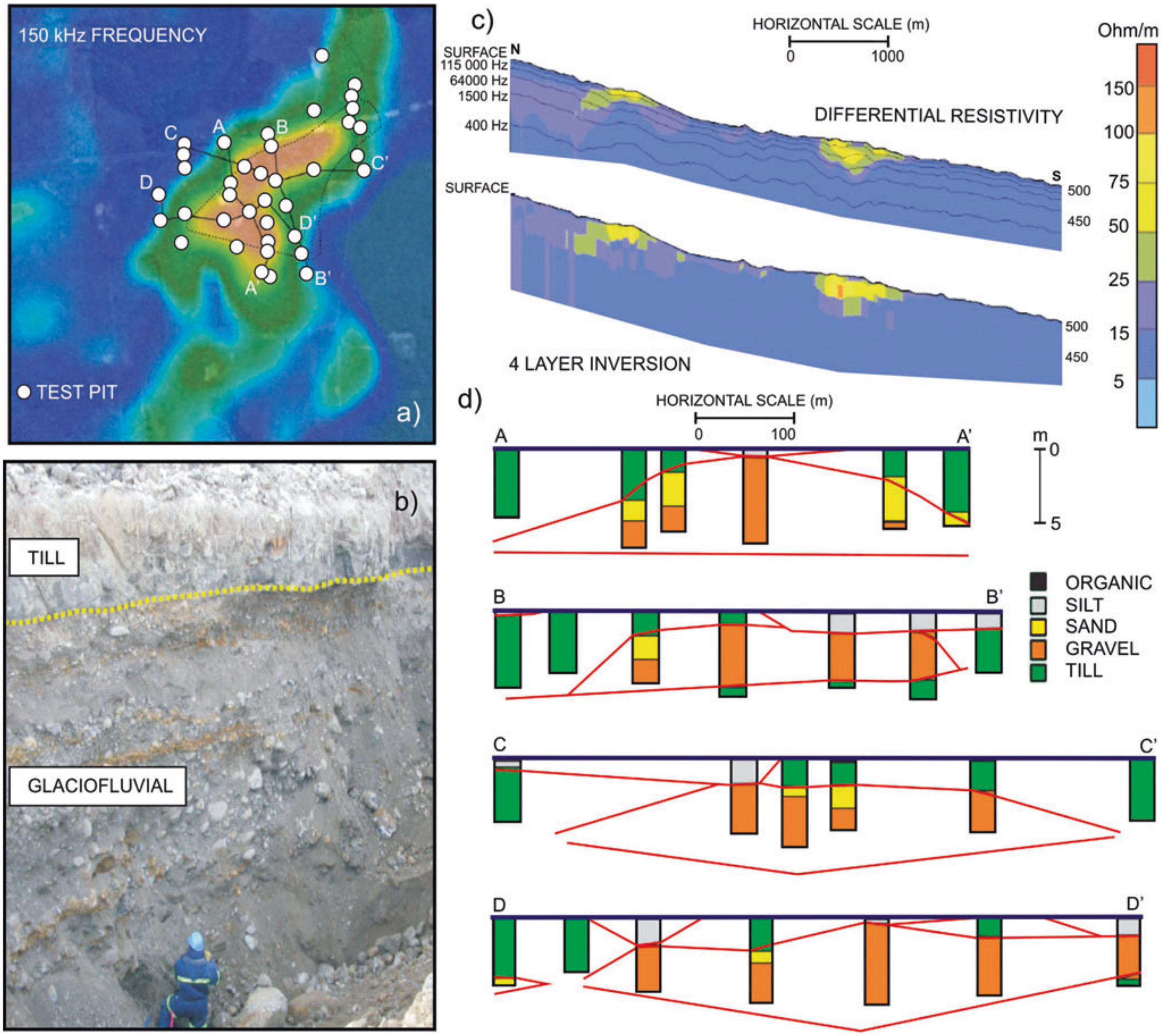
and mitigating the impacts of climate change. Anthropogenic greenhouse gases include: carbon dioxide (CO_2) from the burning of hydrocarbons, coal, and vegetation; methane (CH_4) from agriculture, landfills, hydrocarbon, and coal extraction; nitrous oxide (N_2O) from agriculture and industrial processes; and fluorocarbon gases (C_xF_y) as by-products of industrial activities.

Sequence stratigraphy is an important tool in the subsurface search for suitable and unsuitable lithostratigraphic or structural carbon sinks (Fig. 2). Robust experimental and numerical sequestration models are constrained by physical data on petrology, lithology, geological structures, geochemistry, reservoir quality, and depth of stratigraphic units (e.g., Bachu and Adams 2003; Lackner 2003; Figueroa et al. 2008; Ketzer et al. 2009; Saadatpoor et al. 2010). By predicting facies changes and sequence boundaries, engineers and geologists can design and operate injection wells in deep sedimentary basins with complex structurally controlled architecture and boundaries. Saline aquifers in basalt and sandstone formations are favorable storage targets. In these reservoirs, injected CO_2 reacts with calcium and magnesium silicate minerals to form carbonate minerals that are stable for geological periods of time. Coal seams can physically absorb large concentrations of injected CO_2 while also yielding commercial methane. Limestone are not suitable storage rocks because acidified injected brines can dissolve $CaCO_3$ and



Sequence Stratigraphy, Fig. 4 When combined with airborne EM and other geophysical techniques, sequence stratigraphy is an effective exploration tool used to identify and delineate buried aggregate sources in drift deposits. (**a**) High resolution airborne EM 150 kHz depth slice, with locations of test pits and cross-sections. (**b**) Aggregate deposit consists of glaciofluvial gravel underlying diamicton (till). (**c**) Pseudo-section of EM data indicating thickness of gravels (warm colors). (**d**) Stratigraphic cross-sections of the aggregate deposit based on test pit data (modified from Levson et al. 2006)

release CO_2 into the karst aquifer. Forward numerical sequence modelling can also help engineers and geologists predict where lithological conditions are unfavorable, preventing brine leakages or blowouts into overlying strata or release of CO_2 back into the atmosphere.

Mineral Exploration and Mining Engineering

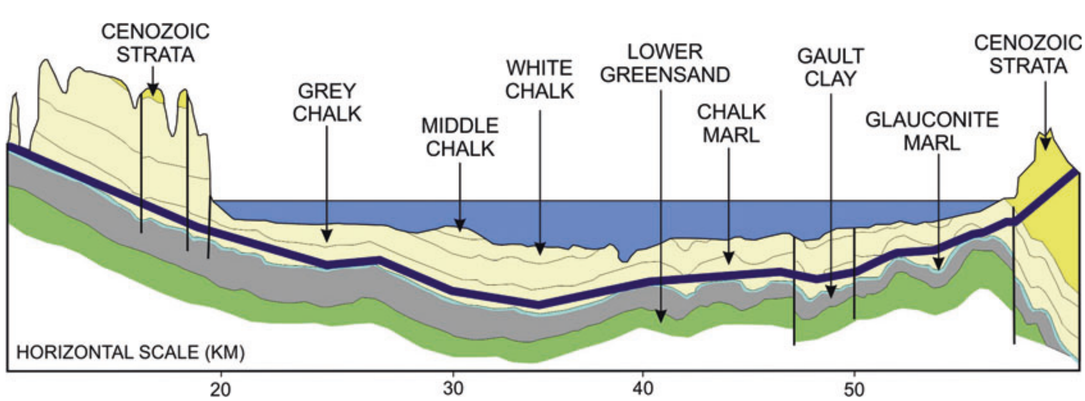
During mineral exploration, engineers and geologists rely on sequence stratigraphy to identify and predict types and thickness of ore deposits in rock units, drift, and soils (e.g., Garven and Freeze 1984; Burkhalter 1995; Abdulkader et al. 2007; Myagkiy et al. 2017; Zhang et al. 2017). Mineral occurrences and ore bodies hosted in sedimentary rocks can be stratabound or occur at weathering surfaces coincident surface unconformities and include coal, salt, uranium, iron, aluminum, other ores, rare earths, and placer deposits (Fig. 3). Petrographic, geochemical, and lithological descriptions, field relationships, borehole logs, geophysical data, core samples, and chronostratigraphic cross-sections are evaluated by engineers and geologists to identify ore-bearing stratigraphic units and their bounding surfaces in surface pits and underground mines. Geological factors controlling selection of mining method (Fig. 3) include the nature of and lateral variations in mineralization, rock types, dip of strata, presence of faults, folds, and other deformation features, intrusions, and groundwater. Through the operational lifespan and during remediation, mine-site damage, tailings failures, environmental contamination, and loss of life are minimized when competent rock or weak beds, faults, fractures, and folds, permeable, and porous units are identified through detailed surface and subsurface stratigraphic analysis and sequence modelling.

Natural Resources Infrastructure and Surface Engineering

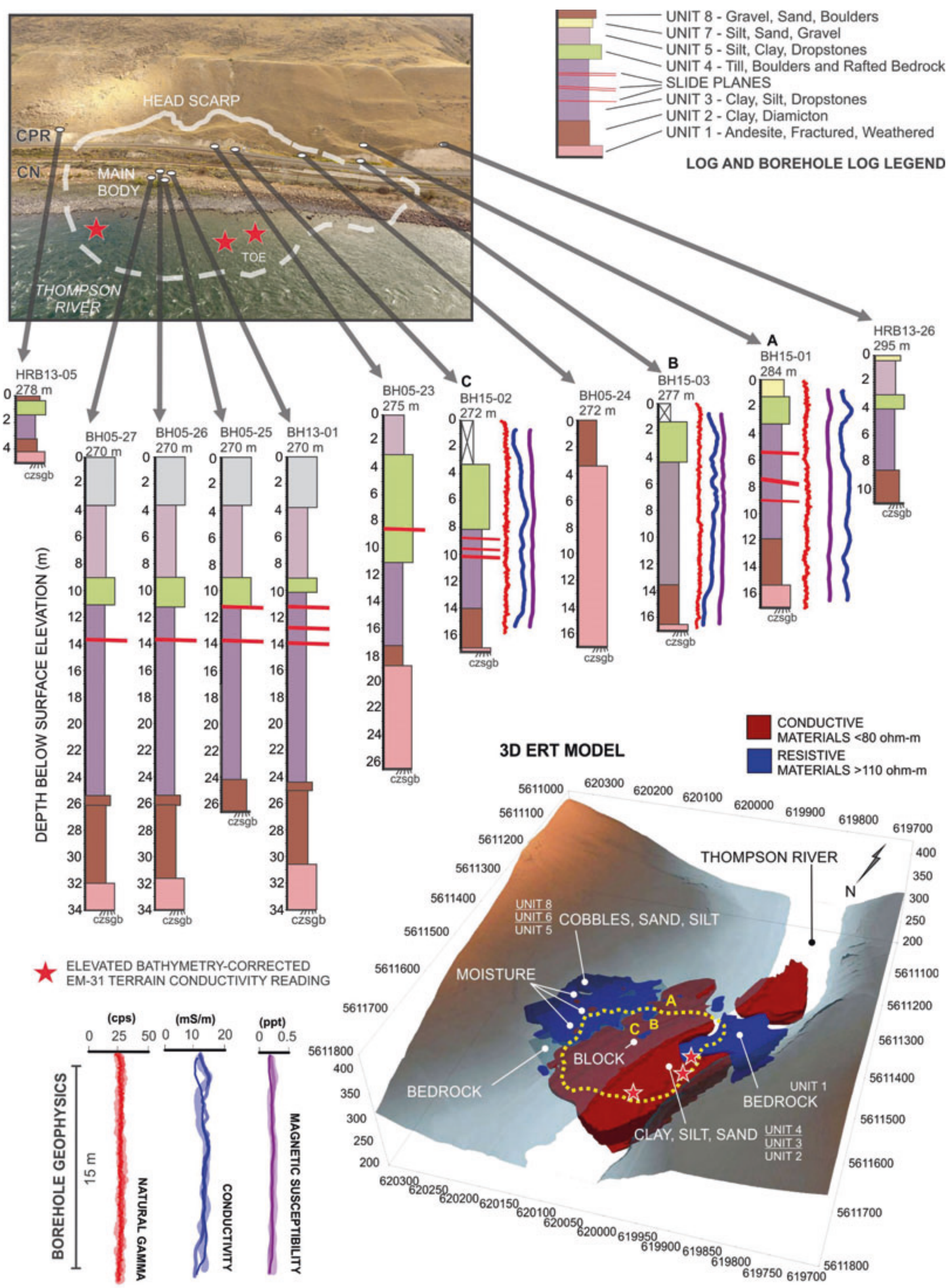
Dams, mines, pipelines, bridges, roads, and railways are essential infrastructure in regions endowed with energy, mineral, and water resources (e.g., Kim 2001; Levson et al. 2006; Hickin 2009). Repetitive episodes of sedimentation along major rivers during the Quaternary Period have produced fluvial aggregates preferred by construction projects. In clay-rich glaciated terrains, sequential changes in ice-advance and ice-retreat facies logged at depth and mapped at surface are used to predict and locate sources of gravel (Fig. 4). Exploration for aggregate sources, crushed rock, and building materials relies on interpretation of ground-based observations, material samples, aerial photography, photogrammetry, light detection and ranging, InSAR and multispectral imagery to map the surface expressions and textures of bedrock, surficial units, soils and their bounding subaerial unconformities, diastems, and conformable surfaces. Airborne and ground-based geophysical methods including frequency domain electromagnetics, electrical resistivity tomography, and ground penetrating radar are commonly employed techniques used to cover relatively large areas with vertical depth of penetration on the order of several tens of meters (Fig. 4). From a geological engineering perspective, mineral and energy resource projects rely on sequence stratigraphy to predict types and thicknesses of rock strata, unconsolidated deposits and soils encountered during construction, operation, and decommissioning of surface infrastructure, and to identify potential geohazards during environmental impact assessments.

Tunnelling and Subsurface Engineering

Sequence stratigraphy is an important tool in the planning, design and construction of tunnels, and the underground infrastructure that support military installations, hydroelectricity generation, hydrocarbon and nuclear waste storage, mining and ore-processing, potable water transfer, sewage treatment, and dwellings (Eisenstein 1994; Warren and Mortimore 2003; Zarei et al. 2012; Lui et al. 2015; Filbà et al. 2016; Scheidler et al. 2017). Subsurface exploration for underground routes and project sites are selected following detailed analysis of stratigraphic cross-sections based on mapped field relationships and correlation of borehole logs and cored materials. These data sources provide important information by characterizing changes in petrographic, lithological, biochemical, geophysical, and geomechanical properties at depth along tunnel routes. The spatial relationships and correlation between facies and bounding surfaces in outcrop, borehole sections, and core samples are used to predict the geometry of fractured, permeable, and porous units through which tunnelling will take place (Fig. 5). Whether in surficial deposits or bedrock, precise bed-by-bed lithostratigraphic descriptions from cored boreholes aid the engineering description, classification, and numerical modelling of tunnelled media, design of tunnelling machines, and construction methods, specifications, and monitoring. Correlation of marker beds and bounding surfaces between boreholes and in outcrop helps define the regional erosion surfaces and faulting patterns. These influence the engineering properties and preservation of the different stratigraphic units hosting transportation tunnels and underground spaces



Sequence Stratigraphy, Fig. 5 Simplified geological section of the Channel Tunnel from portal (UK, left) to portal (France, right), showing route in relation to major stratigraphic units. Tunnelling was mostly confined to the chalk marl lying above the Gault Clay marker bed. This stratal layer has a low permeability and is less fractured than overlying chalk. Figure modified from Geological Society (2018)



Sequence Stratigraphy, Fig. 6 Lithostratigraphic controls on landslide form and function (modified from Huntley et al. 2017). Terrain mapping, borehole logs, and electrical resistivity surveys reveal a 30-m deep bedrock basin infilled with sequences related to the advance and retreat phases of glaciation. Slope movement occurs along sub-horizontal shear planes in sub-till clay deposits. Glacial deposits are stratified, locally deformed, and cross-cut by vertical fractures. Slide activity is driven by fluvial erosion of the toe slope, variations in porewater pressure across the main body, and soil moisture inputs upslope

(Fig. 5). Geomechanical problems during and after tunnel construction are often the result of groundwater circulation. Stratigraphic and hydrogeological models make it possible to undertake sensitivity analyses and test how changes in boundary conditions and hydraulic properties influence calculated groundwater flow regimes.

Geohazards and Slope Engineering

Landslides, earthquakes, tsunamis, floods, and other geohazards challenge the development and maintenance of safe and resilient communities, and infrastructure on land and at sea (e.g., Eisenstein 1994; Gee et al. 2006; Huntley and Bobrowsky 2014; MacDonald et al. 2017). Engineers and geologists are tasked with understanding landslide form and function, and designing monitoring instrumentation and mitigation measures for unstable terrestrial slopes and infrastructure. Landslides, ranging from rapid rock falls to slow-moving earthflows, are influenced by many geological factors. Lithology, bedding and deformation structures, geochemistry, hydrogeology, chronostratigraphy, and facies relationships control the distribution of porosity, permeability, and geomechanical strength of failing rock units, surficial deposits and soil types. Engineering descriptions, classifications, numerical landslide models, and mitigation solutions are all aided by analysis of outcrop relationships, laboratory tests on core samples, interrogation of borehole logs, and a range of geophysical data and hydrogeological cross-sections (Fig. 6).

In coastal zones, landslides, floods, earthquakes, and tsunamis are significant challenges to resilient infrastructure and safe communities (Clague and Bobrowsky 1994; Hutchinson and Clague 2017; Shaw et al. 2017). Forward modelling of geohazard events based on stratigraphic evidence preserved in salt marsh peats and forest soils near sea level assist engineers and geologists with construction designs, environmental impact assessments, implementing remediation measures and undertaking risk analyses. In the offshore, submarine slides, slumps and shallow gas can be a challenge for projects developing shallow hydrocarbon resources, nearshore structures, and submarine transportation tunnels (Eisenstein 1994; Bünz et al. 2003; Gee et al. 2006; Lamb et al. 2017; Shaw et al. 2017). It is important for engineers and geologists to identify and predict the mode of failure and geometry of shallow slope failure surfaces in Quaternary and older marine deposits since they control the migration paths and trapping of shallow gas hydrates and groundwater. In stratigraphic cross-sections, submarine failure planes are diachronous regressive surfaces of erosion and shoreline ravinements formed during intervals of glaciation and falling sea level. In borehole logs and seismic data, these scoured surfaces are sharply overlain by glacial age coarse-grained slide debris and postglacial muds.

Summary

Sequence stratigraphy is a multidisciplinary tool with numerous engineering applications. Predictive modelling of stratigraphic sequences allows engineers and geologists to understand the controls on deposition and erosion in a sedimentary basin and to correlate and forward predict stratigraphic patterns as they appear in outcrops, borehole logs, and cross-sections. This improved geological understanding helps to minimize exploration, development, and landscape restoration risks in resource-endowed areas and to ensure that energy, mining, and infrastructure projects and communities are safe, resilient, and environmentally sustainable.

Cross-References

▶ Acid Mine Drainage
▶ Aerial Photography
▶ Aeromagnetic Survey
▶ Aggregate
▶ Aquifer
▶ Aquitard
▶ Artesian
▶ Borehole Investigations
▶ Bridges
▶ Building Stone
▶ Cap Rock
▶ Characterization of Soils
▶ Classification of Rocks
▶ Classification of Soils
▶ Climate Change
▶ Coal
▶ Coastal Environments
▶ Cross Sections
▶ Crushed Rock
▶ Dams
▶ Designing Site Investigations
▶ Drilling
▶ Drilling Hazards
▶ Earthquake
▶ Engineering Geological Maps
▶ Engineering Geomorphological Mapping
▶ Engineering Properties
▶ Environmental Assessment
▶ Environments
▶ Erosion
▶ Evaporites
▶ Facies
▶ Faults
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- ▶ InSAR
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- ▶ Shale
- ▶ Site Investigation
- ▶ Soil Properties
- ▶ Subsurface Exploration
- ▶ Tailings
- ▶ Tsunamis
- ▶ Tunnels
- ▶ Wells

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Shale

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Synonyms

Clayshale; Fissile mudrock; Fissile mudstone; Mudshale

Definition

Shale is a fine-grained, siliciclastic, laminated sedimentary rock composed mainly of indurated silt and clay size particles of clay minerals, fine-size quartz, and feldspar.

Characteristics

Shale is the most frequently encountered rock in engineering projects associated with sedimentary basins. Sedimentary basins, having low water energy, provide suitable conditions for deposition of fine-grained suspended particles to form shale. The presence of fissility, the ability of a rock to readily split into thin pieces with a maximum thickness of 10 mm, along laminations (Czerewko and Cripps 2012) is the key indicator to distinguish shale from other mudrocks such as mudstone and claystone. The predominant grain size of shale is smaller than 0.063 mm. In addition to clay minerals, fine-size quartz and feldspar, carbonate minerals (calcite, dolomite), pyrite, heavy minerals, and different amounts of organic particles are also present in shale. Shale is found in many different colors: black, gray, red, light brown, and dark brown, depending on the percentages of constituents. Color is a distinctive characteristic that helps to determine depositional environment and composition of a shale. Black color reveals deposition in an oxygen-deficient environment,-providing proper conditions to prevent weathering of organic materials, whereas red color indicates deposition in oxygen-rich conditions containing iron oxide or iron hydroxide minerals.

Physical and mechanical properties of shale generally indicate anisotropic behavior, depending on dip and dip direction of lamination. In addition, shale exhibits a wide range of strength and deformability properties depending on geological age, mineralogical composition, lithological characteristics, and degree of induration. Furthermore, the strength and deformability properties of less durable shale containing relatively high amounts of swelling clay minerals deteriorate dramatically with increasing water content. The slaking behavior of shale, as a result of interaction with water, is another commonly encountered problem. Slaking behavior is responsible for various geotechnical problems such as slope instability, embankment failures, and surface mine highwall failures (Dick et al. 1994). In addition, Steiger and Leung (1992) emphasized that shale form more than 75% of drilled formations and at least 90% of wellbores' instabilities during shale gas exploration can be attributed to inherent nondurable, water sensitive characteristics of shale bearing formations.

Shale is a very significant rock in context of their importance for conventional oil, unconventional oil and natural gas productions, and raw material to produce clay and cement. Organic shale, containing oil and natural gas, have provided, directly or indirectly, adequate energy for industrialization, engineering, and technological advancements for decades. Permeability values for shale range between 0.01 and 100 nd at atmospheric pressure (Freeze and Cherry 1979). However, current advancements in horizontal drilling technology and hydraulic fracturing have made it possible to extract oil and gas in large quantities from shale deposits. Further research may determine if low permeability and good self-sealing behavior may show shales to be a host rock for many modern engineering projects such as nuclear waste disposal (Fisher et al. 2013) and sequestration of CO_2.

Cross-References

▶ Clay
▶ Expansive Soils
▶ Hydraulic Fracturing

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Shear Modulus

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Definition

Shear modulus (G) is the ratio of shear stress (τ) to the corresponding shear strain (γ) produced by it. It is sometimes called the modulus of rigidity.

The generalized Hooke's law can be written to include shear stresses and shear strains, and where the normal stresses (σ) are not necessarily principal stresses:

$$\varepsilon_x = \frac{\sigma_x}{E} - \nu\frac{\sigma_y}{E} - \nu\frac{\sigma_z}{E} \tag{1a}$$

$$\varepsilon_y = -\nu\frac{\sigma_x}{E} + \frac{\sigma_y}{E} - \nu\frac{\sigma_z}{E} \tag{1b}$$

$$\varepsilon_z = -\nu\frac{\sigma_x}{E} - \nu\frac{\sigma_y}{E} + \frac{\sigma_z}{E} \tag{1c}$$

$$\gamma_{xy} = \frac{\tau_{xy}}{G} \tag{1d}$$

$$\gamma_{xz} = \frac{\tau_{xz}}{G} \tag{1e}$$

$$\gamma_{yz} = \frac{\tau_{yz}}{G} \tag{1f}$$

where E is Young's modulus and ν is Poisson's ratio.

For biaxial stress with σ_1 as maximum and σ_2 as minimum, with $\sigma_3 = 0$, the strains (ε_1, ε_2, ε_3, γ_{xy}) are

$$\varepsilon_1 = \frac{\sigma_1}{E} - \nu\frac{\sigma_2}{E} \tag{2a}$$

$$\varepsilon_2 = -\nu\frac{\sigma_1}{E} + \frac{\sigma_2}{E} \tag{2b}$$

$$\varepsilon_3 = -\nu\frac{\sigma_1}{E} - \nu\frac{\sigma_2}{E} \tag{2c}$$

$$\gamma_{xy} = \frac{\tau_{xy}}{G} \tag{2d}$$

For pure shear, $\sigma_1 = +|\tau_{xy}|$ and $\sigma_2 = -|\tau_{xy}|$; so the principal strains ε_1 and ε_2 can be written as

$$\varepsilon_1 = \frac{\tau_{xy}}{E} + v\frac{\tau_{xy}}{E} = \frac{\tau_{xy}}{E}(1+v) \tag{3a}$$

$$\varepsilon_2 = -v\frac{\tau_{xy}}{E} - \frac{\tau_{xy}}{E} = -\frac{\tau_{xy}}{E}(1+v) \tag{3b}$$

The magnitude of the maximum shear strain is the diameter of Mohr's circle for strain, which is $2 \times \varepsilon_1$, thus

$$\gamma_{\max} = \gamma_{xy} = 2\varepsilon_1 \tag{4}$$

Substituting values from Eqs. 2d, 3a, and 4,

$$\frac{\tau_{xy}}{G} = \frac{2\tau_{xy}(1+v)}{E} \tag{5a}$$

$$\frac{1}{G} = \frac{2(1+v)}{E} \tag{5b}$$

$$G = \frac{E}{2(1+v)} \tag{5c}$$

Shear modulus can be calculated from two basic elastic properties: Young's modulus and Poisson's ratio.

A dynamic shear modulus (G_d) can be calculated using geophysical measurement of shear-wave velocity (Vs) and estimates of mass density (ρ) representative of the volume of Earth materials included in the geophysical measurement.

$$G_d = \rho {V_s}^2 \tag{6}$$

where ρ is in mass units (kg/m^3) and Vs is in velocity units (m/s), which gives G_d in MPa or GPa. The geophysical measurements reflect rock mass properties, including rock structure (bedding, faults, joints, and fractures). Shear waves are not affected by water saturation, whereas compression waves have a characteristic velocity in saturated granular materials of about 1.48–1.50 km/s, which should be considered if geophysical measurements of compression and shear-wave velocities are used to calculate a dynamic Poisson's ratio.

Cross-References

- ▶ Expansive Soils
- ▶ Hooke's Law
- ▶ Poisson's Ratio
- ▶ Stress
- ▶ Young's Modulus

Shear Strength

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Definition

Shear strength is the maximum magnitude of shear stress that a material can withstand.

Introduction

The following describes the frictional nature of soils and the stress states and parameters used to describe the strength. Followed by a description of the effect of the pore water pressure developed within the soil, the drainage of this pore pressure, and large amounts of strain.

Shear Strength of Soils

The shear strength of a soil is a result of inter particle friction, with a component of cohesive strength. This shear strength (S) is most commonly represented by a linear Mohr-Coulomb failure surface.

$$S = c + \sigma \tan \phi$$

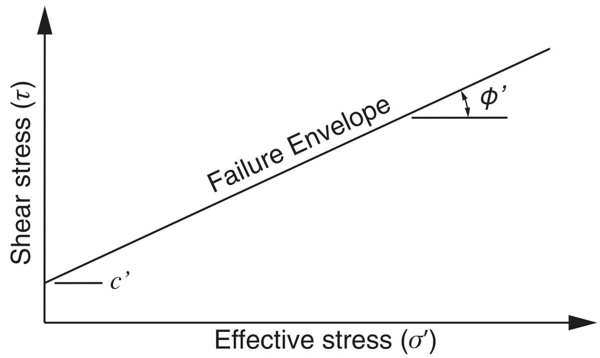
where, σ is the stress normal to the plane on which the shear stress is evaluated, and c and ϕ are material properties which represent the component of the strength resulting from cohesion and the angle of friction, respectively. ϕ is the angle between the direction of σ and the direction of the resultant of σ + S (Terzaghi et al. 1996).

S

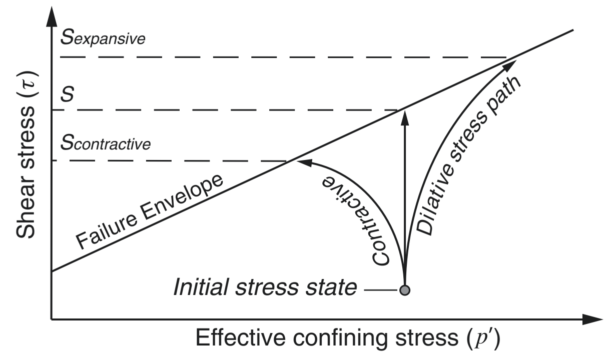
Effective Shear Strength

When the voids between soil particles are filled with water the pressure of the water (u) reduces the stress applied to the soil structure, and thus reduces frictional strength that can be developed from inter-particle friction. For this common case, the strength is a result of this reduced inter-particle stress, or the effective stress (σ'), and effective material strength properties for the cohesion (c') and angle of friction (ϕ'). The Mohr-Coulomb failure surface in terms of σ', c', and ϕ'.

$$S = c' + \sigma' \tan \phi'$$



Shear Strength, Fig. 1 Mohr Coloumb failure envelope defined for effective stress parameters



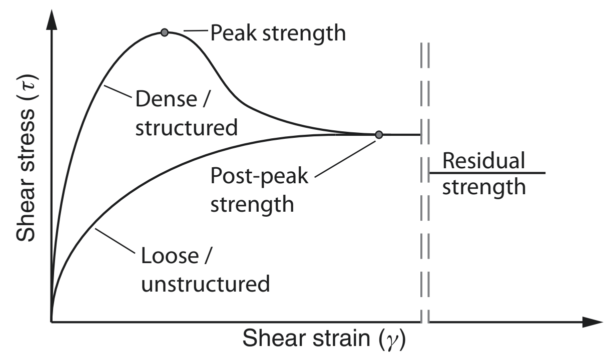
Shear Strength, Fig. 2 Effect of the effective stress path on the development of shear strength (S) for both contractive and dilative soils

The cohesion term for both σ and σ′ formulations of S may be the result of cementation between soil particles. However, this is often referred to as the "apparent cohesion" as when soils without cementation are tested at very low values of σ′ the values of c′ also become very low and is thus only apparent within a range of σ′. In such cases, much of the magnitude of c′ can be attributed to the result of fitting a linear Mohr Coulomb failure envelope to nonlinear or bimodal failure envelopes (Fig. 1).

Result of the Effective Stress Path on Effective Shear Strength

The development of shear strain results in a change in u. Dense soils, such as overconsolidated clay and heavily compacted sand tend to dilate with shear strain resulting in a reduction of u (Fig. 2). Normally consolidated clay and quick clay may exhibit a contractive behavior as they shear which results in an increase in u (Fig. 2). As σ′ is a direct result of u, then the strength of a soil, with a constant σ, can increase or decrease with shear strain.

A very low rate of shear allows the change in u, due to strain, to dissipate resulting in a constant S for a constant σ. This rate of shear is referred to as the drained condition, and the constant S as the drained strength. The limiting rates of shear for a drained condition are a function of the permeability of the soil and the breadth of the material that is undergoing shear. Faster rates of shear result in a change of u and a corresponding change in S for a constant σ. This rate of shear strain is referred to as an undrained condition, and a change in the S to the undrained strength (either $S_{contractive}$ or $S_{expansive}$, Fig. 3). Thus, the undrained strength for a normally consolidated or quick clay is typically less than the drained strength, and the undrained strength for an overconsolidated clay may be higher than the drained strength (Figs. 2 and 3) (Terzaghi et al. 1996).



Shear Strength, Fig. 3 Common stress-strain response behaviors showing strain hardening behavior with a peak and post-peak strength, and a simple strain hardening material

Effect of Strain on the Achievable Shear Strength

The assignment of a value of S to a stress-strain response requires consideration for the stress strain response of the soil (Fig. 3). For an intact material, the strength is often the peak shear stress achieved during the shearing of a specimen with a constant rate of strain. For soil structure that will undergo localized yielding, the strength within the yielded zone will be limited to a lower post-yield strength. Soil that has been subjected to large shear strains, such as an existing failure surface within a landslide, the strength that can be mobilized reduces further to a residual strength defined by a residual effective angle of friction (ϕ_r') (Duncan, 2014).

Summary

The shear strength of a soil is a result of inter-particle friction and is dependent on the magnitude of the normal or confining stress. When the voids between soil particles are filled with water the pressure of this water reduces inter-particle stress and thus the confining stress and strength is determined by effective stress. Shear stress may result in further changes in pore pressure, this can be either positive or negative depending on the stress history and structure of the soil. The drainage conditions during this shearing determine if these pore pressures affect the effective stress and thus strength. The shear strength changes with the amount of shear strain that the soil has been subjected to, thus potentially having distinctive peak, post-peak, and residual strengths.

Cross-References

- ▶ Dilatancy
- ▶ Mohr-Coulomb Failure Envelope
- ▶ Shear Stress
- ▶ Stress

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Shear Stress

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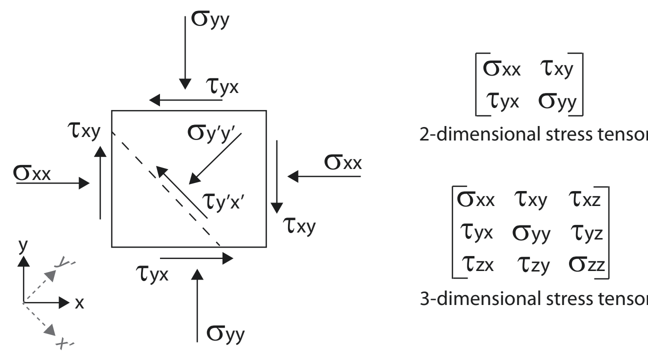
Definition

Shear stress is the component of the stress tensor, at any given point within a rock or soil mass, which is acting on the plane of interest that passes through that point.

Overview

The stresses acting at any point and on any arbitrary plane within a rock or soil mass can be expressed in terms of the stress vector normal to the plane and the stress vector parallel to the plane (Fig. 1) (Jaeger et al. 2007). The stress component acting on the arbitrary plane is the shear stress for that particular plane of reference. This plane of reference can be an imaginary section of the soil or rock mass or may represent a real boundary or discontinuity (joints, sliding surfaces).

For a given set of internal and external forces, the shear stresses on a plane depends on the location (position vector) and direction of the plane (its outward unit vector). However, the state of stresses at any given point is uniquely defined by a



Shear Stress, Fig. 1 Illustration of stress components, in two dimensions (plane stress condition), parallel and normal to the planes is defined by the xy coordinate system and on a rotated plane defined by the x′y′ coordinate system; the positive z axis is pointed out of the page toward the viewer. Also shown are the stress tensors in two and three dimensions. The letter τ denotes shear stresses whereas σ denotes normal stresses

second-order tensor (stress tensor), which is independent of the direction of any plane of reference. The shear (and normal) stresses at a particular point can then be calculated for any plane of interest, from the stress tensor at that point. The stress tensor will vary for different points within the rock or soil mass. The plane on which shear stress is zero will correspond to maximum and minimum normal stresses, which are the principal stresses.

The convention for positive normal and shear stresses is given in Fig. 1. In engineering geology applications, the stress state is commonly visualized using the Mohr circle. Here, normal and shear stresses are readily associated with any arbitrary plane cutting the rock or soil element, and the principal stresses are defined by the Mohr circle intersecting the normal stress axis. The relationship between normal and shear stresses becomes important for shear strength determination, particularly when evaluating the stress/strength state along discontinuities and sliding surfaces.

Cross-References

- ▶ Effective Stress
- ▶ Mohr Circle
- ▶ Mohr-Coulomb Failure Envelope
- ▶ Normal Stress
- ▶ Shear Modulus
- ▶ Shear Strength
- ▶ Stress

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Shear Zone

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Definition

The zone of concentrated deformation within a rock or soil mass. The shear zone is characterized by the relative movement of rock wall segments (in rock masses) or soil particles with respect to each other. Usually associated with fully formed discontinuities, rock fragmentation, and soil softening.

Overview

Concentration of strains within a shear zone can lead to a ductile rearrangement of rock wall segments (or soil particles) within the zone, or to brittle behavior characterized by the formation of one major discontinuity and several secondary discontinuities. At large strains within rock masses, shear zones can be composed of rock fragments, crushed by the continuous deformation, and with varying soil content. Within soil masses, shear zones can be characterized by frequent fissures, and in some cases softening of the material (clays) and particle crushing (sand).

Shear zones can be observed at geologic scales such as in the vicinity of major discontinuities (i.e., faults), at engineering scales such as in landslide phenomena, and in laboratory scale (i.e., triaxial and shear testing). These zones are associated with degradation of engineering properties and different hydraulic conductivity when compared to the more competent strata (i.e., shear modulus, shear strength) (Gylland et al. 2013, Passchier and Trouw 2005).

Cross-References

▶ Deformation
▶ Faults
▶ Hydraulic Action
▶ Landslide
▶ Rock Laboratory Tests
▶ Shear Modulus
▶ Shear Strength
▶ Soil Laboratory Tests

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Shotcrete

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Definition

Shotcrete is the name used to refer to wet concrete that is pneumatically projected onto a prepared surface at high velocity through a nozzle.

Applications for shotcrete in engineering geology tend to be cut slopes that are marginally stable or that will tend to degrade with time by sloughing, slaking, or raveling. The surface on which the shotcrete will be applied needs to be prepared with drainage for positive relief of hydrostatic pressures and reinforcing steel bars or mesh, if needed. Steel fibers may be added to the shotcrete mix for reinforcement in lieu of applying reinforcing steel bars or mesh.

The wet concrete mix design must meet certain specifications and be deliverable at high velocity through a nozzle (Morgan and Totten 2008). Cementing materials are Portland cement plus optional fly ash and silica fume additives. Water must meet drinking water standards. Mineral aggregates must be normal weight and meet the durability and alkali reactivity requirements for conventionally placed concrete. The maximum coarse aggregate size is nominally 10–12.7 mm (3/8–1/2 in.), depending on jurisdictional and application-thickness requirements. A gradation envelope for well-graded coarse aggregate must be met. The maximum water-to-cementitious material ratio is 0.45. Air content as shot is to be $4 \pm 1\%$. Slump at discharge into the pump hopper is to be 60 ± 20 mm ($2–1/2 \pm 1$ in.). The minimum compressive strength is to be 20 MPa (7-day cure time) and 30 MPa (28-day cure time).

The applying nozzle should be held approximately perpendicular to the receiving surface and at a distance that allows the air volume to produce shotcrete that has maximum consolidation and complete encapsulation of reinforcing steel. A Nozzleman's helper is needed to remove shotcrete rebound and overspray material and to help ensure that shotcrete builds up from behind reinforcing steel to encase it.

Shotcrete must be kept moist for a minimum 7-day cure time. General work requirements for placement and curing of conventional concrete under hot and cold weather conditions must be followed for shotcrete to ensure acceptable performance.

Cross-References

- ▶ Aggregate
- ▶ Cement
- ▶ Concrete
- ▶ Landslide
- ▶ Mass Movement

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Silt

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Definition

A loose granular substance resulting from natural erosion or from splitting of larger rock and sand particles and having two meanings: a textural class if used in terms of aggregates of silt-sized grains; or a size class if used in terms of a single grain.

The two most widely used size classification schemes to define grain size are the Unified Soil Classification System (USCS), ISO 14688, and the Krumbein phi scale (Tables 1 and 2). The Unified Soil Classification System combines silt with clay into fine-grained sediment (often referred to as "mud") but distinguishes the two based on their plasticity and organic content. ISO 14688 grades silt between 0.002 and 0.063 mm. The Krumbein phi scale defines silt as particles 0.0039–0.0625 mm. There are other less common classification systems. As a textural class, the soil or sediment beds can be called silt if the silt content is greater than 80% (ASTM 2000).

Silt particles are relatively spherical. This means they have a slippery consistency. As a general rule, dry silt feels like flour when rubbed between the fingers, it is not sticky like clay and not gritty like sand. When wet, it is more difficult to roll into a string than clay. It is able to promote water retention, but at the same time is able to allow some air and water circulation (Moss and Green 1975).

Unlike sand and gravel, silt is not used in large quantities in the construction industry, due to its water retention characteristic and slippery nature. Some silt may be present in cement or mortar, but it weakens the product. In many environmental applications, the presence of silt is often considered to be negative. Natural siltation can fill in navigation areas such as lakes and harbors, and may require costly maintenance dredging. Uncontaminated dredged silt can be used as soil conditioners for farming. Often humans cause increased siltation near water bodies which can clog the breathing capability of aquatic plants and animals. Similarly, silt can block water intakes required by coastal infrastructure.

On the beneficial side, silt is very fertile and useful in agriculture. This is due to its balanced water retention and circulation properties, and ease of tilling (spherical particles). Some of the very first civilizations, including the Mesopotamians and the ancient Egyptians, owe their success to proximity to large fertile silt beds. An example of a modern day silt-dwelling civilization is on the Chars of Bangladesh. Chars are low-lying silt bars in Bangladeshi rivers which host over 600,000 people. The economics of the char lands are largely based on agriculture, fishing, and livestock-rearing. An ambitious engineering geology application aims to create dams to intercept and redirect large quantities of silt over the next 20 years to reclaim a total of 10,000 square kilometers of land. The captured silt will also be used to raise the existing land to mitigate against sea level rise (Islam 2015).

Silt is most commonly composed of silica (SiO2) and feldspar. Other minerals may be present. The main shaping process for silt is abrasion. This may be through fluvial, glacial, or aeolian (wind-blown) processes. Silt produced and carried through aeolian processes is termed loess. Silt is

Silt, Table 1 Silt sizes according the Unified Soil Classification System

Fine grained soils 50% or more passing the No. 200 (0.075 mm) sieve	Silt and clay liquid limit <50	Inorganic	ML	Silt
			CL	Clay of low plasticity, lean clay
		Organic	OL	Organic silt, organic clay
	Silt and clay liquid limit ≥ 50	Inorganic	MH	Silt of high plasticity, elastic silt
			CH	Clay of high plasticity, fat clay
		Organic	OH	Organic clay, organic silt

Silt, Table 2 Silt sizes according the Krumbein phi scale

Type	Φ	Size in mm
Very fine silt	8 to 7	1/256 to 1/128
Fine silt	7 to 6	1/128 to 1/64
Medium silt	6 to 5	1/64 to 1/32
Coarse silt	2 to 3	1/32 to 1/16

also formed by chemical, frost, and salt weathering processes. All of these processes cause the particles to cleave on either prominent edges or natural grain fracture plains, and result in the small and spherical shape of silt grains.

On the Krumbein phi scale, the phi value for grain size is given by

$$\Phi = -\log 2D$$

where D is the particle size in mm.

For silt, the value of Φ varies from 2 to 8 or 0.0039 to 0.0625 mm.

Cross-References

- ▶ Aeolian Processes
- ▶ Aggregate
- ▶ Aggregate Tests
- ▶ Alkali-Silica Reaction
- ▶ Aquifer
- ▶ Beach Replenishment
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Coastal Environments
- ▶ Current Action
- ▶ Desert Environments
- ▶ Engineering Properties
- ▶ Erosion
- ▶ Floods
- ▶ Fluvial Environments
- ▶ Glacier Environments
- ▶ Groundwater
- ▶ Hydrocompaction
- ▶ Hydrogeology
- ▶ Igneous Rocks
- ▶ Infiltration
- ▶ Landforms
- ▶ Loess
- ▶ Noncohesive Soils
- ▶ Quicksand
- ▶ Sedimentary Rocks
- ▶ Sediments
- ▶ Soil Field Tests
- ▶ Voids

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Sinkholes

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Synonyms

Dolines

Definition

Sinkholes are among the most obvious karst phenomena. They are closed depressions in the ground surface resulting from downward movement into karst cavities of exposed soluble rocks; of soluble rocks together with covering soils, rocks, and particles; or of covering material only without dissolution. This movement can be the result of dissolution, erosion, and various hydraulic failures, as well as by gravitational failure of rocks and soils. Sinkholes may be full of water (Fig. 1). Old sinkholes that are filled with, and often covered by, younger sediments are termed buried sinkholes.

Sometimes the term sinkhole is interpreted more widely to include features that result from the above-mentioned processes but which are not karst phenomena. From this perspective, sinkholes form above cavities having natural or even anthropogenic origins. This definition has been criticized by Beck (1984), who proposed referring to all closed non-karstic depressions as sinkhole-like features or subsidence pits. More rarely, these depressions are referred to as pseudosinkholes (Mendonça et al. 1993).

Genetic Types

Based on global (Waltham et al. 2005), European (Gutiérres et al. 2008), and Russian (Khomenko et al. 2013) data, the following classification of sinkholes can be proposed based on the mechanism of sinkhole formation (Fig. 2):

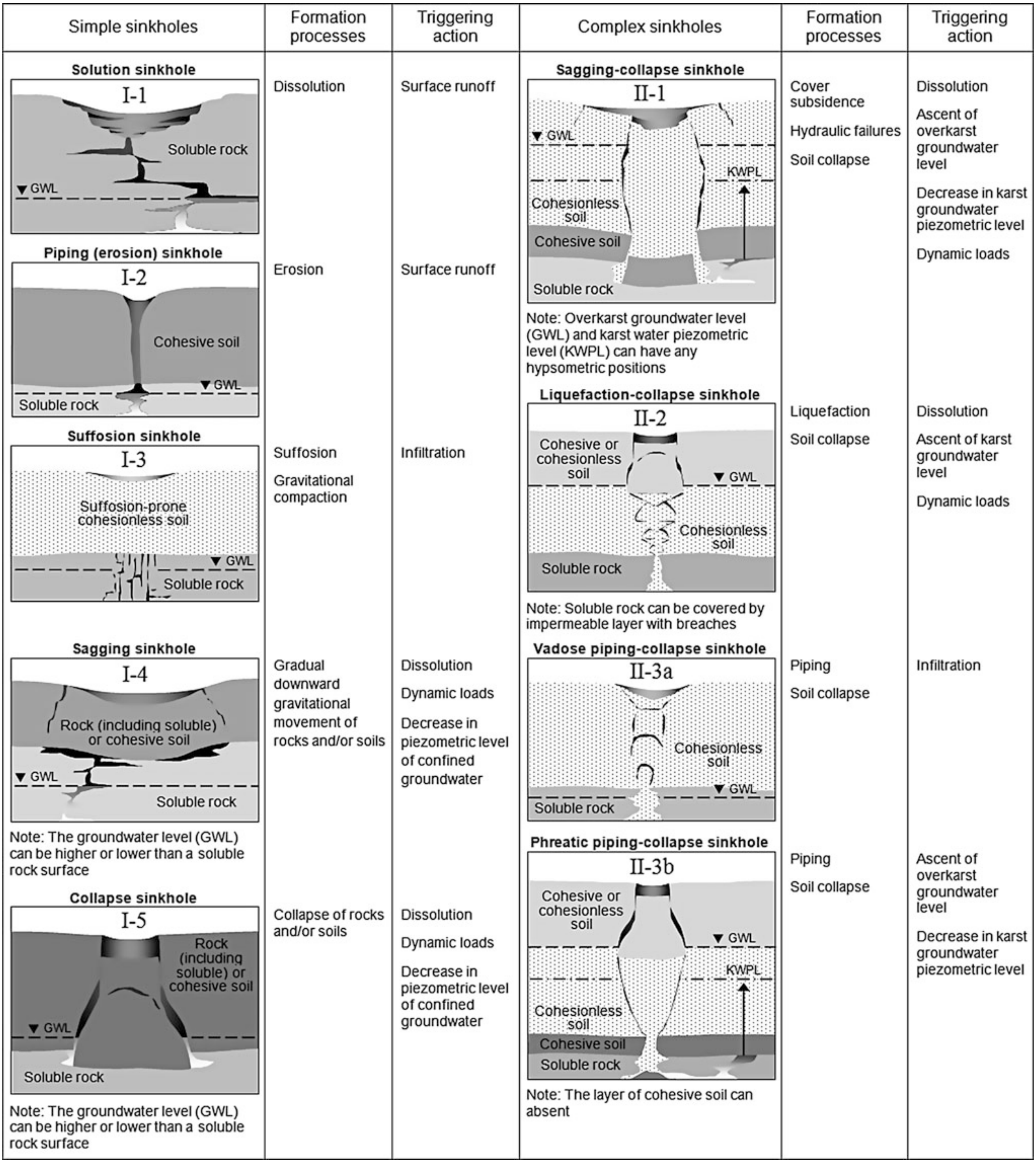
Sinkholes, Fig. 1 Sinkhole formed in 2012 in Belozerye village, Nizhny Novgorod region, Russia

I. Simple sinkholes
   1. Solution sinkholes
   2. Piping (erosion) sinkholes
   3. Suffosion sinkholes
   4. Sagging sinkholes
   5. Collapse sinkholes
II. Complex sinkholes
   1. Sagging-collapse sinkholes
   2. Liquefaction-collapse sinkholes
   3. Piping-collapse sinkholes
      (a) Vadose piping-collapse sinkholes
      (b) Phreatic piping-collapse sinkholes

Simple sinkholes occur due to failure of one dominant type of soil or rock. Both solution and piping (erosion) sinkholes result from surface run-off, have mainly cone-like forms, and develop around swallow holes, but the two differ in one important way: solution sinkholes appear in the open surface of soluble rock as a consequence of its dissolution, whereas piping (erosion) sinkholes occur due to erosion of the walls of a vertical cavity in cohesive soil covering soluble rock. The cavity can have various origins and configurations, but it requires a passage from the ground surface to a cavity in soluble rock because this makes down-washing of particles from the surface possible. Suffosion sinkholes can form when fractured soluble rocks are covered by cohesionless soils and water seepage transports soil particles downward into fractures and between more coarse particles. A saucer-shaped depression can appear on the ground surface as a result of gravitational compaction of this loosened cover. Sagging sinkholes are generated by gradual downward gravitational movement of rocks and/or soils into a cave in soluble rock. The usual saucer-shaped depressions are sometimes edged by tension cracks. Collapse sinkholes have the same origin but form via a different mechanism: the movement is a sudden collapse of a cave's roof. Unlike sagging sinkholes, collapse sinkholes are always edged by scarps, and their initial forms and sizes change over time due to slope failures and surface runoff. The initial configuration of collapse sinkholes can be dome-like or cylindrical (Fig. 3a), but eventually they invariably have a conical shape (Fig. 3b).

Complex sinkholes occur in covered karst areas and are caused by several successive failures in cover soils leading to the final collapse. Sagging-collapse sinkholes form in areas where soluble rock underlies a layer of cohesive soil covered by saturated cohesionless soils. This type of sinkhole is the result of cover subsidence, hydraulic failures, and collapse. Initially, a sagging sinkhole appears in the ground surface, and then a bowl-shaped collapse sinkhole appears in its central part. Liquefaction-collapse sinkholes are specific phenomenon caused by discontinuous and cyclic hydraulic-gravitational failures of cohesionless soils covering soluble rocks. During this complex process, cavities in cohesionless soils appear and disappear, with each cavity appearing above a previous cavity that has disappeared. The process begins in the bottom of the cohesionless soil, above a cavity in soluble rock, and spreads up to the karst water piezometric level. Piping-collapse sinkholes are the result of hydraulic failure of unsaturated (vadose piping-collapse sinkholes) or saturated (phreatic piping-collapse sinkholes) cohesionless soils by downward water seepage and its movement into cavities in soluble rock. Liquefaction-collapse and piping-collapse sinkholes have approximately the same configurations as collapse sinkholes.

Sinkholes can be classified into groups based on their triggers. Types I-1, I-2, I-3, and II-3a are triggered by a natural or anthropogenic water input (rainfall, irrigation, etc.) to soils and rocks, particularly soluble rocks, from the ground surface. Types I-4, I-5, and II-2 form as a result of dissolution

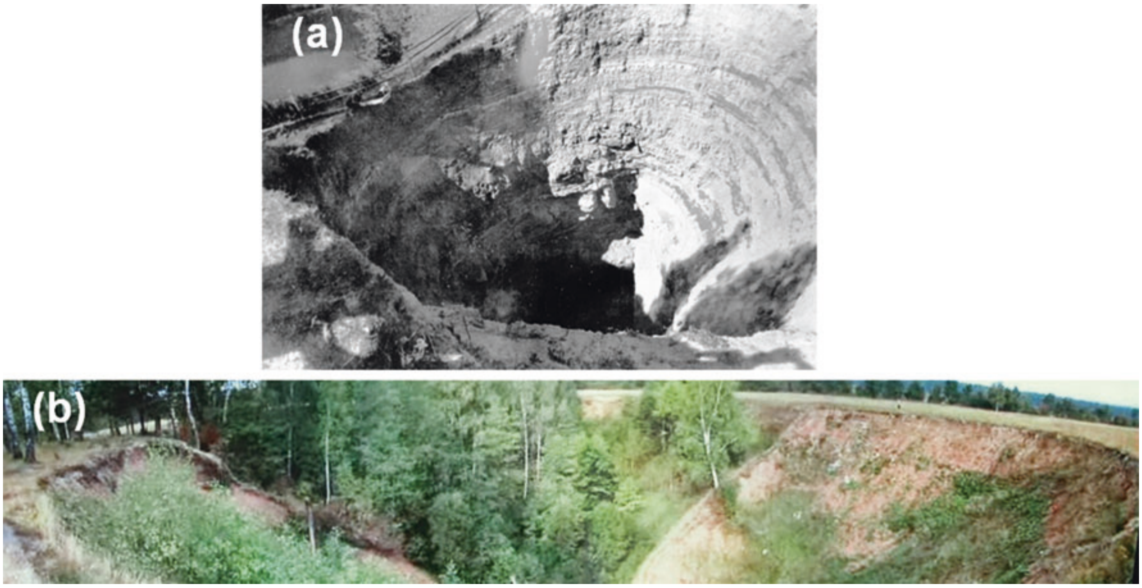


Sinkholes, Fig. 2 Nine main types of sinkholes with schematic cross sections

combined with dynamic loading (earthquakes, vibrations caused by human activity, etc.). Types II-1 and II-3b form only in the areas of covered karst, where the cover includes saturated cohesionless soil (mainly sands). Both of these types of sinkholes are complex and very sensitive to variations in karst and overkarst groundwater levels.

Dangers

Generally, sinkhole formation (especially sudden) under a building or structure is considered an emergency. Dangers include the possible destruction of buildings, threats to human life, and negative effects on the environment, which can all



Sinkholes, Fig. 3 Sinkhole formed in 1959 near Pivovarovo village, Vladimir region, Russia. Photographs taken in 1959 (**a**) and 2005 (**b**)

Sinkholes, Fig. 4 Industrial building in Dzerzhinsk, Russia, destroyed in 1992 by the sudden appearance of a large sinkhole

lead to social, financial, and environmental losses. The most catastrophic event of this kind occurred in 1962 in Transvaal, South Africa, when a sinkhole destroyed a crusher plant at a mine and 29 people perished (Waltham et al. 2005). Thirty years later, a similar catastrophe occurred in a metalworking plant in the Nizhny Novgorod region, Russia (Fig. 4), but fortunately, the sinkhole appeared at night so no workers were on site, and no one was harmed.

The relative dangers of sinkholes can be ranked based on factors related to their formation, including visibility of their development, invisibility of early formation processes, sensitivity to anthropogenic influences, and possibility of downward soil movement into fractures in soluble rocks. The danger ranking based on these factors is as follows: solution sinkholes → piping sinkholes → suffosion sinkholes → vadose piping-collapse sinkholes → sagging sinkholes → collapse sinkholes → liquefaction-collapse sinkholes → sagging-collapse sinkholes → phreatic piping-collapse sinkholes.

Buried sinkholes are also dangerous because of the possibility of suffusion or compaction of the fill (Waltham et al. 2005), as well as the possibility that the karst process may be reinitiated (Gutiérres et al. 2008), as this may result in the appearance of a depression under a building or structure situated above a buried sinkhole. Additionally, cohesive soil lying between soluble rock and cohesionless soil often has breaches due to buried sinkholes, which can contribute to the formation of liquefaction-collapse and phreatic piping-collapse sinkholes induced by anthropogenic causes.

Prediction

Currently, three factors are used to predict sinkhole formation: (1) assessment of the probability of sinkhole emergence within a given time interval; (2) assessment of where a sinkhole will form; and (3) assessment of the possible size of a future sinkhole. It is desirable and sometimes necessary to use a conceptual model of sinkhole formation in a particular area to successfully complete these assessments.

The first prediction factor can be addressed by processing chronological information about sinkhole emergence in a particular terrain to determine sinkhole frequency, that is, the number of sinkholes occurring during a unit of time in a unit area. This is the basic parameter for probabilistic assessment and can be considered constant for a particular

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region or as a function of different factors. For the latter, extrapolation and analogy can be applied to obtain a temporal prediction of sinkhole formation.

The second prediction factor can be addressed using three approaches. The first is based on the idea that the probability of a new sinkhole emerging at some time is higher if it is located near an existing sinkhole. This approach involves determining corresponding zonations of the investigated area using special mathematical processing of data about existing sinkholes. The second approach involves interpreting caves and buried sinkholes as "hypocenters" of sinkholes that will eventually appear on the ground surface. This involves using the results of borehole drilling, penetration tests, and geophysical investigations. However, unlike caves, buried sinkholes are difficult to identify (Khomenko 2010). The third approach involves searching for places with early signs of sinkhole formation, such as changes in soil cover and subtle deformations of the ground surface (Gutiérres et al. 2008). This approach can also include statistical processing of site-investigation data (Khomenko 2008).

The third prediction factor, sinkhole size, is usually assessed based on predicted sinkhole diameter because this parameter is important for foundation engineering in sinkhole-prone areas. A sinkhole's diameter can be interpreted as a stochastic or non-random value depending on the information needed. The former involves predicting a new sinkhole's diameter by statistical analysis of data about the diameters of existing sinkholes. The latter involves prediction based on simulation modelling of sinkhole formation (Sorochan et al. 1989) or on calculations from soil mechanics models (Khomenko and Leonenko 2015).

Mitigation

Some approaches and techniques are applicable to protect land, buildings, and structures from sinkhole formation (Tolmachev et al. 1986). These approaches may or may not affect geological processes (i.e., active or passive approaches) and can be carried out before or after the emergence of a sinkhole (i.e., preventively or operatively).

Planning is one protection measure. This includes optimal placement of buildings and structures, rational selection of their form, and regulation of urban density. The basis of this approach is (mainly prognostic) engineering geological mapping.

Structural engineering measures are very reliable but very expensive, especially if they are carried out operatively, for example by underpinning. This technique involves using special construction (mainly special foundations) that can guarantee the safety of a building if a sinkhole appears underneath it. Deep foundations can help, but only if the bearing stratum is hard rock that underlies any soluble rock with cavities. If friction piles are required, they must allow the possibility of free-fall from the pile cap. Tensile geogrids can help change a collapse sinkhole into a sagging sinkhole; this technique does not guarantee safety but reduces the danger considerably (Gutiérres et al. 2008).

Instrumentation control of sinkhole development may be very effective in terms of preventive protection. This involves monitoring: soils and rocks that are in contact with a structure, especially its foundation; groundwater; the ground surface; and tensions and deformations in constructive elements of building and structures. Geophysical techniques can be useful for this process. For railways and other important infrastructure, the control system can include warning signals.

Planning, special construction, and control are passive methods to protect against sinkhole formation. Active methods include: improving cover soils (mainly grouting); diversion of surface runoff, liquidation of swallow holes, remediation of sinkholes, and creation of watertight screens on and under the ground surface; prevention of downward groundwater seepage; drainage; and filling of fractures and cavities. Of these, the final three measures must be used circumspectly, and only after specialist hydrogeological investigation and analysis.

Summary

The term sinkhole generally refers to depressions in the ground surface, which are widespread in karst areas. Sinkholes can be classified into nine types according to the mechanism of their formation. The sudden occurrence of a sinkhole under a building or structure can have catastrophic consequences. Currently, methods are available to predict the emergence of a sinkhole within a certain time interval in a limited area and to determine the diameter of a possible sinkhole based on random or nonrandom parameters. Various mitigation measures can be used to protect land, buildings, and structures from sinkhole formation. Most of these involve civil engineering practices.

Cross-References

▶ Biological Weathering
▶ Chemical Weathering
▶ Dissolution
▶ Engineering Geological Maps
▶ Geohazards
▶ Hazard Assessment
▶ Karst
▶ Physical Weathering
▶ Piezometer
▶ Subsidence

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Site Investigation

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Definition

Site investigation is a core activity of an engineering geologist. A site investigation involves gathering site-specific geologic conditions related to the location, design, construction, and maintenance of engineered structures and the remediation of site conditions present from existing or previous land use. A site investigation will also determine how site-specific conditions are influenced by ongoing geologic processes (USBR 1998; Johnson and DeGraff 1985).

Introduction

An engineering geologist must ensure that the site investigation develops sufficient data for analyses to produce needed interpretations and recommendations. This requires the engineering geologist to be cognizant of any professional or regulatory guidelines applicable to either the particular site or the proposed project under investigation. These guidelines commonly establish a minimum set of requirements for conducting and documenting a site investigation and are applicable to a wide variety of projects (Gonzalez de Vallejo and Ferrer 2011; AGS 2011; CGS 2013a).

In most instances, an engineering geologist is conducting the site investigation as a member of an inter or multidisciplinary team for a specific project. These projects can be as diverse as constructing a highway tunnel, a natural gas pipeline, or an airport building to remediation of a contaminated industrial site or supporting disaster preparedness planning. Additionally, the area of a site investigation may be limited to a single building site involving a few hectares or less or it might be encompassing thousands of hectares such as in a metropolitan area. Consequently, it is the engineering geologist's responsibility to define the necessary elements of a site investigation consistent with the project needs and applicable guidelines.

Phases of a Site Investigation

There are multiple phases or stages in site investigation (Johnson and DeGraff 1985; Gonzalez de Vallejo and Ferrer 2011). Site investigations commonly have three phases carried out from the inception through completion of a project: (1) preliminary study, (2) site exploration (for predesign and detailed design), and (3) implementation (during construction). Following completion of a project, it can be valuable to initiate monitoring as a fourth phase. Monitoring can provide information on project performance in relation to the established geologic conditions. This can deliver valuable insights for improving engineering geology efforts during future projects or with similar geologic conditions.

These phases of site investigation will involve a variety of techniques and methods for obtaining needed geologic information. However, there should always be a map or maps prepared as part of this effort (WSGLB 2008; CGS 2013b). Maps may be the principal output for some types of projects

(USBR 1998). At a minimum, there should be a map showing the location and boundaries of the site. When there is sampling or other location-specific data collection at the site, a map should represent the sample points, transects, and observation locations for pertinent data. Where subsurface conditions are important to the project, a cross-section or similar representation of the subsurface should be made. Cross-sections can be developed by interpreting subsurface conditions encountered between borings or test pits. It may also result from mapping of exposed vertical cuts in trenches and other excavations (USBR 1998). It is important to have the alignment of any cross-section accurately placed on one or more of the maps.

During the first phase of site investigation, the focus is on how the site-specific geologic conditions will influence the proposed project. The other focus is on how the project might impact the site. This means the preliminary study should be designed to address the feasibility of the project and provide information needed for environmental documents and permits required by governmental entities. Project feasibility will necessitate applying the geologic information to the question of site or design suitability. Consequently, the preliminary study may be collecting geologic information at a primary location and alternative ones. It might also require determining the effect of geologic conditions on more than one proposed design (Johnson and DeGraff 1985). Another way preliminary studies contribute to feasibility is by generating the geologic information necessary to make project cost estimates. This might be information on the volumes of material required to be removed or imported, whether dewatering during construction will be required, or that rock will need to be blasted rather than ripped. The same geologic information needed for assessing project feasibility should in most cases be sufficient to also satisfy environmental document and permit requirements.

An important part of conducting a preliminary site investigation is to identify the key factors about the project site(s) and design(s) that will require evaluation in terms of geologic information. This initial step prevents the preliminary study from collecting more data or data in greater detail than is appropriate at this point in the project (Johnson and DeGraff 1985). Having identified critical data needs, the engineering geologist can then gather existing geologic reports and maps and extract other required information that is currently available. Existing remote sensing data such as aerial photography or multispectral imagery should also be obtained (Johnson and DeGraff 1985).

A preliminary field examination of the site is an important part of establishing what critical information still requires further study. With an understanding of the project and site, the field investigation can proceed to verify expected geologic conditions and secure through mapping, geophysical methods, test pits, borings, and sampling any missing data or data requiring more detail than were available from existing sources (Gonzalez de Vallejo and Ferrer 2011). Costs associated with conducting the field part of the preliminary assessment provide insight on likely costs for the next phase of site investigation.

Assuming the project moves forward, the next phase of site investigation is site exploration (Johnson and DeGraff 1985). Site exploration endeavors to identify the geologic information relevant to making sure the design will function as intended and the Earth materials present can be handled or used to successfully implement the design. The initial work during a site exploration gathers additional data for defining the final design. It is an effort to delineate the geologic and site conditions sufficiently to identify the final design elements. Later, more detailed testing essentially confirms the defined character of the geologic conditions and confirms the appropriateness of the final design. Field and laboratory testing is carried out on a finer scale during site exploration. Consequently, a greater number of test results are available for analysis. Site exploration emphasizes identifying any unknown geologic conditions that might cause a designed project to malfunction once built (Gonzalez de Vallejo and Ferrer 2011). After this phase of site investigation, adjusting the project design to any unexpected geologic conditions important to the project could be costly.

The final phase of site investigation is implementation when the actual project is constructed. It is important for some engineering geology studies to continue. Such studies can identify and verify conditions different than those expected (Johnson and DeGraff 1985). This possible circumstance is recognition that geologic conditions are complex and there is always some level of uncertainty. This is especially true for subsurface conditions. The large numbers of tests conducted throughout the preliminary study and site exploration only ensure an acceptable level of uncertainty about geologic conditions remains. Implementation studies offer the chance to identify anomalous situations requiring some modification in order to finish the project. These situations are generally referred to as a changed condition. In contracted work, a changed condition can result in more time and more or different materials being needed. Inevitably, this will change the cost of the contract. The data generated during implementation studies can enable the engineering geologist to assist in determining an equitable cost adjustment. The same data can also serve to improve the design of similar projects or projects undertaken in areas with similar geologic conditions. This reduces the uncertainty present when undertaking these future projects.

Communicating Site Investigation Findings

Site investigation is more than collecting data and presenting these on maps, cross-sections, graphs, and tables. It is really a scientific study carried out to support the completion of some effort to improve the human environment (Johnson and DeGraff 1985). It may be as straightforward as ensuring that a constructed building will not be destroyed by a dormant landslide or as complex as identifying the relative risk of landslide activity over time across a large area to use in guiding land use determinations in the future. The preliminary studies for any project can be likened to gathering data to formulate a hypothesis; how the geologic conditions present will potentially affect the proposed project and how the project will alter the natural environment. The site exploration is an effort to collect data to prove or disprove the initial conclusions formed under the preliminary assessment. Implementation studies, and monitoring studies should these be undertaken, are the means to confirm the final conclusions.

Throughout the site investigation phases, the collected data are being analyzed, compared, and classified according to established scientific principles. The analysis results are then interpreted to enable design of the project to fit the actual conditions present. The interpretations produce conclusions and recommendations used by other professionals involved with the project. This enables them to better carry out their technical responsibilities whether it is the engineer specifying the materials to be used or the construction manager who is preparing the site plans. For this part to happen, the site investigation results including data, analyses, interpretations, and recommendations must be effectively communicated to fellow participants in the project (Johnson and DeGraff 1985; USBR 1998). The attention given to producing consistent and clear reports in engineering geology is recognition of how important communication is to site investigation (WSGLB 2008; CGS 2013b; OSBGE 2014). Reports do not simply represent documentation of the site investigation. Reports are a vital means of communication.

Summary

Site investigations are the primary means by which an engineering geologist develops data for analysis to support interpretations and recommendations. The data are generally collected in three phases during the project as the engineering geologist functions as a member of a multi or interdisciplinary team member. The variety of techniques and methods used to obtain data for a particular project are dependent on the needs of the project including geologic information required for project design, construction management, and environmental assessment. Output from site investigation is commonly documented in reports or annotated maps. The interpretations and recommendations produced by analysis of the data must be both scientifically sound and effectively communicated.

Cross-References

▶ Aerial Photography
▶ Cross Sections
▶ Designing Site Investigations
▶ Engineering Geology
▶ Environmental Assessment
▶ Field Testing
▶ Land Use
▶ Monitoring
▶ Pipes/Pipelines
▶ Subsurface Exploration

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Slurry Trench

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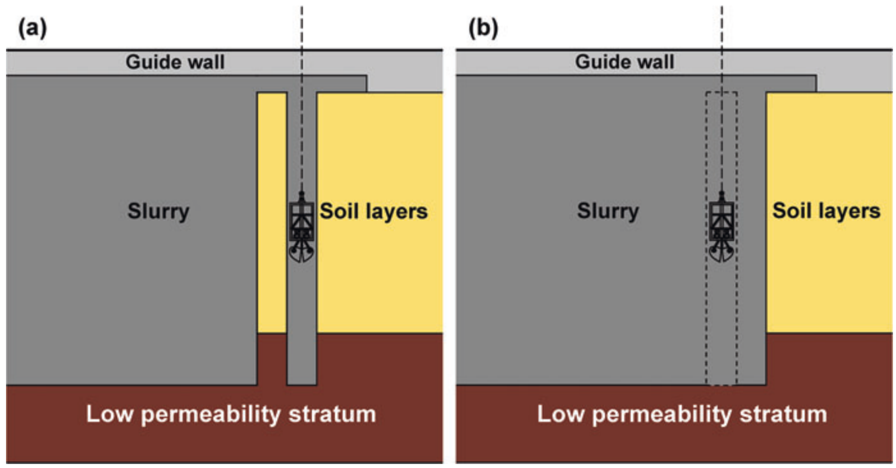
Synonyms

Cut-off wall

Definition

A slurry trench is a long narrow vertical excavation, progressively filled with slurry as it is excavated, typically used in the construction of diaphragm walls in civil engineering or cutoff walls (vertical barriers) for the inhibition of groundwater flow or control of contaminant transport (Paul and Davidson 1992).



Slurry Trench, Fig. 1 Typical procedure of excavation of slurry trenches by a clamshell: (**a**) the first panel is excavated with slurry filling the excavated space simultaneously; (**b**) the second panel is excavated to form a continuous slurry trench

Construction

Slurry trench construction is often started with a set of guide walls, which are constructed on the ground surface to outline the desired slurry trench and guide the excavation machinery. The slurry, generally a thick colloidal mixture of bentonite (an absorbent aluminum phyllosilicate clay consisting mostly of montmorillonite) and water, typically consisting of 5% solid and 95% water by weight, is mixed for hydration before emplacement. Trenches shallower than 25 m may be excavated using a backhoe, whereas deeper trenches are usually excavated by a clamshell or dragline. The typical procedure for excavation of slurry trenches by a clamshell is illustrated in Fig. 1. With either method, to prevent trench collapse during excavation, the trench is always fully filled with slurry so that it exerts pressure on the trench walls to balance the Earth pressure and hydraulic pressure from the surrounding soils. The slurry also penetrates into the surrounding soil and forms a low-permeability “filter cake” on the trench side wall that contributes to the low permeability of the completed walls. Once a suitable length of slurry trench is excavated, backfill materials, such as reinforced concrete, plastic concrete, soil-bentonite, and cement-bentonite, are placed into the trench to form diaphragm walls or cut-off walls by displacing the less-dense slurry. Diaphragm walls are often used as a structural system in civil engineering either for temporary excavation support or as part of permanent structure. Cut-off walls are commonly used to control groundwater flow or contain subsurface contamination. The excess displaced slurry is usually pumped out, filtered, and stored in tanks for reuse in a new trench or recycled. For cut-off walls, in most cases the excavated slurry trench is extended 60–90 cm into a low-permeability strata such as clay or bedrock to ensure minimal leakage under the final wall; although in some cases “hanging” slurry trenches, which penetrate the groundwater table, are used to stop the movement of floating contaminants or gases. Slurries fabricated with biodegradable polymers, instead of bentonite, may be used in slurry trenches for specific applications such as drainage trenches and permeable reactive barriers.

Cross-References

- ▶ Cohesive Soils
- ▶ Collapsible Soils
- ▶ Excavation
- ▶ Floods
- ▶ Fluid Withdrawal
- ▶ Groundwater
- ▶ Hydraulic Action
- ▶ Infiltration
- ▶ Lateral Pressure
- ▶ Normal stress
- ▶ Percolation
- ▶ Retaining Structures
- ▶ Water

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Soil Field Tests

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Definition

Soil field tests are visual-manual methods for field examination of soil. The field tests provide the means for a consistent description of soil including developing information for a preliminary identification of soil type under the Unified Soil Classification System (USCS) during field mapping and site investigation (Johnson and DeGraff 1988; USBR 1998).

Introduction

Societies depend on soil as a material for use in building structures and infrastructure and providing the foundation to support them. Consequently, studies of potential building sites and areas being considered for development should include mapping the soil present at and near the ground surface (Johnson and DeGraff 1988; USBR 1998). The exploratory mapping would include areal extent of soils at the ground surface and those present in shallow exposures made by test pits, trenches, and hand-dug auger holes (Keaton and DeGraff 1996; USBR 1998). Documenting the character of the soil being mapped requires using a reliable approach to describing the soils present including the likely soil type based on the Unified Soil Classification System (USCS). Such documentation is needed for clear communication of study results and for comparison to soils found present at other locations.

Perhaps more importantly, the descriptors including the identified USCS soil type developed by soil field tests can be related to the expected engineering properties such as shear strength, compressibility, and permeability (USBR 1998). Determining which engineering properties are important depends on the intended utilization for the site or area and the design requirements of that intended use. Exploratory soil map results will show which soils are representative, which may be problematic, and which are situated at points critical to project development. Gonzalez de Vallejo and Ferrer (2011, their Fig. 7.4) illustrate project-level soil mapping results supporting a geotechnical classification for an area on the island of Tenerife in Spain's Canary Islands.

Soil field tests enable soils to be an element in producing preliminary engineering geological maps (see ► Engineering Geological Maps). Soils are an important parameter in the development of an engineering project whether it involves predicting shallow landslide activity across a broad area (Gioia et al. 2016), identifying alignments suitable for pipelines or irrigation (USBR 1998), locating aggregate resource sites (Doornkamp et al. 1979), or mapping geo-mechanical characteristics and geological factors affecting land use suitability or construction activities (Gonzalez de Vallejo and Ferrer 2011; their Figs. 7.1 and 7.2). Johnson et al. (2016) used soil and geologic data in mapping to effectively evaluate slope instability in a quarry where excavation to significant depths was well below the local groundwater table. Pit slope stability would commonly be a straightforward function of slope height, angle, and the strength characteristics of the coarse, granular material present. Their mapping determined by the geologic conditions influencing pit slope stability was more complex (Johnson et al. 2016). Consequently, the likelihood that the measures put in place to maintain slope stability will be successful is high. These examples represent how exploratory soil mapping can be combined with other field data such as geomorphic and bedrock mapping to produce engineering geological maps.

The results of exploratory soil mapping serve as a guide for collecting samples for laboratory analysis. Laboratory analysis soil samples provide the detailed information suitable for making decisions about project feasibility and appropriate engineering design criteria (see ► Characterization of Soils). Whereas laboratory results for various soil tests provide more definitive data than equivalent soil field tests, soil field tests are significantly less expensive to perform. So simply sampling the site's soils on a grid or similar systematic layout for laboratory testing may generate too many samples to be cost effective. Exploratory soil mapping ensures the samples collected for laboratory sampling are representative of the soils found at the project site and those soils underlying locations important to project success. For example, field mapping of soils and related data at an airport site in Dubai, UAE, supported a need for 18 test pits and 19 boreholes to establish sufficient engineering data for a foundation study whereas another site with similar soils, but no soil mapping, utilized a total of 86 test pits and 30 boreholes (Doornkamp et al. 1979). Although the number of laboratory soil samples collected will likely be less than the number collected by a systematic layout, there will be greater confidence in the analysis results being an accurate representation of the engineering qualities of those soils present.

Soil field tests are less precise than laboratory tests for determining engineering properties, but they better represent the *in-situ* soil character. Soil field tests can be collected at targeted points and yield data more quickly than laboratory testing. Soil field tests enable soils results to be linked to physical features in the mapping area. Albeit the USCS classification of a soil does not represent or imply the soil's origin, the geomorphic character of where the soil is present can have engineering significance (see ► Engineering Geological Maps). Doornkamp et al. (1979) use several engineering projects to demonstrate how geomorphological maps, drawing heavily upon soils data, can be produced rapidly and generate information important to the engineering design or suitable project location.

Field Tests for Classifying Soils

Soil is essentially a mass of solid particles with intervening spaces containing air or water (see ► Soil Properties). The physical and mineralogical character of these solid particles will vary depending on their origin. Consequently, a soil mass can have different capabilities and suitabilities for engineered purposes (see ► Characterization of Soils). Recognition that certain index properties are linked to how well a soil performs for an engineered purpose enabled development of soil classifications for discerning how best to design and utilize soil (see ► Classification of Soils). The Unified Soil Classification System (USCS) is widely used to classify soils based on significant index properties assessed by laboratory testing procedures (USBR 1998).

Soil field tests provide a visual-manual means for making an initial classification of a soil according to the USCS system. Important soils classified during mapping in the field would require laboratory analysis to confirm or refine the field classification. These would include soils that represent the more common ones encountered and those present at locations which may be important to the design and function of the future engineered use.

The field classification under the USCS scheme begins using particle size. Determining the predominance of either coarse-grained or fine-grained particles is the first step. The coarse fraction would be the part of the soil sample in which the individual grains are visible to the naked eye. Table 1 describes common items which facilitate recognizing particle sizes in the coarse fraction of a soil sample.

The determination of having a coarse-grained or fine-grained soil involves whether the soil sample has more than 50% coarse-grained particles making it a coarse-grained soil or more than 50% fine-grained particles making it a fine-grained soil. This percentage is by dry weight (USBR 1998; NSW 2017). This will necessarily be a visual estimation of a collected sample spread on a flat surface or as seen in a vertical exposure for *in-situ* samples. The percentage determination does not include particles larger than 75 mm, that is, cobbles and boulders (USBR 1998). The sample documentation should include the percentage of cobbles and boulders by volume. This is not an easy estimation to make and accuracy comes with experience (USBR 1998).

Following much of the same procedure as described above, another estimation approach would be to separate out the coarse-grained particles from the fine-grained ones in a representative sample. The separation could be done entirely by hand or with the aid of a No. 40 sieve. The two soil fractions could then be placed in identical soil bags and secured to either end of a staff or short pole. The exact midpoint of the pole would be placed on a fulcrum (balance) point. Like a balance scale, the heavier bag would tip the end of the pole down clearly showing which bag (fraction) had the greater weight.

Soil Field Tests, Table 1 A guide to visual estimation of soil particle sizes[a]

Particle descriptor	Size	Examples of familiar items within the size range
Boulder	300 mm or more	Larger than a volleyball
Cobble	300 mm to 75 mm	Volleyball–grapefruit–orange
Coarse gravel	75 mm to 20 mm	Orange–grape
Fine gravel	20 mm to no. 4 sieve (5 mm)	Grape–pea
Coarse sand	No. 4 sieve to no. 10 sieve	Sidewalk salt; rock salt
Medium sand	No. 10 sieve to no. 40 sieve	Openings in window screen
Fine sand	No. 40 sieve to no. 200 sieve	Sugar–table salt (grains that are barely distinguishable)

[a]From USBR (1998)

Identifying Fine-Grained Soils

If a soil sample is determined to be predominantly fine-grained, the soil classification will be based on the silt and clay in the fine-grained fraction. The first question for further classification will be whether these silt and clay have a high or low liquid limit (Table 2). Determining the liquid limit is not easy in a field setting. Start the manual test by selecting about a tablespoon (about 15 g) soil containing only particles which would pass through a No. 40 sieve (NRCS 2014). The soil should be air-dry. Place the sample in the palm of your hand and slowly add a little water. Observe the speed of penetration of the water into the sample. This can be done by carefully lifting the wetted surface with a knife blade or other thin instrument. Soils with a high liquid limit (LL) will not be penetrated by the added water as quickly as low liquid limit (LL) soils. Continue to slowly add water to the sample occasionally kneading the sample to mix the soil and water thoroughly. Continue this process until the sample in your palm has a putty-like consistency. Take careful note of how much water was needed to achieve this consistency. The amount of water added is a measure of the liquid limit with low liquid limit soils requiring less water than high liquid limit soils. This test is very dependent on the experience of the sampler with this test. The sampler may benefit from doing this test on samples with known high and low liquid limits prior to conducting field tests.

Alternatively, you can perform a cube test (NRCS 2014). Take about a tablespoon of the fine-grained soil fraction into your hand. Slowly add water to the soil and knead the soil until it reaches a plastic, putty-like consistency. The kneading should be thorough enough to ensure no dry particles or lumps remain. Form the soil patty into a cube. Then flood the surface of the cube with water before immediately breaking it open. If water had penetrated inside the cube, this is indicative of a low liquid limit. Take care to ensure you are observing water that has penetrated the cube rather than having flowed in when the cube was broken. If no water had penetrated inside the cube, it is indicative of a high liquid limit.

Soil Field Tests, Table 2 Flow chart for identifying the USCS classification for a fine-grained soi1[a]

Fine–grained soils: More than half of material (by weight) is of individual grains not visible to the naked eye, e.g. coarse fraction of soil		Dilatancy Reaction	Toughness	Plastic Limit Evaluation	Shine Test	Dry Strength	Odor	
	Silt and Clay (Low Liquid Limit)	Rapid	Low to None	None	Dull	Slight		**ML**
		Medium to Slow	Medium	Weak	Slight to Shiny	Medium to High		**CL**
		Slow to None	Low (Spongy)	None	Dull to Slight	Medium	Pronounced Organic	**OL**
	Silt and Clay (High Liquid Limit)	Very Slow To None	Medium to High	Weak to Strong	Slight	Medium		**MH**
		None	High	Strong	Shiny	Very High		**CH**
		None	Low to Medium (Spongy)	Weak	Dull to Slight	High	Pronounced Organic	**OH**

[a]Compiled and modified from USBR (1998), NRCS (2014) and NSW (2017)

The identification of whether a sample belongs among the low liquid limit or high liquid limit silt and clay soils in Table 2 may be clearer once the other field tests are performed. But this is not necessarily definitive for grouping by high or low liquid limit (USBR 1998). As Table 2 shows the properties of elastic silt (MH), a high liquid limit soil, are similar to those for lean clay (CL), a low liquid limit soil. While having the sample particles dry quickly on the hand and have a smooth, silky feel when dry is common to silts, a definitive identification of elastic silt (MH) from lean clay (CL) may require laboratory confirmation.

The additional soil field tests used on fine-grained soils are: (1) dilatancy reaction, (2) toughness, (3) plastic limit evaluation, (4) shine (cut surface), and (5) dry strength (USBR 1998; NRCS 2014). A sixth test, odor, is important only for identifying low liquid limit OL soils, high liquid limit OH soils, and highly organic soils (PT).

Dilatancy reaction – Use the same soil sample previously used to determine liquid limit or select about a tablespoon (about 15 g) soil containing only particles which would pass through a No. 40 sieve (USBR 1998; NRCS 2014). Mold the soil material, with additional water added as needed, to form a pat in the palm of your hand. It should be soft but not sticky, in consistency. Use your other hand to sharply strike the palm (horizontally) holding the soil pat. Note the appearance of any water on the surface of the soil pat giving it a shiny or "livery" look. Once the soil takes on this appearance, there will be no change from additional strikes. For low plasticity soils, this change would be seen after 2–4 strikes (NRCS 2014). For high plasticity soils, no change would be seen even after 10 strikes. Once the surface appearance takes on this change, gently squeeze the soil pat by partly closing your hand. Take note of how fast or slow the water disappears changing the soil pat surface appearance to its initial look. There may be no change upon squeezing for some soils. Table 3 shows how the response to shaking and squeezing defines dilatancy reaction or its absence. The dilatancy column in Table 2 indicates which response is associated with different soils classified under the USCS system.

There are several things to consider to avoid getting an incorrect dilatancy reaction result (NRCS 2014). First, the soil pat at the start of the test should not be too wet. Second, be

Soil Field Tests, Table 3 Describing dilatancy reaction[a]

Descriptor	Criteria
None	No visible change in the specimen
Slow	Water slowly appears on the surface of the specimen during shaking and does not disappear or disappears slowly upon squeezing
Rapid	Water quickly appears on the surface of the specimen during shaking and disappears upon squeezing

[a]From USBR (1998)

cautious with soil samples having significant mica flakes. Their shiny appearance could be mistaken for the appearance of water. Third, failure to carefully separate the initial soil sample into coarse and fine fractions could result in the dilatancy test sample having fine-grained material with a significant amount of sand grains. The sand is likely to cause an error by accelerating the reaction.

Toughness and plastic limit evaluation – The first step is to bring the soil sample to near its plastic limit. Start with the same soil sample used earlier for the dilatancy reaction test (USBR 1998; NRCS 2014). Knead the soil pat to aid in drying the soil. It may be useful to also add small amounts of additional soil that passed through a No. 40 sieve (NRCS 2014). Be sure to fully incorporate any added soil into the soil pat. Periodically, as the soil dries, roll the soil out on a flat surface to form a 1/8 inch (3 mm) diameter thread. If the thread begins to crack or crumble, the plastic limit water content of the soil has been reached. Until this point is reached, continue to knead the soil and re-try rolling the 1/8 inch (3 mm) diameter thread to determine if the soil is now at the plastic limit. Once the plastic limit water content is reached, take note of the pressure required to roll the thread at that point. Take the pieces of the thread and lump them together. Then attempt to re-roll the thread and take note of your success in relation to the descriptions in Table 4. Again make a lump from the thread pieces and knead the lump until it begins to crumble.

Your observations from creating, first, a thread and, then, a lump enable you to determine the soil's plasticity. Just match the descriptor in Table 4 to the observations made during testing. Determine the stiffness of the thread and the lump at the point when the soil was at the plastic limit. The stiffness and the pressure determine the toughness. Compare your observations to the criteria in Table 5 to identify the appropriate descriptor for the soil toughness. If there is some uncertainty in your observations, the soil can be carefully re-wetted to enable a second testing of the soil sample. Like the dilatancy reaction test, a significant amount of sand in the soil will dramatically alter the response to these tests (NRCS 2014). Care in preparing the soil sample used in testing is important for ensuring confidence in the results obtained. The toughness and plastic limit evaluation columns in Table 2 indicates which response is associated with different soils classified under the USCS system.

Soil Field Tests, Table 4 Plastic limit evaluation associated with toughness test[a]

Descriptor	Criteria
Nonplastic	A 1/8 (3 mm) inch diameter thread cannot be rolled at any water content
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit
Weak	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit
Strong	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit

[a]Modified from USBR (1998)

Soil Field Tests, Table 5 Describing toughness[a]

Descriptor	Criteria
Low	Only slight pressure is required to roll the 1/8 (3 mm) inch diameter thread near the plastic limit. The thread and the lump are weak and soft
Medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium stiffness
High	Considerable pressure is required to roll the thread to near the plastic limit. The thread and the lump have very high stiffness

[a]From USBR (1998)

Shine test – A third test can be performed using the same soil pat employed in the dilatancy reaction and toughness testing (NRCS 2014). This increases the likelihood of performing the test at water contents near the plastic limit, thus avoiding mistaking the appearance of free water for the shininess produced by the clay content in the soil pat. For the shine test, place the soil pat on a flat surface. Create a smooth surface by either cutting the pat with a knife or stroking the pat surface with a smooth object like a finger nail (NRCS 2014). Observe the appearance of the smooth surface created in direct sunlight. The appearance can be described as:

- Shiny
- Slight to shiny
- Dull to slight
- Dull
- None

A shiny appearance is typical for soils with high plasticity; a dull appearance is common for soils with low plasticity (NRCS 2014). The shine column in Table 2 indicates which response is associated with different soils classified under the USCS system.

Dry strength – Make your sample for determining dry strength from soil containing only particles which would pass through a No. 40 sieve (USBR 1998; NRCS 2014). Mold or knead the soil until it has a putty-like consistency. Water can be carefully added during this preparation step. From this sample, mold a ball or cube ½ inch (12 mm) in diameter. The USBR (1998) recommends preparing three balls of this size from your sample. Whether a single ball or cube or three balls are prepared, allow these test specimens to dry (USBR 1998; NRCS 2014). This may be only a few hours in direct sunlight. Air-drying may necessitate waiting overnight. Once the sample balls or cubes are dried, dry strength is determined by breaking them using finger-thumb pressure. The reaction of the balls or cubes to this pressure as described in Table 6 will identify the appropriate description of the soil's dry strength. The dry strength column in Table 2 indicates which response is associated with different soils classified under the USCS system.

High dry strength is typically associated with high plasticity soils. Low dry strength is commonly found with low plasticity soils. The presence of high-strength water-soluble cementing materials, such as calcium carbonate, may result in very high dry soil strengths (USBR 1998; NRCS 2014). Any calcium carbonate can be detected by applying a few drops of dilute hydrochloric acid (HCL) to see if a reaction (fizzing or bubbling) occurs.

If the soil present in the field is dry, it may seem a good idea to simply use clods for testing dry strength rather than preparing balls or cubes. Be aware that natural clods have a lower dry strength than prepared test ball or cubes (NRCS 2014). Dry strength results obtained from natural clods may also differ due to the presence of coarse sand (USBR 1998) or the presence of natural cementing agents such as calcium carbonate (NRCS 2014).

Odor – Once dilatancy reaction, toughness, plastic limit evaluation, shine, and dry strength are determined for a soil sample, there is one test remaining for classifying a fine-grained soil (Table 2). Odor is the means for distinguishing an organic fine-grained soil from an inorganic fine-grained soil. Although it is only one of several diagnostic criteria for highly organic soils (PT), odor is an important characteristic.

An organic odor is one which is expected in marshy or swamp due to decaying vegetation (USBR 1998). An organic soils is likely to be detectable by odor if it is warm and moist (NRCS 2014). So wetting and warming a dried sample of a possibly organic soil can revive its odor if present (USBR 1998). Organic particles may be visible upon close scrutiny in some organic soil samples. Not surprisingly, organic soils not only have an odor but will be dark brown to black in color (USBR 1998). It is not unusual for excavated organic soils to change color (black to brown) due to exposure to air. The importance of the odor test has more to do with recognizing organic silts and clays (OL and OH) than with identifying highly organic soils (PT). OL and OH soils can be expected to include enough organic particles to influence their soil properties (USBR 1998).

Soil Field Tests, Table 6 Describing dry strength[a]

Descriptor	Criteria
None	The dry specimen crumbles with mere pressure of handling
Low	The dry specimen crumbles with some finger pressure
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface
Very high	The dry specimen cannot be broken between thumb and a hard surface

[a]From USBR (1998)

Identifying Coarse-Grained Soils

If a soil sample is determined to be coarse-grained, the next step is determining whether the more than half of the coarse-grained particles are larger than ¼-inch (6.35 mm; Table 7). In other words, particles too large to pass through a No. 4 sieve (Table 1). Table 7 shows the USCS classification details for coarse-grained soils. Making the determination based on half being larger than ¼-inch (6.35 mm; No 4 sieve) identifies whether the sample falls with the sand and sandy soils subcategory or the gravel and gravelly soils subcategory (USBR 1998; NSW 2017). As when determining whether an entire sample is coarse or fine-grained, this determination is also based on weight.

Determining whether the coarse-grained soil is primarily gravel and gravelly soils or sand and sandy soils can be done without using the No. 4 sieve. A representative sample can be spread on a flat surface, then visually estimate the percentage being gravel and gravelly soil (NRCS 2014). The remaining percentage will be sand and sandy soil. Use the size guide provided in Table 1. Visually separating particles which are the size of peas up to those the size of oranges is relatively straightforward. Remember that it is not the volume of separated gravels and sand which determines whether the coarse-grained soil is predominantly gravel or predominantly sand. Rather, it is whichever weight of these two volumes represents more than half of the coarse-grained soil (NRCS 2014).

A gravel and gravelly soil or a sand and sandy soil is then further subdivided into being classified as clean or dirty (Table 7). Two methods are proposed for distinguishing clean from dirty (NRCS 2014). One method uses a wet palm as noted in the descriptions in Table 7. Place a sample of the soil in your palm and wet it with plain water. The sample is considered dirty if an obvious stain remains on your palm after brushing off the coarse-grained material. A powdery residue forming on your stained palm when it

Soil Field Tests, Table 7 Flow chart for identifying the USCS classification for a coarse-grained soil

Coarse –grained soils More than half of material (by weight) is of individual grains visible to the naked eye, e.g. coarse fraction of soil	**Gravel and gravelly soils** More than half of coarse fraction (by weight) is larger than 1/4 –inch (6.35 mm) size	**Clean gravels** Will not leave a dirt mark on a wet palm	Wide range in grain sizes (well-graded) with significant amounts of all intermediate grain sizes	**GW**
			Predominantly one grain size (poorly graded) or full range of coarse fraction sizes with obvious gaps	**GP**
		Dirty gravels Will leave a dirt mark on a wet palm	Contains some non-plastic fines or fines with low plasticity (same characteristics as ML)	**GM**
			Contains some plastic fines (identifiable by same characteristics as CL or CH)	**GC**
	Sand and sandy soils More than half of coarse fraction (by weight) is smaller than 1/4-inch size	**Clean sand** Will not leave a dirt mark on a wet palm	Wide range in grain sizes (well-graded) with significant amounts of all intermediate grain sizes	**SW**
			Predominantly one grain size (poorly graded) or full range of grain sizes with obvious gaps	**SP**
		Dirty sand Will leave a dirt mark on a wet palm	Contains some nonplastic fines or fines with low plasticity (same characteristics as ML)	**SM**
			Contains some plastic fines (identifiable by same characteristics as CL or CH)	**SC**

Note: For visual classification, the ¼-inch (6.35 mm) size may be used as equivalent to the No. 4 sieve size (USBR 1998)

dries is indicative of fines being present in the sample (NRCS 2014).

The alternative method uses a clear container, like a beaker, filled with clean water (NRCS 2014). Drop a representative sample of the soil you are testing into the beaker and observe if the water becomes cloudy with suspended particles. If an appreciable cloud remains suspended after 30 s, it represents silt and clay-sized particles. Observing these conditions is indicative of a dirty coarse-grained soil (NRCS 2014).

Dirty gravels or sand are both further subdivided on the basis of their fine-grained component being plastic or nonplastic (Table 7). In other words, is their fine-grained component more like MH and CH soil or more like an ML soil? The fines in your dirty gravel or sand will be nonplastic or low plasticity like an ML soil if a dilatancy reaction is rapid (Table 3), and a plastic limit evaluation (Table 4) yields a nonplastic result (NRCS 2014). Other results from both the dilatancy reaction and plastic limit tests would show the fine-grained component to be plastic and to some degree like MH and CH soils.

The amount of fine-grained material in your dirty gravels or sand may be insufficient to easily perform the dilatancy reaction and plastic limit tests. An alternative test would be the ribbon test (NRCS 2014). Knead a wetted sample of the fine-grained material until it is a putty-like consistency. Then press the soil sample over the outside of your index finger with your thumb to form a ribbon of soil. The intent is to create a ribbon of soil about ½ inch (12 mm) wide and as long as possible. The strength of the ribbon can be evaluated by holding one end of the ribbon and shaking gently (NRCS 2014). If no ribbon can be formed or the ribbon is weak, the fines are nonplastic or low plasticity like an ML soil. Any other response to the ribbon test would indicate fines which are plastic like those forming MH and CH soils.

The proceeding approaches for testing the fines in dirty gravels or sand emphasizes testing to clearly identify the fines as non or low plasticity. By default, any other test results would show the fines to be plastic to some degree.

Clean gravel and clean sand are further subdivided on the basis of the range of particle sizes within those two categories (USBR 1998; NRCS 2014). Table 7 shows clean gravels and clean sand will be either well-graded or poorly graded. This terminology can sometimes cause confusion between engineers and geologists (Johnson and DeGraff 1988). Well-graded is the standard terminology used by engineers to describe a wide range of particle sizes including intermediate amounts of intermediate particle sizes (USBR 1998). Geologists apply the term poorly sorted to this same description due to its implications for interpreting sediments. It is recommended that an engineering geologist generally use the engineering terms to avoid miscommunication.

If the clean gravels or clean sand do not display a wide range of particle sizes including intermediate particles sizes within their respective category, then they would be termed poorly graded (USBR 1998; NRCS 2014). A poorly graded clean gravel would mean the material is all one size class such as being all fine gravel (see Table 1) or has all the particles sizes expected for gravel but with some intermediate particle size classes being absent. These same particle size conditions for clean sand would be categorized as poorly graded. Distinguishing between well graded or poorly graded clean gravels or sand, in the context of soil field testing, is necessarily done by a visual assessment of the sample (NRCS 2014).

Field classification of soils under the USCS scheme should always endeavor to find a single designation for a soil.

Because soil field testing is a visual-manual process and significant estimation is used, there can be difficulty in clearly placing a soil in one category or another (USBR 1998; NRCS 2014). This may necessitate using two possible categories separated by a slash (/). This is sometimes termed a borderline designation. For example, the estimate of sand and gravel appearing nearly equal could result in a GP/SP designation. Similarly, unclear plasticity characteristics for a fine-grained soil might require a designation such as CL/ML. Every effort should be used to resolve these situations before resorting to a borderline designation (USBR 1998; NRCS 2014).

It is important to recognize a borderline designation based on soil field tests is not the same as the dual designation sometimes used in laboratory determinations (NRCS 2014). For this reason, the borderline designation use a slash (SP/SM) to separate the soil descriptors rather than the dash (SP-SM) applied to laboratory dual designations.

Description of Physical Properties of Soils

The previous sections on identifying fine-grained and coarse-grained soils provide the means for identifying in the field which USCS soil is dominant in an exposure or representative of the surface across an area. Describing soils found in exposures or encountered in surficial material across a study area will benefit from descriptors in addition to identifying the USCS soil type. Some of the commonly used descriptors are: (1) color, (2) moisture (water content), (3) consistency, and (4) structural characteristics of the soil mass (USBR 1998; NRCS 2014). Two additional properties of coarse-grained soils which may be worth reporting are angularity and shape (USBR 1998).

Soil Color

Color is a soil property important for distinguishing among the soils present in and area being studied. The color of a soil is derived from a number of factors. Geologic materials from which the soil is derived contribute to soil color through the effects of oxidation and hydration on the minerals present. Organic material alters the color derived from mineral material (NRCS 2014). Iron and manganese are particularly prominent minerals responsible for soil colors. So within a given locality, soil color may also be useful in identifying materials of similar geologic units (USBR 1998).

One of the most commonly used methods for determining soil color is comparing a soil to the soil color charts of the Munsell Color System (USBR 1998; NRCS 2014). Soil color is determined from moist soil (USBR 1998). For color determination, collect a lump of soil and break it open to expose a fresh, moist surface (NRCS 2014). If the soil is dry throughout, moisten (without glistening) the exposed surface by adding water a drop or two at a time. Color matching is done outdoors not because of the field setting but to take advantage of natural sunlight. Stand with the sun over your shoulder, allowing the sunlight to shine on the color chart and the sample. Estimate Munsell notation by holding the sample behind apertures (openings) separating the nearest matching color chips (Fig. 1). Use masks provided to facilitate determining color matches; the black mask is for dark-colored samples and the gray one is for intermediate and light-colored samples (NRCS 2014). It is unlikely the sample will match any one chart color perfectly, but the closest match will be evident. Then record the Munsell hue value/chroma, e.g., 10YR 5/8.

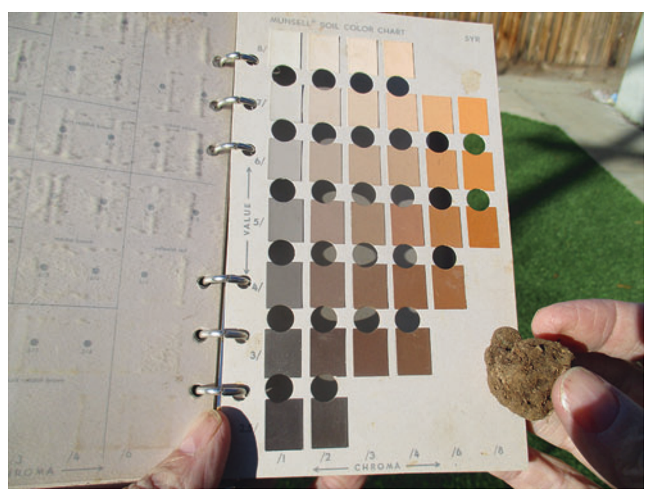
Because the quality of light is important when making a color determination, the sun should not be low on the horizon (NRCS 2014). For similar reasons, do not wear sunglasses when doing this determination. If the color of the soil when dry is requested or needed for the project, follow the above procedure (except for wetting). Be sure to record the color is for a dry sample because a moist sample is the expected condition.

Moisture Conditions (Water Content)

The moisture condition of a soil you are testing should be noted. This clarifies the field situation as you carried out your sampling work. Conditions are given in Table 8.

Consistency

Consistency, or degree of firmness, is determined for intact fine-grained soils. Table 9 provides the descriptors and criteria for making this evaluation.



Soil Field Tests, Fig. 1 Determining the color of a soil clod using the Munsell Soil Color Charts (Charts, MSC, 1994). The moist clod held at the right would be placed behind an opening (aperture) near color chips appearing to closely match the soil clod's color. In this instance, the soil color is 5YR (*yellow-red*) with a value of 3 (*left side vertical scale*) and chroma of 3 (*horizontal scale at bottom*). So the soil color to be recorded for this sample is 5 YR 3/3 (Photo by J.V. De Graff)

Soil Field Tests, Table 8 Describing soil moisture condition[a]

Descriptor	Criteria
Dry	Moisture is absent, dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible free water, usually in soil below local water table

[a]From USBR 1998

Soil Field Tests, Table 9 Describing consistency for in-place or undisturbed fine-grained soil[a]

Descriptor	**Criteria**
Very soft	Thumb will penetrate soil more than 1 inch (25 mm)
Soft	Thumb will penetrate soil about 1 inch (25 mm)
Firm	Thumb will indent soil about ¼ inch (5 mm)
Hard	Thumb will not indent soil but readily indented with thumbnail
Very hard	Thumbnail will not indent soil

[a]From USBR (1998)

Soil Field Tests, Table 10 Describing soil structure (fabric)[a]

Structure descriptor	Visual criteria
Homogeneous	No evident structure and, commonly, also the same color and texture. The soil shows none of the characteristics listed below
Stratified	Soil consists of alternating layers of varying soils or color. If layers are less than about ¼ inch (6.35 mm) thick, describe as laminated[2] (or varved if the layers are fine-grained dark and light in color associated with glacial activity)
Lenses	Small masses or pockets of soil different from the surrounding soil, e.g., small lenses of sand within a mass of clay
Fissured[b]	Soil that has a tendency to break along definite planes with little resistance to fracturing or exhibiting open cracks in an exposed surface. If the planes appear polished or glossy, describe them as slickensided
Blocky[b]	Soil which breaks into small angular lumps which resist being breakdown further

[a]Modified from USBR (1998), NRCS (2014)
[b]Structure descriptors which apply only to fine-grained soils except for laminated which can apply to fine sands

Structure (Soil Fabric)

Soil structure or fabric generally describes characteristics of the soil mass seen in exposures. These exposures may be natural such as a steep river bank or an artificial one created by trenching or excavating a pit (Table 10).

Particle Descriptions

As noted earlier, particles are an important aspect in describing soil. The gradation is a primary basis for determining soil classification under the Unified Soil Classification System (USCS) (Tables 1, 2, and 7). Additionally, the percentage of the total soil consisting of cobble and boulder-sized particles should be estimated and recorded. The shape and angularity of visible particles should also be observed and recorded (Tables 11 and 12).

Soil Field Tests, Table 11 Descriptors used for relating the shape of gravel, cobbles, and boulders[a,b]

Descriptor	Criteria
Flat	Particles with width/thickness >3
Elongated	Particles with length/width >3
Flat and elongated	Particles meet criteria for both flat and elongated

[a]For purposes of defining shape, length, width, and thickness refer to the greatest, intermediate, and least dimensions of a particle, respectively
[b]From USBR (1998)

Soil Field Tests, Table 12 Angularity descriptors for coarse-grained particles, only[a]

Descriptor	Criteria
Angular	Particles have sharp edges and relatively planar sides with unpolished surfaces
Subangular	Particles are similar to angular criteria but have rounded edges
Subrounded	Particles have nearly planar sides but well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

[a]From USBR (1998)

Summary

Soil field tests enable engineering to efficiently map and describe soil present in a project area. Describing soils with preliminary soil identification under the Unified Soil Classification System (USCS) facilitates communication with other technical specialists and gives an initial understanding of some soil conditions important to engineering design. The soil field tests can be applied rapidly to assess what soils are generally present including identifying those which may be critical to project success. Such information in the form of a map output can assist with comparison of alternative sites, provide input for project feasibility assessment, and serve as a basis for site design. Because laboratory testing for geotechnical data will be necessary to fully develop a project, soil field testing ensures that the location and number of laboratory samples are sufficient.

Cross-References

- ▶ Boulders
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Clay
- ▶ Dilatancy
- ▶ Engineering Geological Maps

▶ Engineering Geomorphological Mapping
▶ Exposure Logging
▶ Sand
▶ Silt
▶ Site Investigation
▶ Soil Laboratory Tests
▶ Soil Properties

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Soil Laboratory Tests

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Definition

Soil laboratory tests involve various experimental methods for the determination of soil properties for engineering design and evaluation. Tests are used to obtain basic soil information such as classification according to classification methods such as Unified Soil Classification System (USCS), grain size distribution, and plasticity characteristics as well as determining parameters needed for design and evaluation of infrastructure including the coefficient of permeability, compressibility, and shear strength of the soil mass.

Introduction

The proper design and construction of new infrastructure and evaluation of existing infrastructure greatly depends on the appropriate measurements of the parameters needed in the theoretical and empirical formulations for the problem at hand. One method of determining these parameters is by conducting laboratory soil testing on representative samples obtained from the project site. The procedures for conducting laboratory soil testing outlined by the American Society of Testing and Materials (ASTM) are typically adapted across the United States and many other countries. Also included within these ASTM standards are recommended tolerances for different measurements and the proper reporting procedures. This chapter provides a summary of the basic concepts and typical results from soil laboratory tests, and additional information is present in the referred ASTM standards.

Index Properties

Index properties indicate the probable engineering behavior of soils by providing information regarding the type and composition of the soil. They provide sufficient information to allow for the classification of the soil using one of the many available soil classification systems, including the USCS. Additionally, index properties are correlated with different parameters such as the coefficient of permeability, compression index, and friction angle of a soil mass. Two very important index properties of soils are particle size and Atterberg limits. The relative proportions of soil particles of different sizes are expressed as percentage of the total dry weight of the soil sample when conducting particle size analyses. Atterberg limits describe the water content that acts as boundaries between the different physical states (solid, semisolid, plastic, and liquid) of soil. The methods to perform particle size analyses and determine the Atterberg limits follow.

Sieve Analysis

In sieve analysis, the particle size distribution of soil is determined by mechanical sieving of soil samples through a series of graded sieves. Sieves consist of wires woven together to form square openings with larger sieve numbers corresponding to smaller openings. The procedures for conducting sieve analyses are outlined in ASTM D422-63 (2007), and the procedure is typically used for soils with negligible amounts of soil particles smaller than 75 μm. Sieve analyses are conducted by mechanical sieving of a representative sample through a stack of sieves with different

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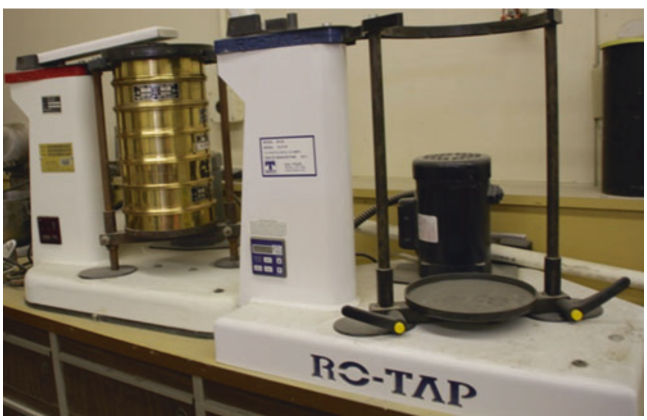
openings, arranged with the largest sieve size at the top and the sieve with the smallest opening at the bottom directly above a catch pan (Fig. 1). The mass of soil that is retained on each sieve and the pan is then measured and used to determine the percent retained on each sieve, which is later used to calculate the percent finer than or percent passing through a specific sieve size. The results are used to create a figure with the sieve opening size in a logarithmic scale on the horizontal axis and the percent finer on the vertical axis. An example grain size distribution curve is shown in Fig. 2. This curve is important to obtain proportion of different types of soils in a soil mass, proportionate distribution of various sized particles, and average particle size.

Hydrometer Analysis

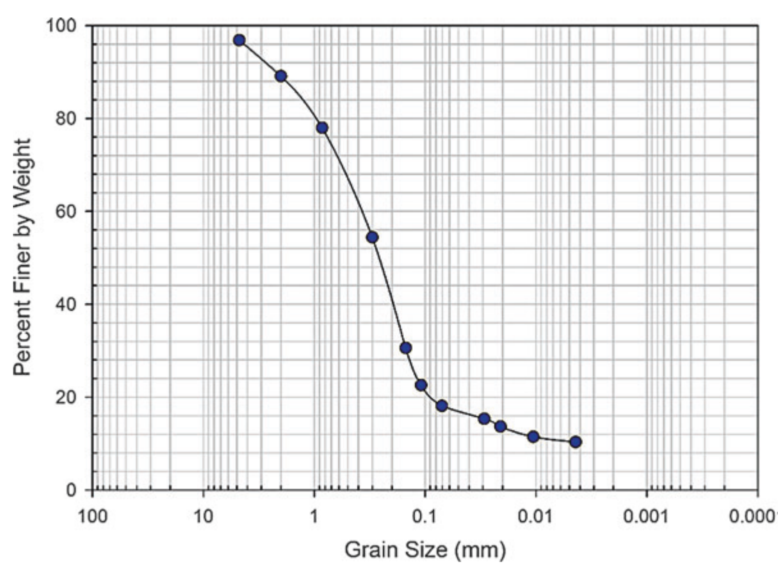
Hydrometer analyses are conducted to obtain the particle size distribution of soils with particles that are primarily smaller than 75 µm. To conduct the hydrometer analysis, a soil sample is first dispersed and fully agitated in water; the settling tube is then placed in a rest position, thus allowing the particles to settle individually. Stokes' law, then, serves as basis for the test. According to Stokes' law, the velocity by which particles settle will depend upon the shape, size, weight, and viscosity of the fluid through which they are settling. ASTM D422-63 (2007) contains the procedures for conducting hydrometer analyses, using an ASTM 152-H hydrometer, shown in Fig. 3a. This hydrometer gives the weight of soil in suspension above the centroid of the hydrometer bulb for a soil with a specific gravity of 2.65. Corrections must be applied for soils with different specific gravities. To conduct a hydrometer analysis, the soil is mixed with a dispersant, typically a 4% solution of sodium hexametaphosphate. An aqueous suspension, established using a mixer, is then transferred to an 18 in (46 cm) tall hydrometer cylinder with a diameter of 2.5 in (6.4 cm) and a capacity of 1000 mL, as shown in Fig. 3b. Changes in the density of the suspended liquid with time are measured. The results obtained are subsequently plotted to develop the grain size distribution curve. Combined sieve and hydrometer analyses are conducted when the soil mass contains significant proportions of particles that are both smaller and larger than 75 µm.



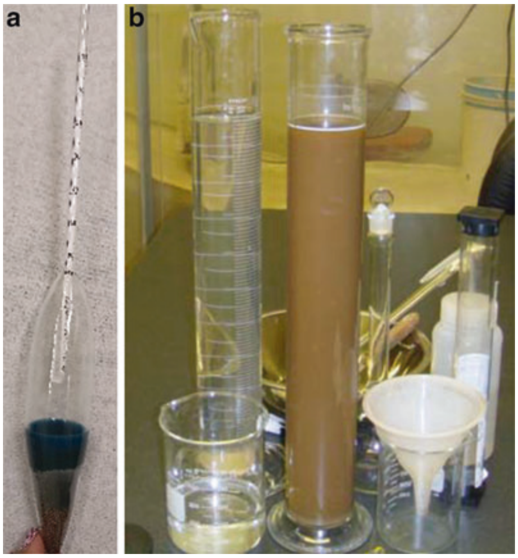
Soil Laboratory Tests, Fig. 1 Stack of sieves in mechanical shaker used to conduct particle size analysis

Specific Gravity

The specific gravity of soil solids is the ratio of the density of soil solids to the density of water. It is used in determining the weight–volume relationships for a soil mass and can be determined using the procedures outlined in ASTM D854-14 (2014). To determine the specific gravity of a soil mass, the weight of a 500 mL etched flask filled with distilled water is first measured (M_a). Half of the water from the flask is then removed and dry soil with weight M_o is placed in the flask with any soil clinging to the inside neck of the flask being washed into the base. The flask is next connected to a vacuum



Soil Laboratory Tests, Fig. 2 Example grain size distribution curve obtained from sieve analysis



Soil Laboratory Tests, Fig. 3 (**a**) ASTM 152-H hydrometer and (**b**) hydrometer cylinder with test in progress

for at least 2 h. To ensure that soil solids remain in suspension, the mixture is frequently agitated during the application of the vacuum. After removing the vacuum, the flask is filled with water and the weight is recorded (M_b). The specific gravity is found using Eq. 1. Correction factors are applied to obtain the value of the specific gravity at 20 °C.

$$G_s = \frac{M_o}{M_o + (M_a - M_b)} \tag{1}$$

Liquid Limit

The liquid limit of a soil mass refers to the moisture content corresponding to the transition of a soil specimen from the plastic to liquid state. The procedures for determining the liquid limit appear in ASTM D4318-17 (2017). Liquid limit measurement requires the use of Atterberg's device, as shown in Fig. 4. Specifically, the liquid limit is the water content at which a groove cut in a soil sample closes over a length of 0.5 in (1.3 cm) with 25 cranks; each crank refers to lifting and dropping the cup containing soil on the pad, when tested in a liquid limit device. An example of the flow curve obtained during a liquid limit test is given in Fig. 5.

Plastic Limit

The plastic limit is the moisture content at which a soil specimen transitions from a semisolid to solid state. ASTM D4318-17 (2017) describes the procedures for determining the plastic limit, which is the water content at which point a soil just begins to crumble when rolled by palm on a frosted glass into a 1/8 in (3.2 mm) diameter thread. When conducting a plastic limit test, the soil ball is usually formed at a moisture content greater than the plastic limit and rolled

Soil Laboratory Tests, Fig. 4 Atterberg liquid limit device

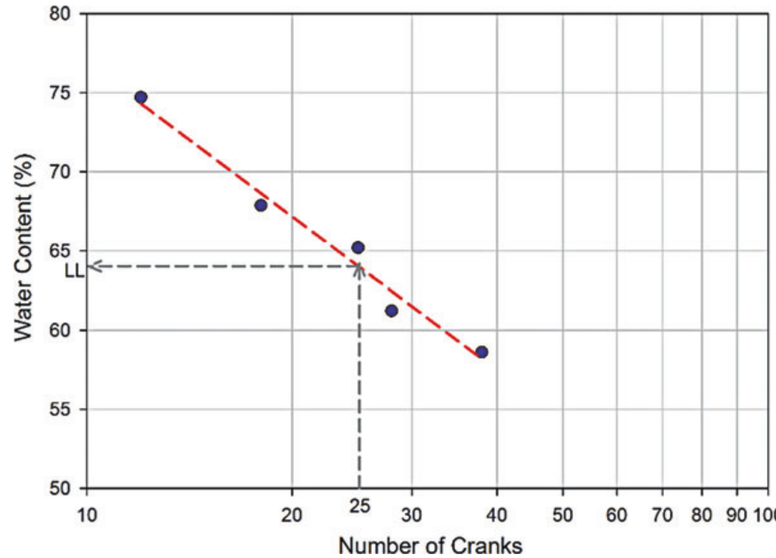
so that the moisture is adsorped by the hand and the frosted plate, as shown in Fig. 6. The plasticity index is determined as the difference between the liquid and plastic limits. It indicates the amount of water that can bind to a soil particle to mold the soil into different shapes without cracking and flowing.

Shrinkage Limit

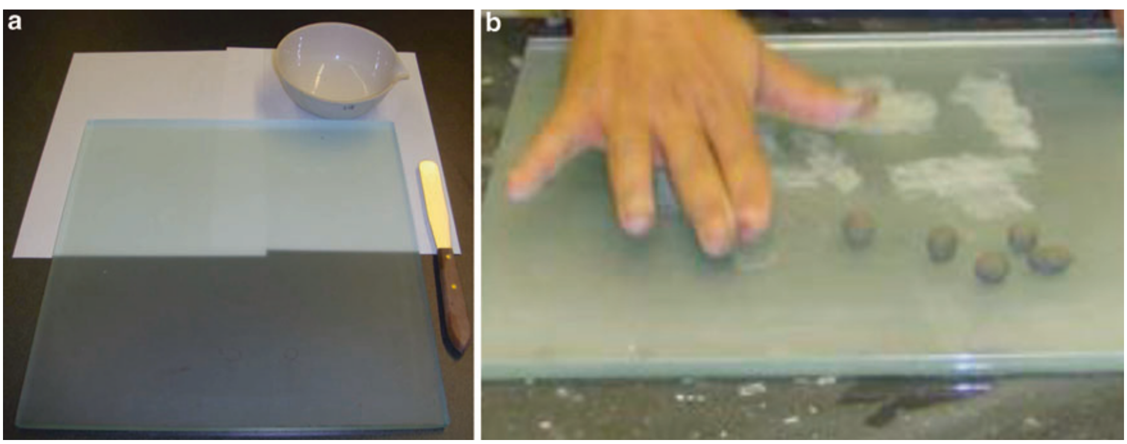
The third consistency limit is the shrinkage limit or the water content at which point a soil specimen transitions from a solid to semisolid state. The shrinkage limit is the water content at which point additional changes in the volume of a soil mass cease to occur with further reductions in water content. The procedures for determining the shrinkage limit are available in ASTM D4943-08 (2008).

Compaction

Compaction is the process of reducing the void space of a soil mass by removing air from the soil voids. It reflects an instantaneous process that occurs with the application of force. Laboratory compaction tests such as the standard and modified Proctor compaction tests are used to determine the maximum dry unit weight and optimum moisture content that provides the basis for field compaction specifications. ASTM D698-12e2 (2012) and ASTM D1557-12e1 (2012) describe the procedures for the standard and modified Proctor compaction tests, respectively. Both methods employ a similar procedure in that soil is compacted in a cylindrical mold having a fixed volume with a number of drops of a standard hammer that can supply a repeatable constant energy during each impact. The equipment used in conducting these tests is shown in Fig. 7. A moisture content–unit weight relationship,



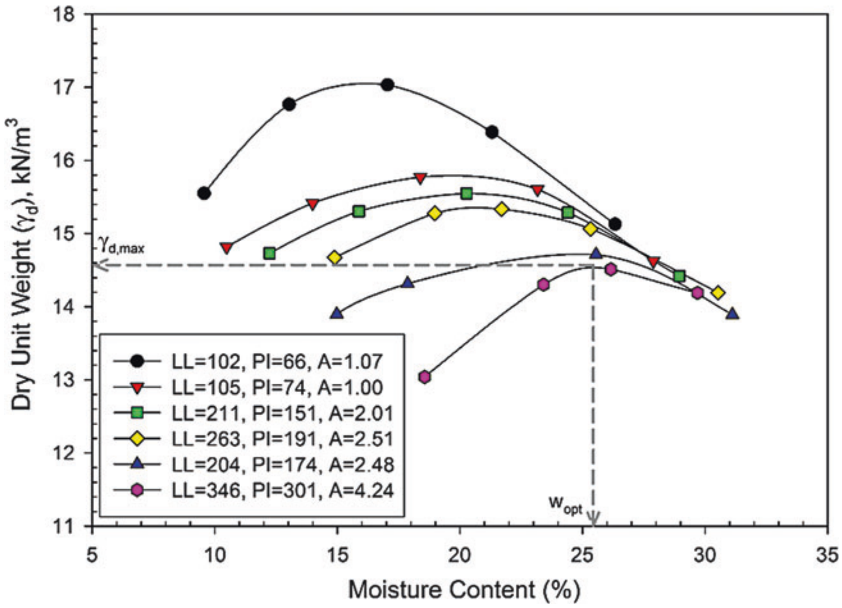
Soil Laboratory Tests, Fig. 5 Example of flow curve obtained from liquid limit test



Soil Laboratory Tests, Fig. 6 Picture of plastic limit test (**a**) equipment and (**b**) test in progress

such as those shown in Fig. 8, is developed by compacting the same soil mass at different moisture contents but with the same compactive effort. The peak of a smooth curve connecting the points of the moisture–unit weight relationship indicates the maximum dry unit weight of the soil, and the corresponding moisture content is referred to as the optimum moisture content. Although both the standard and modified Proctor compaction tests use a mold with a volume of 1/30 ft^3, the differences between the tests stem from the different energy inputs that result from the use of a different number of layers, hammer weight, and drop height. Specifically, the standard Proctor compaction test applies a total input energy of 12,400 ft-lb/ft^3 (600 kN-m/m^3) through three compaction layers, each subjected to 25 drops with a 5.5 lb. (2.5 kg) hammer dropped from a height of 12 in

Soil Laboratory Tests, Fig. 7 Equipment used to conduct standard and modified proctor compaction tests



Soil Laboratory Tests, Fig. 8 Example of moisture content–unit weight relationship obtained from modified proctor compaction test. Figure is adapted from Tiwari et al. (2014)

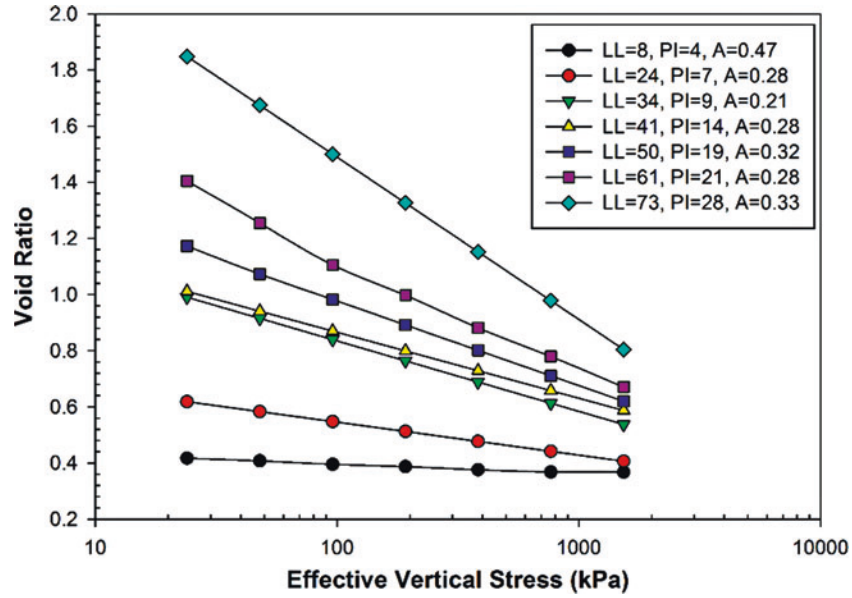
Soil Laboratory Tests, Fig. 9 Picture of a consolidometer

(30.5 cm). Whereas, in the modified Proctor compaction test, an input energy of 56,000 ft-lb/ft^3 (2700 kN-m/m^3) is applied through five compaction layers, each subjected to 25 drops from a 10 lb. (4.5 kg) hammer dropping from a height of 18 in (45.7 cm).

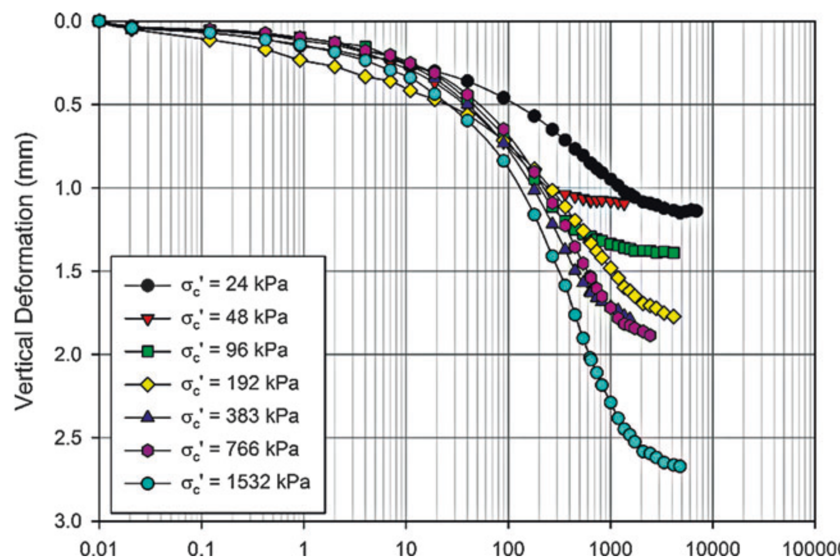
Consolidation

Consolidation is a time-dependent process that results in a reduction in the void space of a saturated soil mass following the expulsion of water from the voids due to the application of a static load. Consolidation tests are conducted following the procedures outlined in ASTM D2435/ASTM D2435M-11 (2011). Figure 9 illustrates the consolidation test setup and a consolidometer. In a consolidation test, a soil sample is placed between two porous stones in a brass ring. The assembly is transferred to the consolidometer where it is subjected to a vertical stress and the resulting deformation is recorded at different time intervals. A complete consolidation test will involve applying a series of vertical stresses to a soil sample until the pressured water drains in each loading step with the intent of determining the relationship between the void ratio and the vertical stress (typically plotted on a logarithmic scale) on the horizontal axis. An example of the void ratio versus vertical stress relationship obtained from a consolidation test appears in Fig. 10. This bivariate relationship provides information regarding the compression, recompression, and swelling indices of the soil as well as an estimate of the preconsolidation pressure. The preconsolidation pressure is the maximum pressure a soil was subjected to in its history. Additionally, curves relating the settlement of the soil mass with time for a constant vertical stress obtained from a consolidation test are used to estimate the coefficient of consolidation using one of the available methods such as

Soil Laboratory Tests, Fig. 10 Example void ratio versus logarithm of vertical stress curves. Figure adapted from Tiwari and Ajmera (2011a). Slopes of each line indicate the compression index



Soil Laboratory Tests, Fig. 11 Example time (in logarithm scale) versus displacement curves for a soil mass at different consolidation stresses ($\sigma_c{'}$)



Casagrande's logarithm of time method or Taylor's square root of time method. Figure 11 is an example of the displacement versus logarithm of time curves. Coefficient of consolidation is used in calculating the time rate of settlement on clay after the application of external load.

Permeability

Permeability is a measure of the ease of flow of water through a soil volume. Two commonly used techniques to measure the coefficient of permeability for a soil mass in the laboratory are the constant head permeability test and the falling head permeability test.

Constant Head Permeability Test

The constant head permeability test is typically conducted for coarse-grained soils containing sand and gravel. The procedures for conducting this test are outlined in ASTM D2434-68 (2006). In the constant head permeability test, a soil sample is placed between two porous stones in an assembly as shown in Fig. 12. The inflow consists of a funnel constantly refilled with water in order to maintain a constant head on the sample. The volume of outflowing water over a period of time is

Soil Laboratory Tests, Fig. 12 Constant head permeability test equipment

recorded. The coefficient of permeability (k) can then be calculated using Eq. 2, in which V is the volume of water collected at a head difference of h through a soil sample with a cross-sectional area, A, and length, L, over a period of time, t. The measured coefficient of permeability must be corrected to obtain the value at a temperature of 20 °C.

$$k = \frac{VL}{hAt} \tag{2}$$

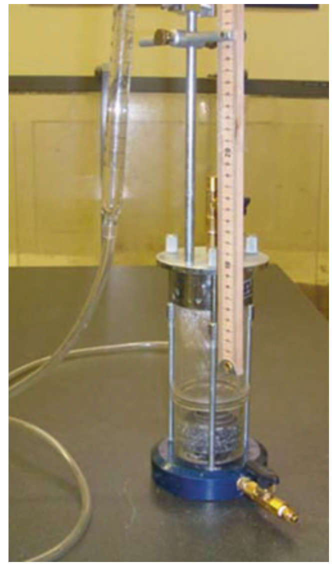
Falling Head Permeability Test

The falling head permeability test is used to measure the coefficient of permeability for fine-grained soils. The methodology for conducting a falling head permeability test is provided in ASTM D5084-16a (2016) and employs a setup similar to the constant head permeability test. In this test, the inflow is made through a standpipe with a cross-sectional area (a), which has an initial head of h_o at the start of the test. The head drops as the time progresses and a second head measurement (h_1) is taken after time, t, has elapsed. A photograph of the setup for a falling head permeability test is visible in Fig. 13. The coefficient of permeability is computed using Eq. 3, where A is the cross-sectional area of the soil sample. A temperature correction may be required to determine the coefficient of permeability at a temperature of 20 °C.

$$k = \frac{aL}{At} \ln\left(\frac{h_o}{h_1}\right) \tag{3}$$

Shear Strength

The shear strength of soil is the measure of the internal resistance of soil to shearing forces. It is governed by two factors, namely the friction between soil particles and the work required to cause volume changes within the sample.



Soil Laboratory Tests, Fig. 13 Picture of setup for falling head permeability test

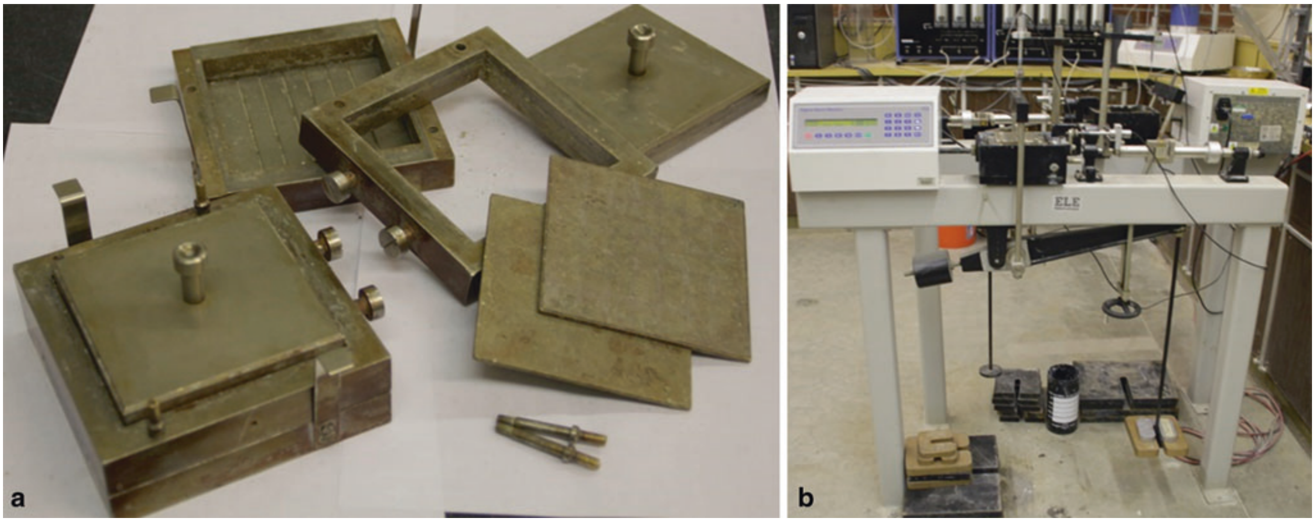
The shear strength can be measured using several laboratory testing procedures.

Direct Shear Test

The direct shear test is the simplest and most common test used to obtain the shear strength of the soil. As it is typically conducted under drained conditions (volume changes are allowed during the entire duration of the test), the direct shear test can be used to estimate the effective friction angle and cohesion intercept. ASTM D3080/D3080M-11 (2011) contains the procedures for conducting a direct shear test. To conduct a direct shear test, a soil sample is placed in a direct shear box, as shown in Fig. 14a, between two porous stones. It is then transferred to the direct shear device (Fig. 14b) where it is consolidated under a vertical stress before being subjected to a horizontal shear force that displaces the lower or upper half of the box against each other along a horizontal plane. During the shearing phase, information related to the vertical displacement, horizontal displacement, and shear force is recorded. Figure 15 illustrates the horizontal displacement versus vertical displacement and horizontal displacement versus shear stress curves obtained from a direct shear test. The test procedure continues until the peak shear strength is obtained and then it is repeated for a series of different vertical stresses in order to determine the effective friction angle and cohesion intercept corresponding to the Mohr–Coulomb failure envelope (Fig. 16).

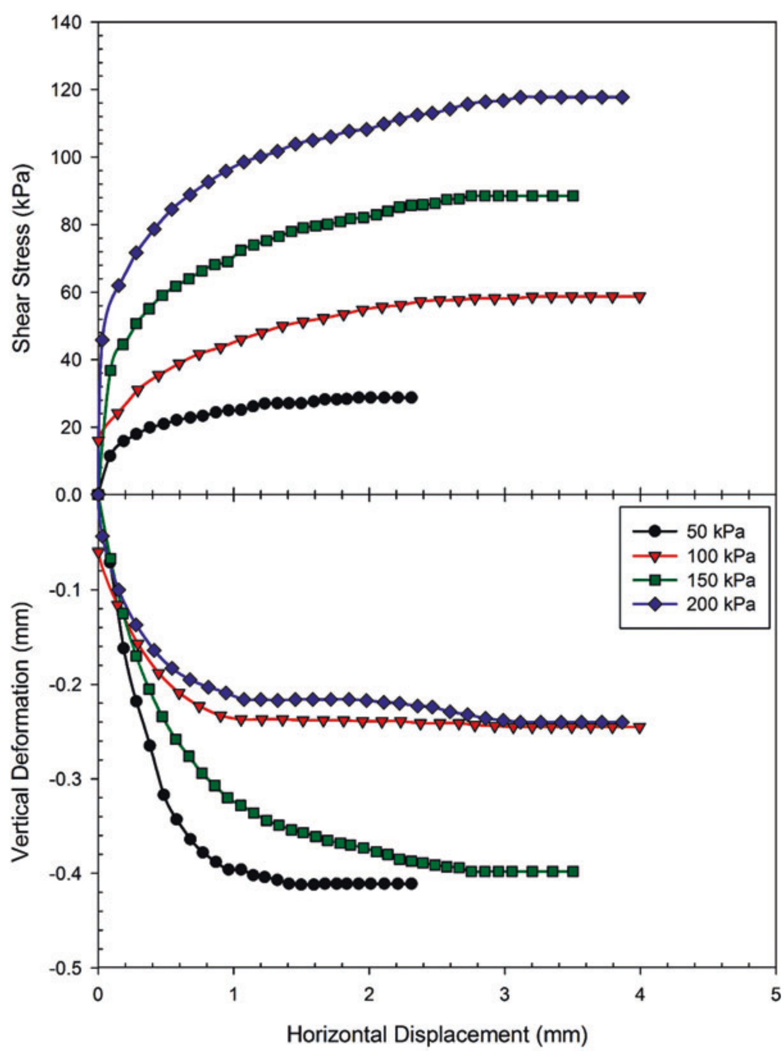
Triaxial Tests

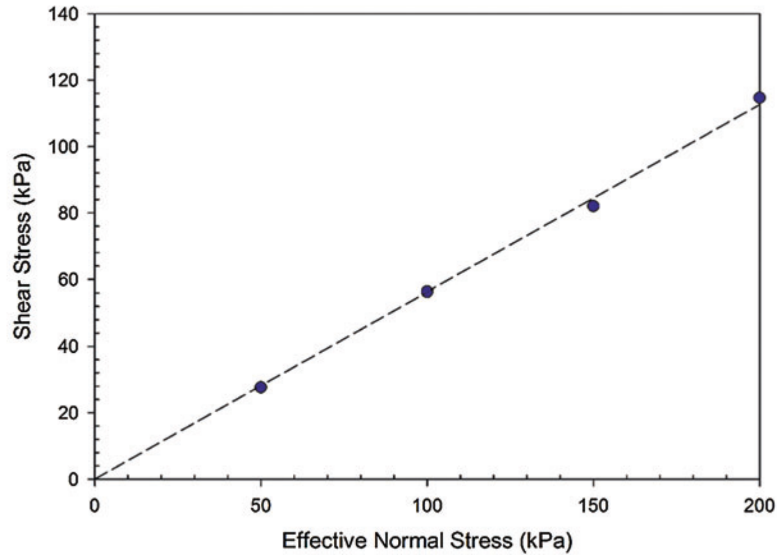
Triaxial tests are more sophisticated methods for determining the shear strength of a soil mass. Triaxial testing equipment (see Fig. 17) is used to conduct three different types of tests,



Soil Laboratory Tests, Fig. 14 Photographs of (**a**) direct shear box and (**b**) direct shear device

Soil Laboratory Tests, Fig. 15 Example of horizontal displacement versus vertical displacement and horizontal displacement versus shear stress curves from direct shear test, conducted separately at effective vertical stresses ranging from 50 to 200 kPa



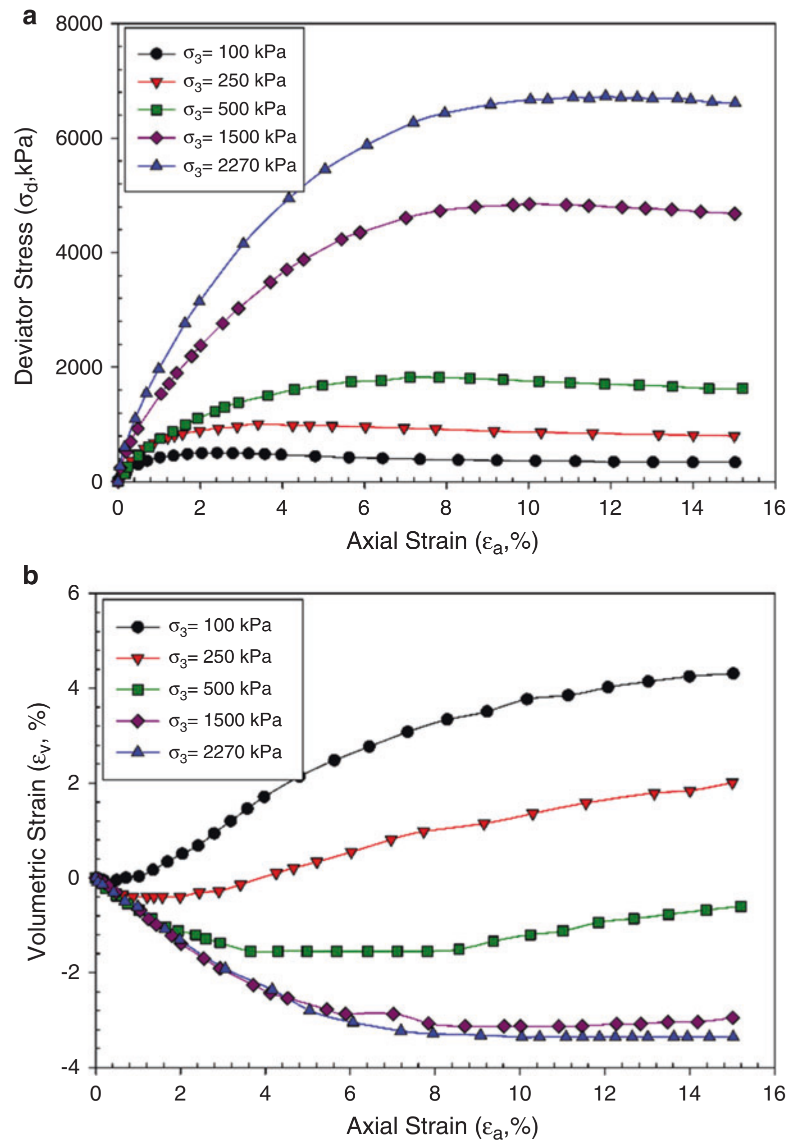


Soil Laboratory Tests, Fig. 16 Example of failure envelope obtained from direct shear test (Adapted from Tiwari and Ajmera 2011b)

Soil Laboratory Tests, Fig. 17 Photograph of triaxial test setup

which can measure the friction angle and cohesion intercept of a soil mass for both total and/or effective stress conditions. Specifically, the equipment is used to conduct consolidated-drained (CD), consolidated-undrained (CU), and unconsolidated-undrained (UU) triaxial tests. In all three types of tests, a cylindrical soil specimen with a height to diameter ratio of at least two is used. This specimen is sandwiched between two porous stones and encased in a thin rubber membrane in a plastic cylindrical chamber. A cell (or confining) pressure is applied to the sample by increasing the pressure of the fluid in the chamber surrounding the sample. Then, the vertical stress applied on the sample is increased by raising the platen at a fixed rate in a strain-controlled test or by applying loads on the sample at a fixed rate in a stress-controlled test.

CD Triaxial Test: The procedures for conducting a CD triaxial test are given in ASTM D7181-11 (2011). In this test, the sample is mounted in the triaxial chamber, then the cell pressure is increased and the corresponding increase in the pore water pressure is recorded in order to determine the value of Skempton's pore pressure coefficient (*B*) using Eq. 4. The sample is typically considered saturated if the B-value is greater than 0.95, although different values may be adapted depending on the type of soil being tested. If the desired B-value is not achieved, a back pressure is applied to saturate the soil sample. After several hours, the cell pressure is increased again and the B-value is measured. This process is known as back-pressure saturation. If the desired B-value is achieved, the cell pressure is increased to obtain the desired confining pressure, a point when the sample is allowed to consolidate. Following the consolidation process, the sample is sheared at a specified strain rate, which is calculated based on the consolidation data to ensure that all excess pore pressures are dissipated during the shearing process. The shearing stage is allowed to continue until either the sample achieves a peak deviator stress or the sample experiences 20% axial strain, if the deviator stress continues to increase. In the latter case, the deviator stress at 20% axial strain is considered the

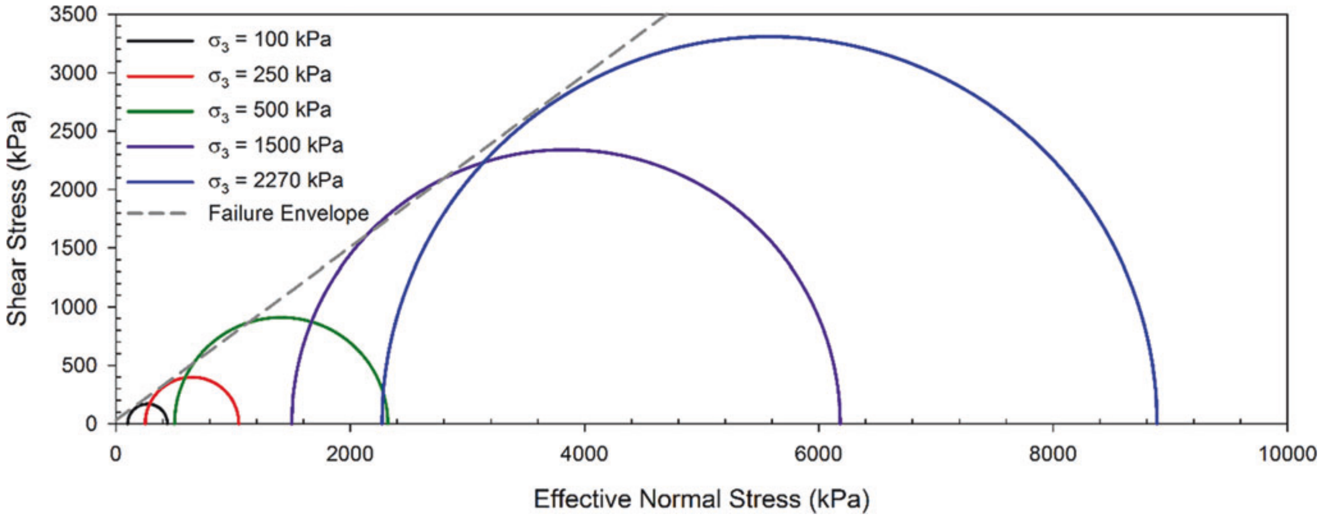


Soil Laboratory Tests, Fig. 18 Example of (**a**) axial strain versus deviator stress and (**b**) axial strain versus volumetric strain curves obtained from CD triaxial test conducted separately at confining stresses ranging from 100 to 2270 kPa

strength of the soil sample. During the shearing stage, data corresponding to the cell pressure, pore pressure, sample volume change, axial deformation, and deviator stress are recorded. The test is repeated for several different confining pressures. Figure 18 is an example of the axial strain versus deviator stress and axial strain versus sample volume change from a CD triaxial test. An example of Mohr circles and the resulting failure envelope obtained from CD triaxial tests are shown in Fig. 19.

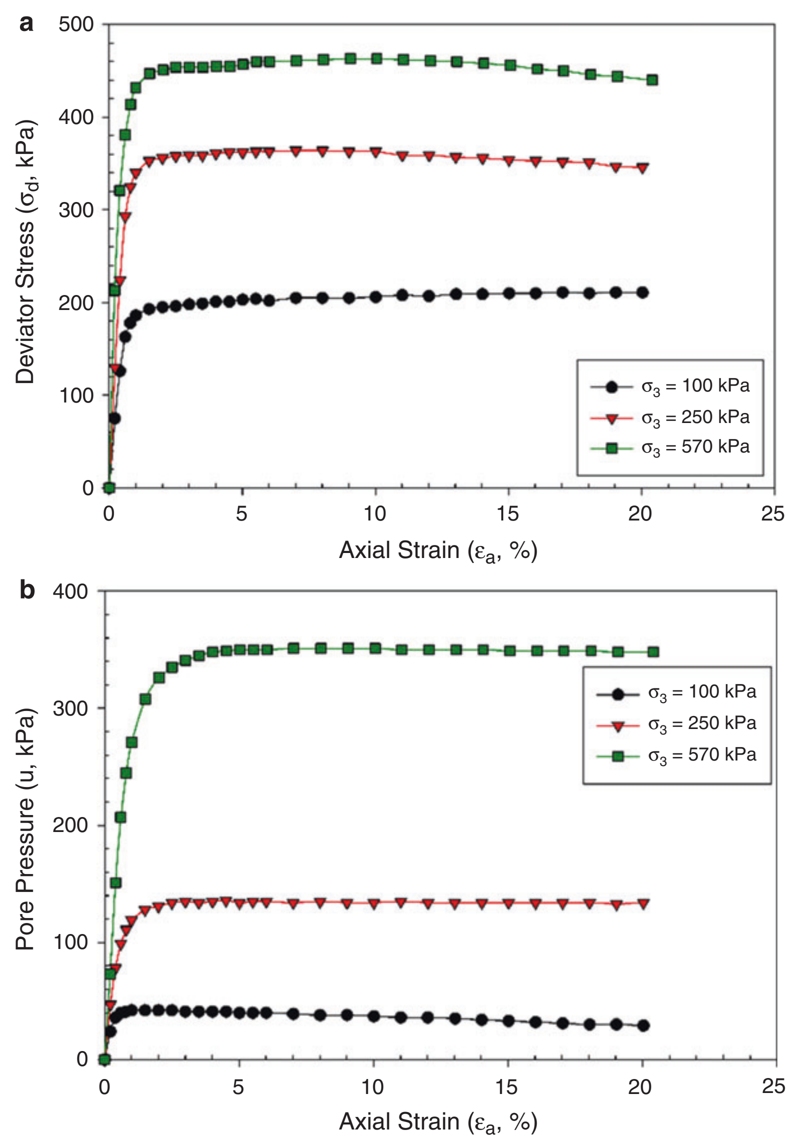
$$B = \frac{\Delta u}{\Delta \sigma_3} \tag{4}$$

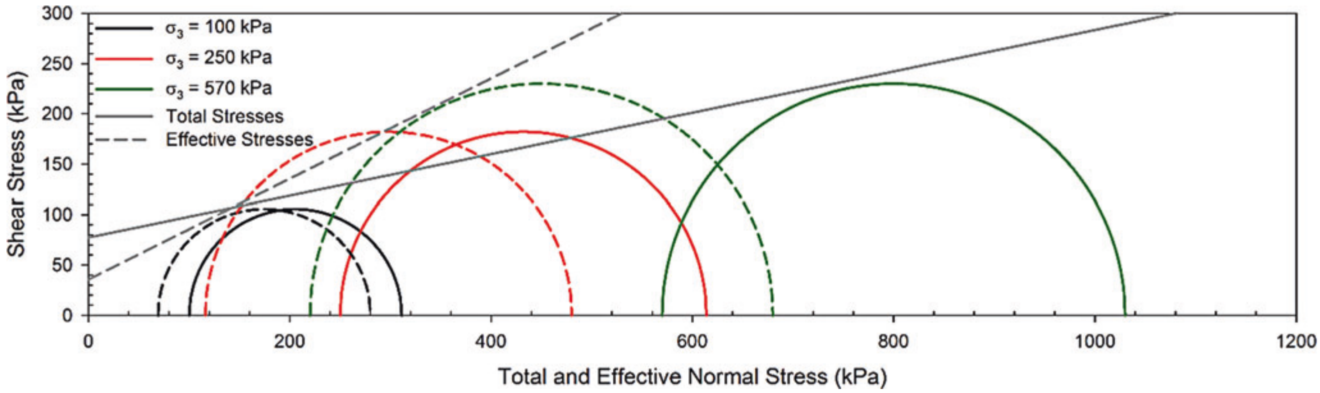
CU Triaxial Test: ASTM D4767-11 (2011) describes the procedures for conducting a CU triaxial test. The methodology described for the CD triaxial test is followed through the consolidation process after which the drainage valves are closed and the sample is sheared in an undrained condition (volume changes in the sample are not permitted), resulting in the generation of excess pore water pressures. The shearing rate is typically eight times faster than the rate used in a CD triaxial test. During the shearing phase, data related to the axial deformation, deviator stress, and excess pore water pressure are recorded. The shearing phase is terminated when the peak strength is obtained or the sample experiences 20% axial strain, whichever occurs first. The test is repeated on different samples for several different confining pressures. An example of the axial deformation versus deviator stress and axial deformation versus excess pore pressure curves obtained from a CU triaxial test is given in Fig. 20. The results from a CU triaxial test are used to determine the friction angle and cohesion intercepts corresponding to both total and effective stress conditions. Figure 21 contains an example of Mohr circles and failure envelopes for both conditions.



Soil Laboratory Tests, Fig. 19 Example of Mohr circles and corresponding failure envelope from CD triaxial test

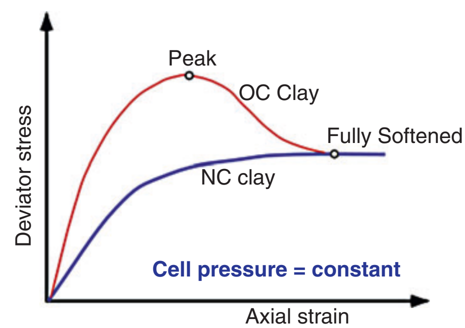
Soil Laboratory Tests, Fig. 20 Example (**a**) axial strain versus deviator stress and (**b**) axial strain versus excess pore pressure curves obtained from a CU triaxial test conducted separately at confining stresses ranging from 100 to 570 kPa



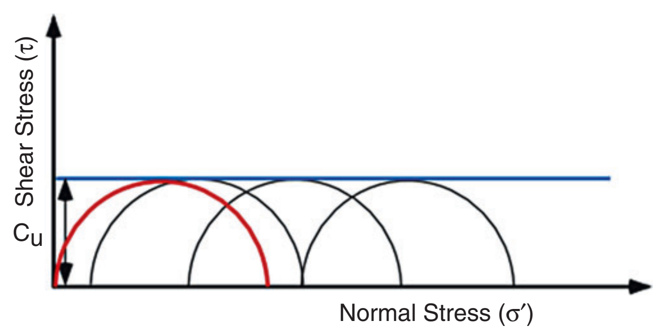


Soil Laboratory Tests, Fig. 21 Example Mohr circles and failure envelopes for total and effective stress conditions from a CU triaxial test conducted separately at confining stresses ranging from 100 to 570 kPa

UU Triaxial Test: The methodology employed to conduct a UU triaxial test is outlined in ASTM D2850-15 (2015). As drainage is not allowed in a UU triaxial test, consolidation becomes unnecessary. Therefore, regardless of the confining pressure, the deviator stress at failure will be approximately the same resulting in a horizontal failure envelope (or a friction angle of zero). The results from the UU triaxial test indicate the undrained cohesion for the soil sample. When conducting a UU triaxial test, the procedures through back-pressure saturation described for the CD and CU triaxial tests are followed. After saturation the sample is sheared at a high strain rate, typically 0.5%/min, until the peak deviator stress is recorded or until the sample experiences an axial strain of 20%. The axial deformation, deviator stress, and excess pore pressure during the shearing stage are recorded. The test is repeated on different samples at different confining pressures. Examples of the axial strain versus deviator stress curves and Mohr circles as well as the horizontal failure envelope obtained from a UU triaxial test are shown in Figs. 22 and 23, respectively.



Soil Laboratory Tests, Fig. 22 Example of axial strain versus deviator stress curves from UU triaxial test



Soil Laboratory Tests, Fig. 23 Example of Mohr circles and horizontal failure envelope from UU triaxial test

Unconfined Compression Test

Unconfined compression test is a quick method of determining the undrained cohesion of a soil mass. The unconfined compression test is conducted using the procedures in ASTM D2166/D2166M-16 (2016). When conducting an unconfined compressive test, a cylindrical sample, with a height to a diameter ratio of at least two, is mounted into the unconfined compressive strength testing device, as shown in Fig. 24. It is subjected to an axial load until failure. A shearing rate of 0.5%/min is typically applied to the sample during the shearing phase, which is continued until the peak deviator stress is measured or the sample experiences an axial strain of 20%. During the shearing phase, axial deformation and the deviator stress are recorded. The test is repeated several times. The axial deformation versus deviator stress curves obtained from an unconfined compression test are similar to that presented in Fig. 22, but with zero cell pressure. As such, the Mohr circle with zero cell pressure (minor principal stress) in Fig. 23 represents the result of the unconfined compression test.

Simple Shear Test

The simple shear test measures the shear strength of a soil mass by shearing it under plain strain conditions. The friction angle and cohesion intercept can be determined for both total and effective stress conditions. The procedures for running a simple shear test are in ASTM D6528-17 (2017). When conducting the simple shear test, the sample is confined by either a stack of Teflon® rings or a wire mesh. This allows shearing of the soil sample at a constant area throughout its height. Once the assembly is placed in the simple shear device (Fig. 25), the sample is consolidated to the desired vertical stress before it is subjected to a shear force. During the shearing phase, a constant volume is maintained by the device and the change in the effective vertical stress required to maintain a constant volume corresponds to the pore water pressure developed in the sample. Data pertinent to the shear strain, shear stress, and changes in the effective vertical stress (or pore water pressure) are measured during the shearing phase. The sample is sheared until the peak shear strength is measured or until the sample experiences 25% shear strain, whichever occurs first. The test is repeated for several samples subjected to different consolidation stresses. Figure 26 contains an example of the shear strain versus shear stress and shear strain versus pore water pressure curves, whereas Fig. 27 shows the failure envelopes for total and effective stress conditions.

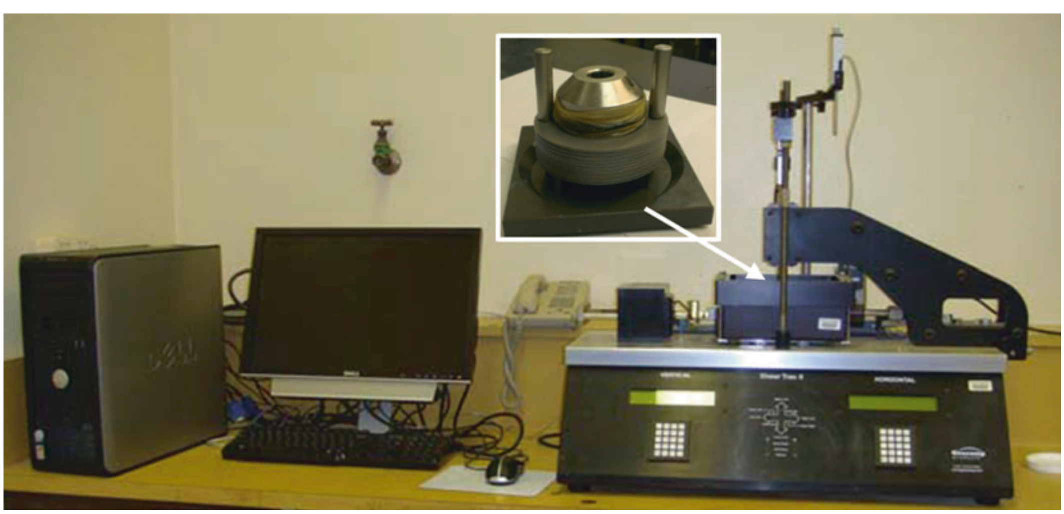
Soil Laboratory Tests, Fig. 24 Photograph of unconfined compressive test device

Ring Shear Test

The residual shear strength is the lowest shear strength of the soil mass and is achieved after large displacements. Since the ring shear device has an unlimited displacement capacity without changing the contact area, the ring shear test can be used to determine the residual shear strength of a soil mass. ASTM D6467-13 (2013) details the procedures for conducting a ring shear test. When conducting the ring shear test, an annular shaped sample is first consolidated to the desired vertical stress and then subjected to torsional shearing at the drained condition. During the shearing stage, the vertical displacement, torque, and horizontal displacement are recorded. A photograph of a ring shear device is given in Fig. 28. The example of the results obtained from the ring shear device is similar to the one obtained from the direct shear tests.

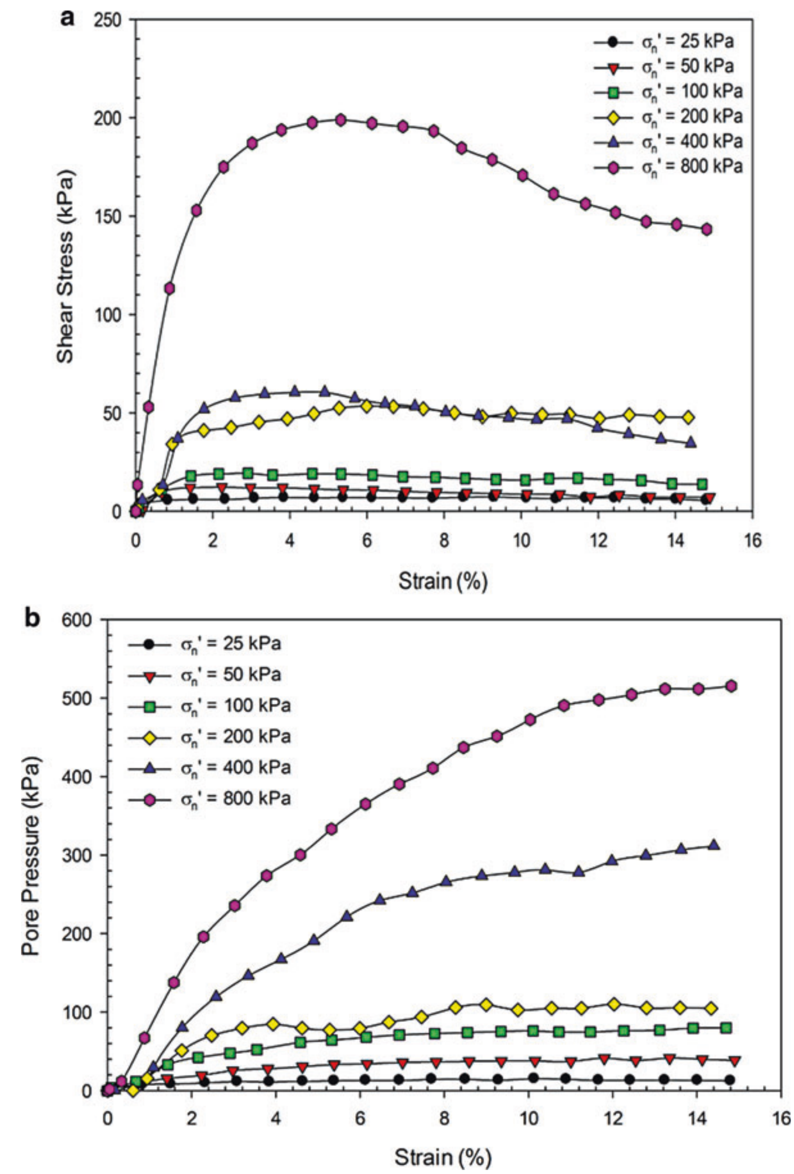
Cyclic Behavior

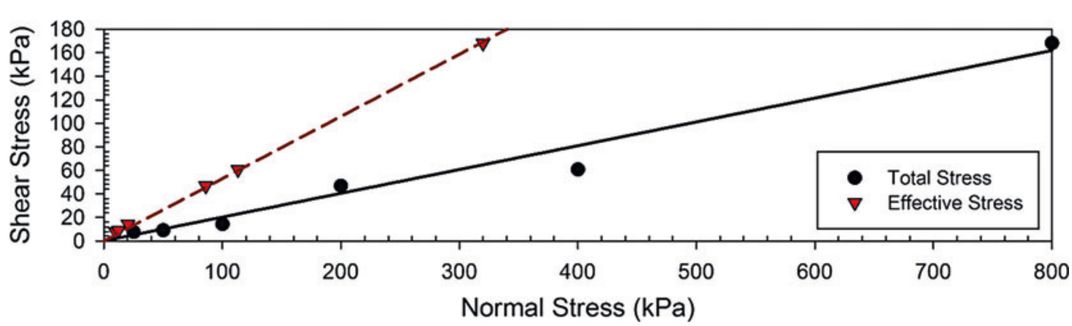
The dynamic properties of soils are required in a number of applications. These properties include the maximum shear modulus, shear wave velocity, and the variations in the shear modulus and damping ratio with shear strain. Additionally, the design and evaluation of infrastructure can also require knowledge of the reduction in shear strength that



Soil Laboratory Tests, Fig. 25 Picture of simple shear device

Soil Laboratory Tests, Fig. 26 Example of (**a**) shear strain versus shear stress and (**b**) shear strain versus pore water pressure curves from simple shear test conducted for effective vertical stresses ranging from 25 to 800 kPa (Adapted from Ajmera et al. 2012)





Soil Laboratory Tests, Fig. 27 Example of failure envelopes obtained from simple shear test (Adapted from Ajmera et al. 2012)

results in soils due to the application of cyclic loads. Some of the testing methods used to evaluate dynamic soil properties follow.

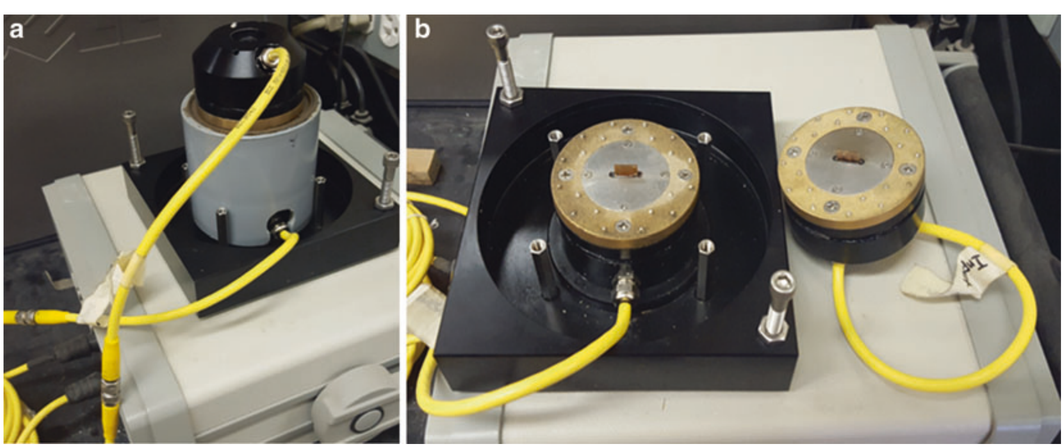
Bender Element Test

The bender element test determines the maximum shear modulus and shear wave velocity of a soil mass. A bender element consists of two piezoelectric devices bonded together. A voltage across the face of a bender element will cause one piezoelectric device to contract and the other to expand resulting in the bending of the element, which when embedded in a soil mass transmits a wave through the sample and will cause lateral displacements through the sample reaching a second bender element some distance away. Each type of soil will have its own characteristics to transmit waves. The lateral displacements of the second bender element will produce a voltage that is recorded. The time required for the wave transmitted from the first element to be received by the second element is recorded along with the distance between the tips of the two elements. This information is used to compute the velocity of the wave. A picture of a bender element test appears in Fig. 29, whereas Fig. 30 contains an example of the voltage recordings from the transmitted and received waves during a test.

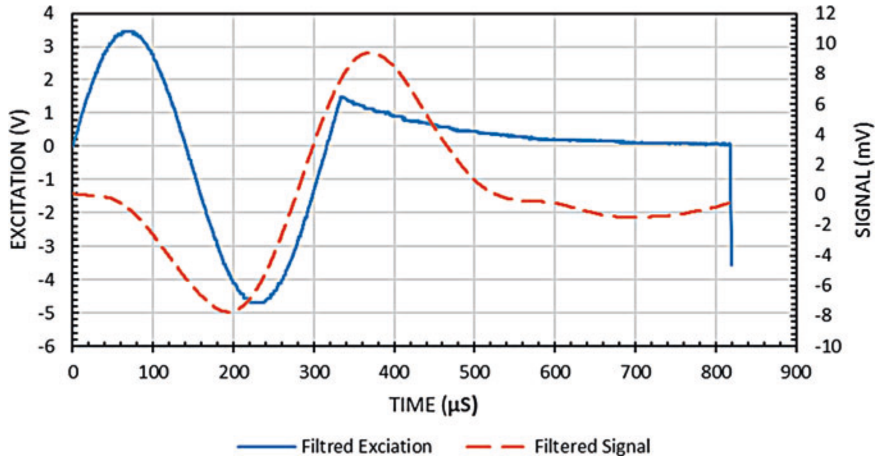
Soil Laboratory Tests, Fig. 28 Photograph of ring shear device

Cyclic Triaxial Test

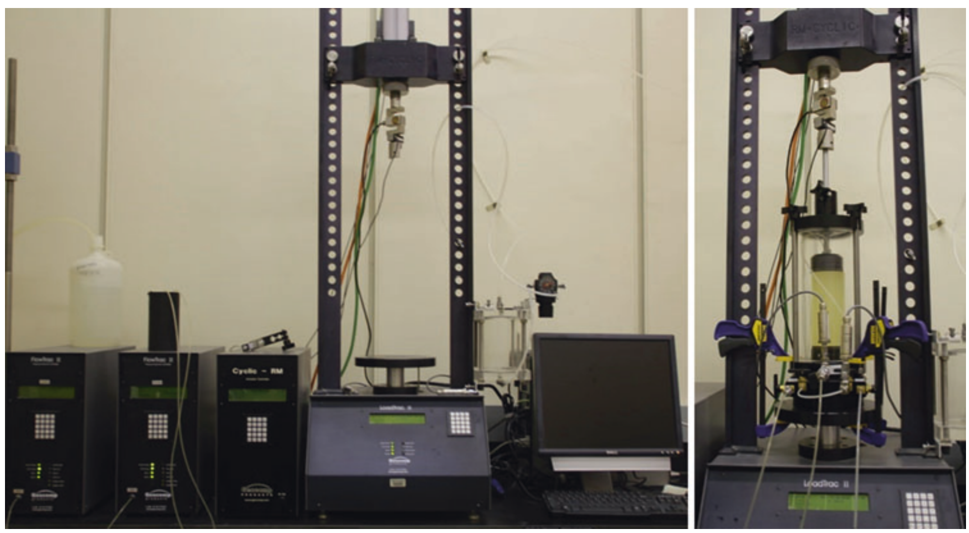
The cyclic triaxial test is probably the most commonly used method to determine the dynamic properties of soils in the laboratory. ASTM D5311/D5311M-13 (2013) provides the procedures for conducting a cyclic triaxial test in stress-controlled conditions. Although the cyclic triaxial test can also be conducted under stain-controlled conditions, an ASTM standard that describes the suggested methodology is not currently available. Figure 31 contains a photograph of a cyclic triaxial device. The sample preparation and testing procedures for a cyclic triaxial test are the same as those described elsewhere for a triaxial test through the consolidation stage after which the sample is subjected to cyclic loads. In a stress-controlled test, cyclic loads are typically applied in the form of a sinusoidal wave function, whose amplitude is determined based on the cyclic stress ratio determined from Eq. 5 in which $\sigma_{d,cyc}$ is the amplitude of the cyclic axial stress and $\sigma_3{}'$ is the confining pressure applied during the consolidation phase. During the cyclic loading phase, data pertinent to the deviator stress, excess pore water pressure, confining pressure, and axial deformation are recorded. Termination criteria for the cyclic loading phase in a cyclic triaxial test can vary substantially in the literature and typically include at least one of the following: (1) a minimum value of the pore pressure ratio, or the ratio of the excess pore pressure to the confining pressure, (2) a minimum value of single amplitude (maximum strain from the origin in compression or extension) or double amplitude axial strain (the difference between the maximum strain in compression and extension during a given cycle), and/or (3) a maximum number of cycles of loading. ASTM D5311/D5311M-13 (2013) recommends either a double amplitude axial strain of 20%, a single amplitude axial strain in either compression or extension of 20%, or



Soil Laboratory Tests, Fig. 29 (**a**) Photograph of bender element test and (**b**) photograph of bender element in bottom platen



Soil Laboratory Tests, Fig. 30 Example of results obtained from bender element test



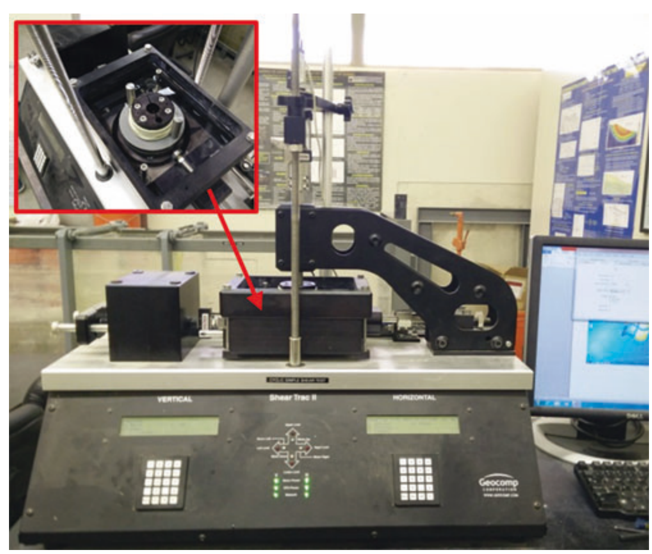
Soil Laboratory Tests, Fig. 31 Photograph of a cyclic triaxial device

a maximum of 500 number of cycles of loading. The soil sample is subjected to axial compression after the cyclic loading phase to measure the shear strength and its resulting reduction. The results from the cyclic loading phase are used to plot stress–strain hysteresis loops that can be used to determine the variation in the damping ratio and shear modulus with shear strain. If several stress-controlled cyclic triaxial tests are conducted, the results can be combined to determine the backbone curve and estimate the maximum shear modulus.

$$CSR = \frac{\sigma_{d,cyc}}{2\sigma_3{}'} \tag{5}$$

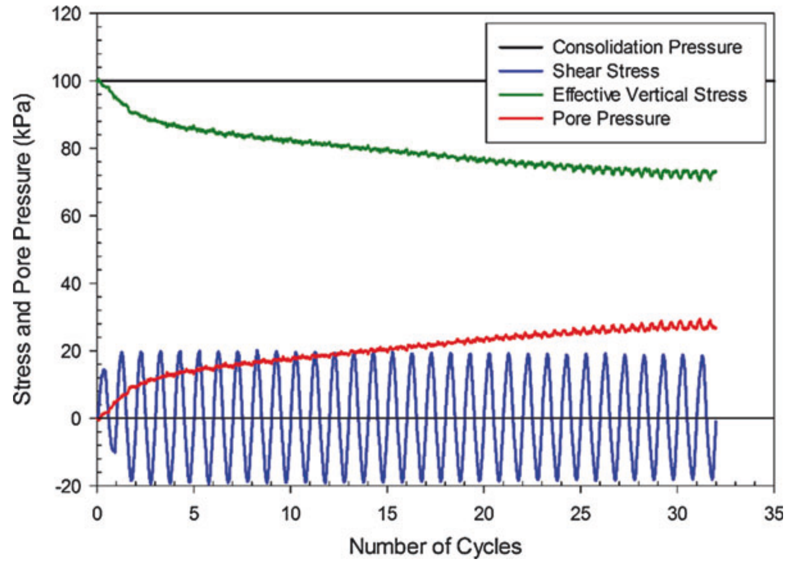
Cyclic Simple Shear Test

The cyclic simple shear test is another laboratory test that is used to determine the dynamic properties of soils. Figure 32 shows a cyclic simple shear device. The test is conducted in either stress-controlled or strain-controlled conditions. An

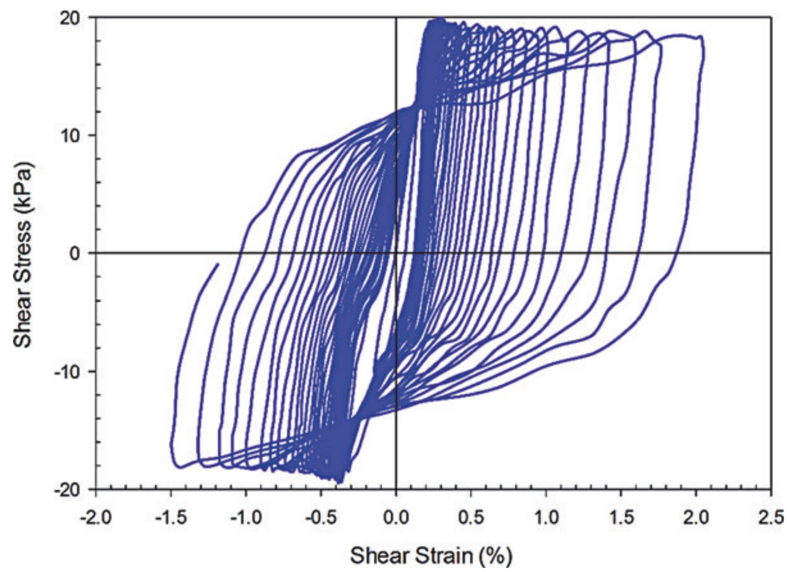


Soil Laboratory Tests, Fig. 32 Picture of cyclic simple shear device

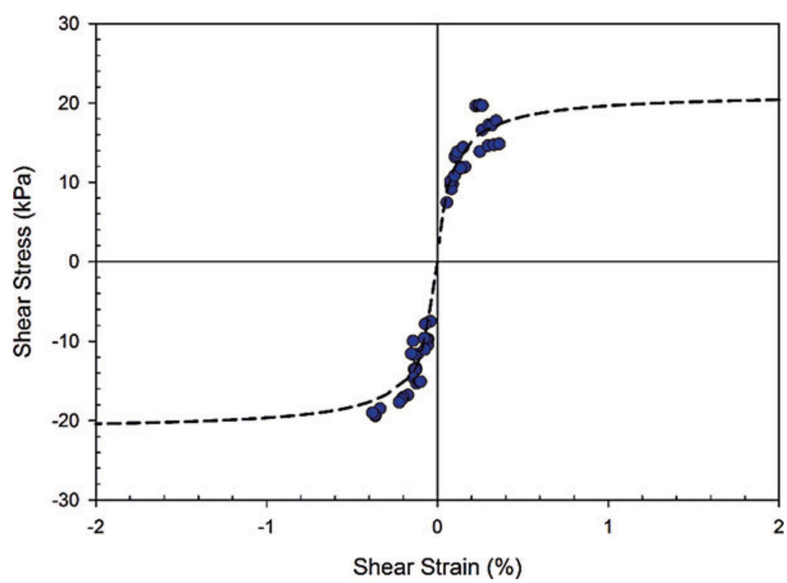
Soil Laboratory Tests, Fig. 33 Example of results obtained from cyclic simple shear Test (Adapted from Tiwari et al. in press)

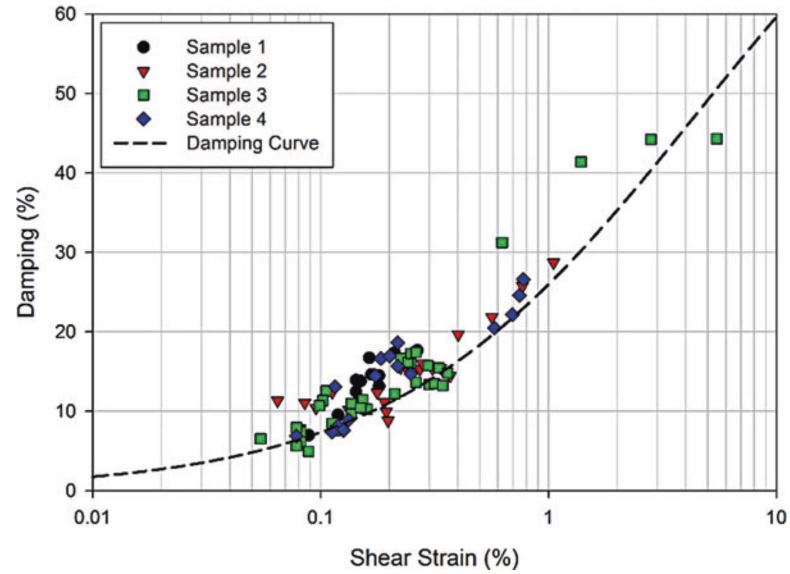


Soil Laboratory Tests, Fig. 34 Example of stress–strain hysteresis loops from cyclic simple shear test (Adapted from Tiwari et al. in press)

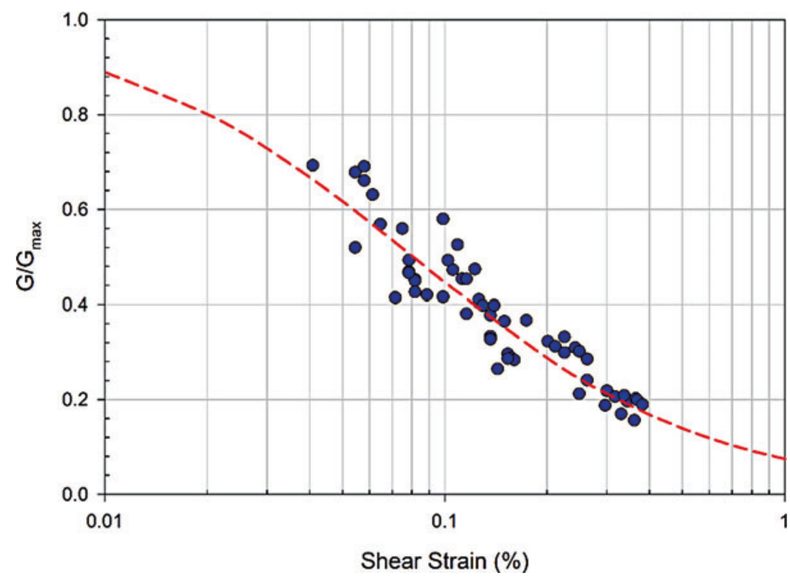


Soil Laboratory Tests, Fig. 35 Example of backbone curve from cyclic simple shear test (Adapted from Tiwari et al. in press)





Soil Laboratory Tests, Fig. 36 Example of the variation in the damping ratio with shear strain (Adapted from Tiwari et al. in press)



Soil Laboratory Tests, Fig. 37 Example of the reduction in the shear modulus with shear strain (Adapted from Tiwari et al. in press)

ASTM standard describing the procedures for a cyclic simple shear test is currently not available. Although the literature contains a number of procedures for conducting this test, Ajmera et al. (2017) adapts the procedures from ASTM D6528 (2017) for the simple shear test and ASTM D5311/D5311M-13 (2013) for the cyclic triaxial test to the cyclic simple shear test. Specifically, the sample preparation and testing procedures through the consolidation process are the same as those summarized elsewhere for the simple shear test. However, after the completion of consolidation, the sample is subjected to cyclic loads. This is typically in the form of a sinusoidal loading function whose amplitude in a strain-controlled test would be determined from the cyclic stress ratio, defined in Eq. 6. In this equation, τ_{cyc} is the amplitude of the cyclic stress and σ_c' is the consolidation pressure. The cyclic loading is applied to the sample until the desired termination criteria are reached. These criteria are typically defined as at least one of the following: (1) a minimum value of the pore pressure ratio, (2) a minimum single amplitude shear strain, (3) a minimum double amplitude shear strain, and/or (4) a maximum number of loading cycles. In Ajmera et al. (2017), these criteria are selected as 10% double amplitude shear strains or 500 cycles of loading, whichever occurs first. The shear displacement, effective vertical stress, and shear force are recorded during the cyclic loading phase. The data collected (Fig. 33) are used to draw stress–strain hysteresis loops, such as those shown in Fig. 34, the backbone curve, shown in Fig. 35, and determine relationships for the damping ratio with shear strain (Fig. 36) and the reduction in the shear modulus with shear strain (Fig. 37).

$$CSR = \frac{\tau_{cyc}}{\sigma_c{}'} \quad (6)$$

Summary

The soil properties required for engineering design and evaluation can be determined through laboratory soil testing. Available laboratory tests are used to obtain information about the basic soil properties ranging from the data required to classify the soil to understanding its Atterberg limits to establishing the type and composition of a soil mass. These can provide estimates of different soil properties through available correlations and prior experience. Additionally, laboratory soil testing is used to determine specific parameters such as those needed for compaction control, determination of seepage discharge, settlement due to external load, bearing capacity of foundation, or in the evaluations of stability of slopes. Laboratory soil testing is also used to establish the dynamic properties of the soil mass for different applications.

Cross-References

► Atterberg Limits
► Casagrande Test
► Characterization of Soils
► Classification of Soils
► Consolidation
► Dynamic Compaction/Compression
► Engineering Properties
► Liquid Limit
► Mohr Circle
► Mohr-Coulomb Failure Envelope
► Plastic Limit
► Plasticity Index
► Pore Pressure
► Shear Strength
► Shear Stress
► Strain

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ASTM D4318-17 (2017) Standard test methods for liquid limit, plastic limit, and plasticity index of soils. ASTM International

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Soil Mechanics

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Definition

Soil mechanics is a subdiscipline within civil engineering and geological engineering that is based on engineering mechanics, engineering geology, and soil physics (Mayne et al. 2002). Engineering mechanics is the field of study of forces acting on bodies and the response of those bodies in terms of motions, stresses, strains, and deformations (NAVFAC 1986). It is based fundamentally on physics and mathematics, with emphasis placed on solving problems and supporting engineering design. Mechanics of materials initially addresses ideal materials that are homogeneous, isotropic, linear, and elastic, and then includes nonideal materials, such as composite materials. Soil is a natural composite material consisting of solid, liquid, and gaseous phases; it is heterogeneous, anisotropic, nonlinear, and plastic to brittle depending upon whether it is completely dry, fully saturated, or partially saturated. Soil may exhibit elastic behavior over a small range of stress, but it also may be composed of discrete particles that act as a composite material only under the conditions of confinement. Many soil types exhibit creep deformation under constant stress applied for a substantial period of time, whereas other soil types exhibit substantial loss of strength under certain loading conditions that are applied for a short period. Some soil types transmit groundwater readily, whereas other soil types inhibit groundwater flow. In some contexts, soil mechanics may be considered with rock mechanics as end members of geomechanics.

Soil mechanics, as part of geotechnical engineering, involves acquiring and interpreting field, instrumentation, and laboratory data on soil, rock, and groundwater conditions; evaluating distribution of stresses including pressures on buried structures; analyzing settlement and volumetric expansion, seepage and drainage, and slope stability and protection; and understanding structural foundation loading data for design and construction of a variety of facilities. In addition to theoretical aspects of deformations resulting from load combinations, soil mechanics also involves the knowledge of construction methods, geology, and hydrology. Soil mechanics in support of engineering design is focused on the characterization of Earth materials for the purpose of understanding the behavior of soil materials, stratified and distributed soil systems, and soil-water mixtures, and predicting their performance when subjected to external loading imposed by excavations or construction of buildings or embankments supported by them. Soil mechanics provides the analytical tools to evaluate stresses, strains, and deformations in soil materials and the effects of fluids within pore spaces surrounding mineral grains. It is a subdiscipline of civil engineering (geotechnical engineering) that typically requires collaboration with professionals with expertise in other subdisciplines or disciplines, including engineering geology, structural engineering, water resources engineering, transportation engineering, and architecture.

Cross-References

► Borehole Investigations
► Characterization of Soils
► Classification of Rocks
► Classification of Soils
► Collapsible Soils
► Designing Site Investigations
► Dewatering
► Effective Stress
► Excavation
► Expansive Soils
► Foundations
► Geotechnical Engineering
► Groundwater
► Instrumentation
► Mass Movement
► Pore Pressure
► Shear Strength
► Shear Stress
► Site Investigation
► Soil Laboratory Tests
► Soil Properties
► Subsurface Exploration

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Soil Nails

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Synonyms

Ground anchor installation; Tendon installation

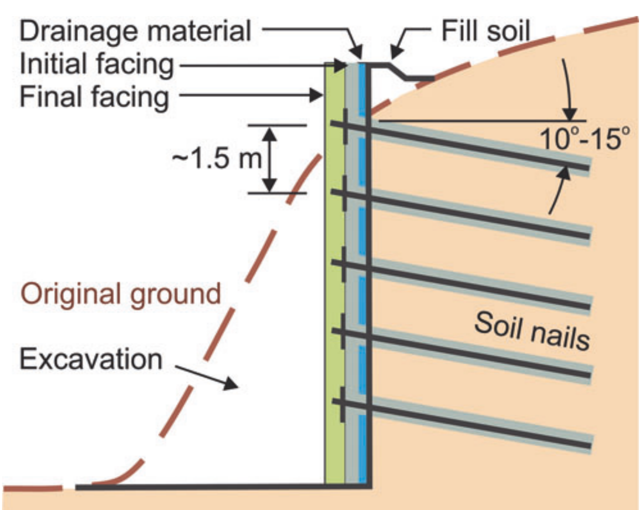
Definition

Soil nails are steel tendons installed to increase the stability of slopes or earth retaining walls. These are passive elements that reinforce the ground primarily to support excavations in soil and weak rock material and to stabilize slopes with relatively shallow slip surfaces (Lazarte et al. 2015).

The use of passive steel elements to stabilize soil material was an expansion of a method developed for stabilizing rock material in tunnel excavation with rock bolts and shotcrete. Reinforcing elements that are tensioned after installation (post-tensioned) are active elements called ground anchors or rock anchors. Passive elements develop tensile resistance as a result of ground deformation toward the excavation or slope face that produce shear stresses in the ground-soil nail system.

Soil nails (tendons) are installed in holes drilled into the soil or weak rock materials and then grouted in place. Tendon diameters typically range from 25.4 mm (#8 bar) to 34.9 mm (#11 bar) and can be solid or hollow; most soil nail tendons are threaded bars, but they can be deformed reinforcing steel bars. The ground-surface end of the bar must be threaded so that a nut can be used to hold a face plate in place at the excavation surface. Nominal tensile strength of tendons is 414 MPa (Grade 60, 60 ksi). Hollow bars fitted with sacrificial drill bits can be installed in self-drilled holes and then grouted without the need for a tremie pipe, whereas solid bars must be installed in pre-drilled holes and grouted using tremie pipes. The grout provides corrosion protection for the steel bar and load transfer by pullout resistance along the ground-soil nail interface. Centralizers are used to keep the tendons in the center of the hole.

A typical soil nail wall is illustrated in Fig. 1. Soil nailing to stabilize open excavations is performed from the ground surface downward as the excavation progresses. Vertical spacing of soil nails typically is 1–1.5 m, which is consistent with common excavation lifts. Horizontal spacing of soil nails is 1.2–1.8 m. As each row of soil nails is installed, shotcrete is applied to hold surface soil material in place and



Soil Nails, Fig. 1 Schematic illustration of a typical soil nail wall

to distribute the load taken by each of the soil nails. Soil nail walls are permanent facilities and can be used in combination with other stabilizing features, such as ground anchors, or earth retaining systems, such as mechanically stabilized earth (MSE) walls.

Cross-References

▶ Retaining Structures
▶ Shotcrete

References

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Soil Properties

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Definition

Soil consists of the mass of solid particles produced by the physical and/or chemical disintegration of bedrock found in various thickness mantling the ground surface (Johnson and DeGraff 1988; USBR 1998). It may or may not contain some proportion of organic material. For engineering geologic purposes, soil should be considered as a mass consisting of the solid particles and the intervening spaces between particles containing either air and/or water (Johnson and DeGraff 1988). This perspective is important because the qualities needed to use soil or some fraction thereof as a building material, to support structures, or to excavate into it are controlled by the mineralogical and physical character of the solid particles in combination with the presence and proportion of air and/or water in the void spaces.

Introduction

Three general properties make soil an important Earth material for human activities. These soil properties are its: (1) relative abundance, (2) widespread occurrence, and (3) being workable. Whether a residual soil formed by physical or chemical weathering of underlying bedrock or a deposit of soil material transported by one or more geomorphic processes, the quantities of soil can commonly be excavated near their intended use. Soil development takes place throughout the world making this Earth material widely available where it is needed. Excavating and shaping soil into a desired form can be done with the simplest of tools. Transport can be accomplished by baskets and bags carried by individuals or animals if necessary.

In prehistoric times, it is likely nomadic people constructing seasonal temporary shelters of brush or other vegetation would mix soil and water to make the mud applied for sealing the structure's interior from wind or rain (Revnelta-Acosta et al. 2010). In arid and semiarid regions where wooden building materials was scarce, soil was used to make adobe blocks either by sun-drying a soil, water, and organic mixture or by mechanically compressing the blocks. The utility of building with this material is evident by present-day structural block remnants at Jericho, Turkestan, and Egypt representing construction from 9,000 to 3,200 BC. Construction with cut blocks of sod (grass-surfaced soil blocks retaining roots) were also used for large structures such as Neolithic passage tombs. While these tombs are present in Brittany, Wales, and England, one of the best-known examples is Newgrange in Ireland's Boyne River Valley (Fig. 1; Stout and Stout 2008). Similarly, soil construction of mounds to form a central platform monument within settlements was built by native cultures in both the American Southwest and Southeast around the start of the Common Era (CE) (Lindauer and Blitz 1997).

Although these large structural remnants are impressive examples of soil used in construction, houses built of adobe and other soil blocks were likely far more numerous. Certainly, housing employing these methods occurred in parts of the world as varied as Mexico and Central America, Cyprus, Yemen, and Spain (Revnelta-Acosta et al. 2010). Sod houses were a common way to build houses in England and Scandinavia during the seventeenth and eighteenth centuries. So it is not surprising that European settlers coming to the United States in the eighteenth and nineteenth centuries continued this practice especially in the extensive grasslands of the Plains states (Revnelta-Acosta et al. 2010). Despite the disadvantage of not being as resistant to water damage as other building materials used for construction, earthen-type structures are estimated to presently house over half the earth's population (Revnelta-Acosta et al. 2010). Such housing is not restricted to countries where alternative building materials are unavailable or considered too expensive as illustrated by an estimate that 20% of new building in Australia involves earth-based construction (Revnelta-Acosta et al. 2010).

Another activity with an equally long history of using soil for construction is agricultural irrigation (Mays 2008). Surface soil is readily trenched for canals and ditches to direct water from natural watercourses and water impoundments. Ancient examples are found in areas as diverse as Jordan, India, and Tanzania (Urban et al. 2013; Shaw and Sutcliffe 2003; Westerberg et al. 2010). In addition to canals, tunnels through soil and rock have also served society since ancient time as a means for bringing water to human settlements ranging from villages to cities (Mays 2008). Soil was also

Soil Properties, Fig. 1 Newgrange is a Neolithic passage tomb in Ireland located north of Dublin in the Boyne River Valley. Built round 3,000 BCE, this monumental structure is predominantly constructed of turf layers. Much of the building took place prior to the early Bronze Age making use of stone tools. Between 1967 and 1974, a reconstruction effort produced the notable white quartz wall (Stout and Stout 2008) (Photo by J.V. De Graff) used for dams to either divert flow from rivers and streams or to create reservoir impoundments to facilitate seasonal water needs. The Egyptians are attributed with having built one of the first large-scale dams in about 2650 BC called the Sadd-el-Kafara dam; 14 m in height with a crest length of 113 m (Mays 2008). Just as the use of soil to build houses persists to present-day, dams continue to be built with soil to impound water to serve agricultural, mining, and recreational uses.

Whereas the abundance, occurrence, and workable nature of soil have made it a common building material from ancient to modern time, simple (non-engineered) use of soil as a building material is recognized as having the potential for failure. Such structures are vulnerable to landslides, earthquakes, and erosion by both surface water and groundwater. Landslides can cause damage or destruction in multiple ways including: foundation undermining, drag, lateral impact, lateral pressure, impact from above, burial, and inundation (Campbell et al. 1985). Earthquakes can produce damaging effects directly from earthquake shaking or from inducing liquefaction in the underlying soil (Saatcioglu 2013; Desramaut et al. 2013). Adobe structures are especially susceptible to these earthquake-induced effects (Blondet et al. 2011; Saatcioglu 2013). Many of the deaths which occurred during earthquakes in El Salvador, Peru, Iran, China, and Chile between 2001 and 2010 are attributable to collapse or severe damage to adobe structures (Blondet et al. 2011). Earthen dams can be vulnerable to erosion by surface water or piping by groundwater and require regular inspection and maintenance to avoid catastrophic failure. Tragedies from earthen dam failure and associated flooding by released water include historic examples such as the Johnstown (Pennsylvania) flood of 1889 and more recently the Ka Loko dam (Hawaii) breach of 2006 (McCullough 1968; Godbey 2007).

Modern Soil Engineering

As taller and heavier structures such as towers and fortifications experienced failure due to exceeding their foundations soil's ability to support them, early engineers began to develop fundamental concepts and principles to avoid such problems in the future. Thus engineering of soil for structural purposes began in the eighteenth century with early concepts and principles such as internal angle of friction and the means for identifying lateral pressures and potential slip surfaces. Many of these early precepts were derived from the practical problems of developing adequate foundations and retaining walls (Das 2005).

The modern beginning of soils or geotechnical engineering is often attributed to Karl Terzaghi (1883–1963) with his publication of Erdbaumechanik in 1925 (Goodman 2002). Dr. Terzaghi not only put forth a comprehensive framework for engineering with soil, his continuing research identified a wide range of field and laboratory methods, principles, and concepts which remain fundamental to those who practice in this field of engineering today (Goodman 2002; Das 2005). As both a proponent and practitioner of properly understanding soils to ensure the best engineering design, he and his collaborator, Ralph Peck, advocated for carefully observing how soil behaved in actual field situations. Their perspective proposed that mistakes might arise by viewing soil only on a purely theoretical basis (Goodman 2002).

As noted in the earlier definition of soil properties, soil should be considered as a mass consisting of the solid particles and the intervening spaces between them, referred to as voids, containing either air and/or water. Modern engineering with soils is concerned with issues; for instance, will soil be strong enough to bear an external load such as the weight of a structure (Gonzalez de Vallejo and Ferrer 2011). The weight of the structure will put stress on the supporting soil mass. Differential settlement, liquefaction, or slope movement can all be manifestations of insufficient soil strength to resist that stress. If a soil will deform under an external load, a structure may function inefficiently as seen in Fig. 2. In the case of bridges, pipelines, and similar structures where the tolerance for misalignment is much less, actual structural failure may result.

A recognition of the limitations posed by a soil in a particular situation or location is important to most engineering projects. Consequently, many different representations were developed to identify important characteristics defining soil capability or suitability and to avoid soils-related problems by modifying structural designs (see ▶ "Young's Modulus," ▶ "Poisson's Ratio," ▶ "Bearing Capacity," ▶ "California Bearing Ratio," ▶ "Shear Modulus," ▶ "Consolidation," ▶ "Organic Soils and Peats," ▶ "Modulus of Deformation"). Many of these representations demonstrating different qualities or limitations of a soil are incorporated into engineering standards or building codes. Soil laboratory tests were developed to ensure consistent and reliable means for determining relevant values for these representations (see ▶ "Soil Laboratory Tests").

Descriptive Soil Properties and Relationships

Commonly, a soil mass is a three-phase system with solids, air, and water present (Johnson and DeGraff 1988; Gonzalez de Vallejo and Ferrer 2011). The most visible part of a soil mass is the solid particles present. The size of the soil particles can range from large, easily visible particles such as gravel and sand to small particles such as silt and clay (see ▶ "Sand," ▶ "Silt," and ▶ "Clay"). Their relative percentage by weight

Soil Properties, Fig. 2 A street in the historic town of Ribe in southwest Juteland, Denmark. Near this street is a Cathedral dating from 948. Known as the oldest town in Denmark, it dates from the early eighth Century. Visible on this street and others nearby are distortion to the buildings. In this view, it is especially evident in the near building on the right when comparing the orientation of the upper story windows to those in the lower story. Differential settlement due to the weight of overlying structures has created the distortion because the underlying soil contains clays and organic material and has water at a shallow depth (Photo by J.V. De Graff)

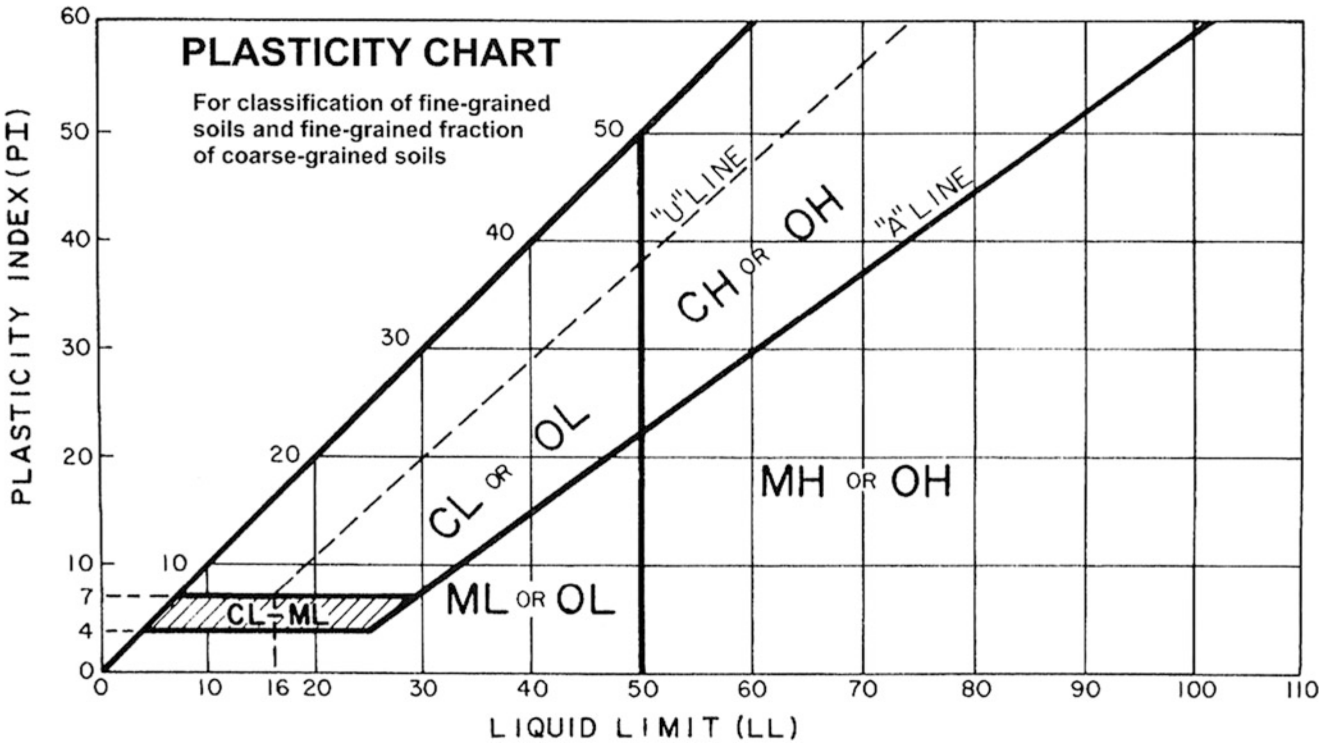
provides a particle size distribution that is plotted as a grading curve. The gradation of a soil is the basis for general classification called the Unified Soil Classification (Burns 2006; Gonzalez de Vallejo and Ferrer 2011). The classification scheme provides a consistent means for naming and communicating information about soil (Fig. 3). Using gradation of particles as the basis for classifications means soils bearing the same name under the Unified Soil classification are reasonably expected to behave in a similar manner regardless of where they are encountered. The classification's emphasis on solid particles is consistent with their predominance within a soil mass (Table 1).

The initial characterization is between coarse-grained or fine-grained soils based on the proportion of particles either retained or passed through ASTM sieve no. 200 (0.075 mm) (Fig. 3). More than 50% retained defines coarse-grained soil whereas more than 50% passed defines fine-grained soil. The coarse-grained soils are further subdivided based on particles passing through a no. 4 sieve (4.76 mm). If more than 50% are retained, it is a gravel soil and if more than 50% passes, it is a sandy soil. Further subdivisions are based on percentage of fines present, the nature of the fines (silt or clay) and grading character (Burns 2006; Gonzalez de Vallejo and Ferrer 2011).

Fine-grained soils are also termed cohesive soils. One of the most basic factors affecting how a soil's deformation behavior under the stress of an external load is the amount of water present. Deformation behavior in a fine-grained soil is referred to as soil consistency and can be defined by the Atterberg limits. The Atterberg limits include the shrinkage limit (SL), plastic limit (PL), and liquid limit (LL) which reflect the range of soil behavior changes attributable to increasing water content (Gonzalez de Vallejo and Ferrer 2011). The liquid limit is the boundary between a semi-liquid and a plastic state can be determined using either laboratory or field tests. Under the Unified Soil Classification, fine-grained soils are subdivided into two categories of silt and clay on the basis of their liquid limits to reflect those fine-grained soils which have high plasticity from those that do not (Burns 2006; Fig. 3). These divisions can be further subdivided by whether the clay is organic or inorganic (Burns 2006; Gonzalez de Vallejo and Ferrer 2011).

The Unified Soil Classification provides a uniform means for describing soils either visible at the ground surface or exposed by excavation or drilling. This facilitates the correlation of characteristics important to the design of structures so that experiences with specific foundation designs can be accumulated to demonstrate which are best used in certain soils and those designs which may perform less favorably. The consistent naming conventions derived from the Unified Soil Classification system makes subsurface correlation between boreholes more reliable (USBR 1998). Field evaluation methods can reasonably identify different soils under this classification systems. Field identification is certainly easier for the coarse-grained soils due to the visibility of the predominant particles sizes. Particle-size observations can be supplemented by describing the angularity and shape of particles (USBR 1998). For fine-grained soils, the difficulty in visually identifying the particles present can be compensated for by using field tests for dry strength, dilatency, toughness, and plasticity (USBR 1998). Other field observations are suitable for both coarse and fine-grained soils. These include soil structure, moisture conditions, cementation, and color, usually based on the Munsell soil color charts.

As stated elsewhere, a soil mass is generally a three-phase system consisting of solids, water, and air. Two physical properties of a sample of soil are its weight and volume. Figure 4 illustrates these properties for a hypothetical sample from a soil mass. The total volume of this sample (V_t) is composed mostly of the volume of solids (V_s) and the volume



Soil Properties, Fig. 3 Plasticity chart used for defining fine-grained soils under the Unified Soil Classification system (USBR 1998). It is primarily a mechanism for consistently distinguishing between "silt" and "clay." Originally published by Arthur Casagrande (1947) one of the important researchers in modern geotechnical engineering, it enables distinguishing between lower and higher plasticity. Under the Unified Soil Classification system, fine-grained soil particles are silt if their liquid limit and plasticity index plot below the "A" line and are clay if the liquid limit and plasticity index plot above the "A" line

of the voids (V_v). If there is only air present in the voids, the volume of voids (V_v) and the volume of air (V_a) will be equal. Because some amount of water is likely to be present, the volume of voids (V_v) will be equal to the volume of air (V_a) plus the volume of water (V_w).

Weight relationships for our soil sample are slightly different but are still linked to the three-phases that can be present in a soil mass (Fig. 4). If only air is present in the voids, the total sample weight (Wt) will be equal to the weight of the solids (Ws). The weight of voids (Wv) would be zero because air has no weight in this situation. Only when all or part of the voids contain water do the voids add weight to the soil mass. Unlike the weight of solids (Ws) which can vary depending on the character of the particles present, the weight of water (Ww) is a constant defined as 9.81 kN/m3 (or 62.4 lbs/ft3).

The volume and weight relationships enable us to develop descriptions for a soil that are useful in understanding and predicting its performance as a material (Johnson and DeGraff 1988; Gonzalez de Vallejo and Ferrer 2011). One of the more important items is unit weight. Assuming the sample consisted of only solids and voids filled with air, we could determine the dry unit weight (γ_d). It would be determined by the weight of solids (W_s) divided by the total volume (V_t). Dry unit weight would be the expected condition for a soil sample which has been oven-dried.

If it is assumed that all the voids in the soil sample are filled with water, we could determine the saturated unit weight (γ_{sat}). This value would be the ratio of the weight of the solids (Ws) and weight of water ($V_w \times 9.81$ kN/m^3) to the total volume (V_t). In a saturated soil sample, the volume of water (V_w) would be equal to the volume of voids (V_t). So either volume could be multiplied by the unit weight of water to yield the total weight of water for the saturated soil sample.

The weight measure that is often of the most interest, it the unit weight of a soil. This assumes the soil sample has a total volume including a volume of solids (V_t) and a volume of voids (V_v) containing both air and water. So in reality, the unit weight of soil is a moist unit weight (γ_m) and represents the ratio of total weight to total volume.

Understanding the volume and weight relationships for soil is important as a means to develop a number of insights into soil behavior. For example, the solid particles of a soil

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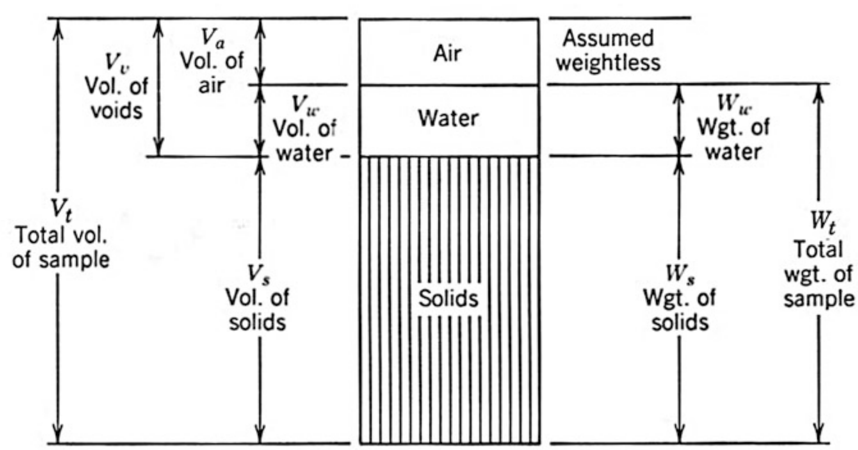
Soil Properties, Table 1 Chart showing the elements of the Unified Soil Classification System. While the appropriate name to apply to a particular soil may require data from laboratory testing, a preliminary name can be readily assigned based on simple field tests (USBR 1998). As Burns (2006) notes, soils which have characteristics of two different soils can be designated by combining group symbols, i.e., GW-GC

Major divisions			Typical descriptive names	USC symbol
Coarse-grained soils (more than 50% material larger than no. 200 sieve)	**Gravels** (more than 50% retained on no. 4 sieve)	**Clean gravels** (<5% fines)	Well-graded gravels, gravel-sand mixtures	GW
			Poorly graded gravels, gravel-sand mixtures	GP
		Gravels with fines (>12%)	Silty gravels, gravel-sand-silt mixtures	GM
			Clayey gravels, gravel-sand-clay mixtures	GC
	Sand (more than 50% passes no. 4 sieve)	**Clean sand** (<5% fines)	Well-graded sand, gravelly sand, little/no fines	SW
			Poorly graded sands, gravelly sand, little/no fines	SP
		Sand with fines (>12%)	Silty sand, sand-silt mixtures	SM
			Clayey sand, sand-clay mixtures	SC
Fine-grained soils (more than 50% material smaller than no. 200 sieve)	**Silt and clay** (liquid limit less than 50)	**Inorganic**	Inorganic silts and very fine sands, rock flour, silty or clayey fine sand, or clayey silt, with slight plasticity	ML
			Inorganic clay of low to medium plasticity	CL
		Organic	Organic silt and organic silty clay of low plasticity	OL
	Silt and clay (liquid limit 50 or more)	**Inorganic**	Inorganic clay of high plasticity, fat clay	CH
			Inorganic silt, micaceous or diatomaceous fine sandy or silty soils, elastic silt	MH
		Organic	Organic clay of medium or high plasticity, organic silt	OH
Highly organic soils			Peat and other highly organic silt. Primarily organic matter, dark in color and organic odor	PT



Soil Properties, Fig. 4 A conceptual representation showing the relationship between volume and weight between the components of solid particles, water, and air within a soil mass. This type of representation is sometimes referred to as a phase diagram (Johnson and DeGraff 1988)

are essentially nondeformable under stress, the stress behavior of a soil mass is dependent on the proportion of the voids containing air and water. Being able to characterize the water present in different ways, such water content and degree of saturation, for use with other variables and relationships is a means for predicting stress behavior for settlement and other deformation important to engineering works (Johnson and DeGraff 1988; Gonzalez de Vallejo and Ferrer 2011).

Summary

Today, we move through a landscape past large earthmovers creating new highways, dodge trucks loaded with gravel and soil bound for construction sites, and relax next to lakes created by earth-filled dams. Yet, the idea that soils are being used to construct the underpinning of our modern society is unlikely to cross our minds. It is safe to say our modern society is as dependent on soil as most early civilizations. The biggest difference is our understanding of how to effectively use soils for far more sophisticated structures than those of Egypt and Mesopotamia. The biggest similarities remain our exploitation of soil's three most important properties: (1) abundance, (2) widespread occurrence, and (3) easy workability. Thanks to our modern understanding of soils for engineering applications, there is a significant body of scientific principles and concepts to guide the matching of soils to intended uses that was unavailable to earlier users. A quick examination of the Unified Soil Classification designations illustrates how salient soil characteristics based on simple concepts of particle gradation and the character of certain particles facilitate matching engineering design and intended use to the existing soil character. For the engineering geologist, it is a beneficial partnership working with the natural landscape to determine, describe, and characterize soils for those who are working for the best technological and economically engineered structures for our modern society to enjoy – even if they do not fully appreciate the importance of such a common earth material.

Cross-References

▶ Angle of Internal Friction
▶ Atterberg Limits
▶ Bearing Capacity
▶ Boulders
▶ California Bearing Ratio
▶ Characterization of Soils
▶ Clay
▶ Cohesive Soils
▶ Consolidation
▶ Effective Stress
▶ Expansive Soils
▶ Gradation/Grading
▶ Liquefaction
▶ Liquid Limit
▶ Modulus of Deformation
▶ Modulus of Elasticity
▶ Mohr-Coulomb Failure Envelope
▶ Organic Soils and Peats
▶ Plastic Limit
▶ Plasticity Index
▶ Poisson's Ratio
▶ Pore Pressure
▶ Quick Clay
▶ Quicksand
▶ Sand
▶ Shear Modulus
▶ Shear Strength
▶ Silt
▶ Soil Field Tests
▶ Soil Laboratory Tests
▶ Strength
▶ Young's Modulus

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Stabilization

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Definition

1. A general term used in engineering geology to describe a variety of methods and materials to improve the performance of in-place Earth materials and Earth structures and applies to soil materials and slopes, as well as rock slopes.
2. The term also has been applied to sites that have soil deposits that, in an untreated condition, would result in poor performance of buildings or other structures founded on them; the term "ground improvement" tends to be used to refer to stabilization of soil deposits at the scale of a site.

Stabilization of pavement subgrade soils for roadways probably represents the largest geotechnical application simply because road networks are so extensive and must provide access to locations that require the roadways to be in or traverse areas of weak or poor-performing soils and other challenging geologic conditions. Stabilization methods and materials are also used for ground improvement at certain high-value sites, and with some fill slopes, cut slopes in soil materials, cut slopes in rock formations, and for erosion control on natural slopes, as well as fill and cut slopes.

Stabilization of subgrade soils for roadways can be accomplished with mechanical or chemical means (Afrin 2017). The primary mechanical means is compaction supplemented with drainage to prevent the subgrade soils from being saturated, if possible. Changing the grain size distribution of the initial subgrade soil by adding coarse sand and durable gravel enhances the performance of the soil, particularly when it is well compacted. Other mechanical means include geofabrics or geotextiles and geogrids that are placed on the surface of prepared subsoil and between layers of compacted subgrade soil. Chemical means of stabilization includes constituents such as lime, cement, fly ash, and others.

Stabilization of high-value sites is performed to allow the sites to be developed for productive use. The cost of stabilization may be justifiable because of site location; the stabilized site usually allows less expensive alternatives for foundation support of buildings, and also enhances the overall performance of the site for pavements and even drainage and landscaping. The primary geotechnical conditions that are treated with site-scale stabilization are liquefiable soils and collapsible or hydrocompactable soils. The traditional approach to stabilizing soils over large areas on sites is excavation and recompaction, which cannot be performed economically, particularly if groundwater is shallow. The methods for stabilizing sites include deep dynamic compaction (repeated dropping of a block of concrete on a regular grid pattern), wick drains installed on a regular grid pattern, large-diameter auger borings that are backfilled with crushed rock (stone columns) on a regular grid, and compaction grouting (low-mobility grouting) on a regular grid. The effects of deep dynamic compaction and compaction grouting typically are verified by a subsurface site investigation following the application to verify attainment of desired results.

Fill slopes can be stabilized by external retaining structures, such as concrete walls or gabion walls, or by internal means including geofabrics or geogrids (Fig. 1) and mechanically stabilized earth (MSE) embankments (Abramson et al. 2002). Simple, low-cost treatment for slopes are available (Saftner et al. 2017). Geofabric can be used to make a cellular product that resembles a honeycomb that is used to confine columns of soil for an effective stabilization. Certain environments with sufficient moisture can support vegetation to an extent that soil bioengineering stabilization methods can be successfully applied (Gray and Sotir 1996). Cut slopes in soil materials can be stabilized by external walls or by soil nails. Deep excavations in soils in densely urbanized areas typically use braced excavation or soldier piles (H piles) on 1.5–2 m spacing with treated timber or concrete lagging between the piles to make a continuous wall.

Cut slopes in rock masses can be stabilized by external walls or application of shotcrete. Rock masses also can be stabilized with internal means, such as rock bolts or tendons. It is possible for rock slopes that need stabilization to be treated with a combination of rock bolts and shotcrete.

Some slopes need stabilization of surficial soils to suppress erosion. In many cases, the condition on the slope itself does not pose a hazard for engineered works, such as roadways or buildings, but the erosion is producing sediment that reaches stream or river channels and degrades water quality for aquatic species. Bioengineering stabilization methods may be effective in these cases (Gray and Sotir 1996). Stabilization for erosion control may be possible with small-scale terraces on slopes that are not too steep. For slopes on which erosion gullies have formed, some specialized products may be effective, such as dry, fiber-reinforced, high-strength cement in a thin composite fabric sheet that can be unrolled, positioned, and placed by hand, and then shaped to exactly fit the form of the gully, anchored to the soil with light-duty screws, and hydrated with water from a hose on a water truck or water trailer. Commercially available products, such as Concrete Canvas, are 3 mm, 5 mm, or 13 mm thick before hydration. Once such a product cures, it becomes a durable and rugged concrete-like channel that is resistant to the hydraulic shear stresses of flowing water.

Stabilization, Fig. 1 Use of a geogrid to stabilize a cut slope, Essex, UK (Photograph by Dr B R Marker)

Cross-References

- ▶ Bearing Capacity
- ▶ Characterization of Soils
- ▶ Classification of Soils
- ▶ Dewatering
- ▶ Erosion
- ▶ Gabions
- ▶ Geotechnical Engineering
- ▶ Geotextiles
- ▶ Mass Movement
- ▶ Retaining Structures
- ▶ Rock Mechanics
- ▶ Shotcrete
- ▶ Soil Mechanics
- ▶ Soil Nails
- ▶ Tension Cracks

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Strain

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Definition

Strain is the response in a rock or soil mass produced by applied load or stress. Strain is a tensor quantity, as is stress (Eberhardt 2009). Stress acting on a rock or soil mass can cause the mass to experience a change in volume, a change in shape, or a change in both volume and shape. The change in length Δl of a solid, right, circular bar can be normalized to its initial length l_o to produce axial or linear strain ε (Holmes 2016).

$$\varepsilon = \frac{\Delta l}{l_o} \tag{1}$$

Similarly, a change in volume ΔV divided by the initial volume V_o is volumetric strain ε_V.

$$\varepsilon_V = \frac{\Delta V}{V_o} \tag{2}$$

Shear strain γ can be explained as a change in the position of one surface of a solid, right rectangular prism relative to a parallel surface Δl caused by shear stress τ applied to the first surface, without an accompanying volume change, divided by the thickness of the prism h_o (Pariseau 2012).

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$$\gamma = \frac{\Delta l}{h_o} \tag{3}$$

Cross-References

- ▶ Hooke's Law
- ▶ Stress

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Strength

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Definition

Strength is the ability of any geo-material to withstand applied forces or stresses up to the point of failure.

Overview

The definition of strength is very broad as applied in engineering geology. Defining the strength of a material in more detail requires defining what it is meant by failure. In some engineering geology applications, failure can be considered as the transition of material behavior from an elastic regime to a plastic regime. Under this definition of failure, the stress at which the material begins to yield is considered its strength. For some applications, failure is defined as essentially complete loss of structural integrity of the material, following a peak stress that is reached as determined in standardized laboratory tests. In such cases, the peak stress is taken as the strength of the material. Some applications define failure as a maximum tolerable deformation. In such cases, the stress associated with the material strain that corresponds to the maximum tolerable deformation of the system can be taken as the material strength.

The relevant measurement of material strength will depend on the type of loading and the stress regime induced. The stress regime at any given location and direction within the material can be expressed through the normal (compressive or tensile) and shear components of the stress tensor acting on a plane of interest. Different stress regimes will render one or more of these components as critical. Example situations where some of these components become critical, and the associated strength, are:

1. Tensile stresses. Critical in some slope toppling mechanisms, overhanging rock conditions, and for rock excavation. Mobilize the tensile strength of the materials
2. Compressive stresses. Critical in walls within underground excavations and foundation performance. Mobilize the compressive strength of materials
3. Shear stresses. Critical in many applications such as slope stability, slip along discontinuities, and foundation support. Mobilize the shear strength of the materials

Depending on the definition of failure for each particular application and the critical components of the induced stress tensor, as well as knowledge about the behavior of the geo-materials, different criteria can be used to define strength. These are commonly known as failure criteria and can be estimated through laboratory and field testing, empirical correlations with other material properties, or a combination of both. The most commonly used failure criteria are the Mohr-Coulomb (for soils, rock masses, and shear along discontinuities), Hoek-Brown (for rock masses and intact rock), and Barton-Bandis (for shear along discontinuities) (Rowe 2001; Yang et al. 2013).

Cross-References

- ▶ Barton-Bandis Criterion
- ▶ Failure Criteria
- ▶ Field Testing
- ▶ Hoek-Brown Criterion
- ▶ Mohr-Coulomb Failure Envelope
- ▶ Rock Field Tests
- ▶ Rock Laboratory Tests
- ▶ Rock Properties
- ▶ Shear Strength
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Properties

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Stress

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Synonyms

Pressure

Definition

Stress acting on a rock or soil mass can cause the mass to experience a change in volume, a change in shape, or a change in both volume and shape. Stress is a tensor quantity, having a magnitude and direction, like a vector quantity, but also requiring reference to a plane across which it acts (Eberhardt 2009). A rock or soil mass represented as a cube with dimensions approaching zero can be oriented such that its faces are parallel to Cartesian coordinate system axes x, y, and z. Considering the area of each face of the reference cube to approach zero demonstrates that stress is a point property. The stress field σ acting on the faces of the reference cube can be resolved into one component that is perpendicular or normal to each face of the cube (normal stresses σ_{xx}, σ_{yy}, and σ_{zz}) and two components that are in the plane of each face of the cube, parallel with its edges, and perpendicular to each other (shear stresses τ_{xy}, τ_{xz}, τ_{yx}, τ_{yz}, τ_{zx}, and τ_{zy}); the subscripts denote the axis perpendicular to the plane on which each component acts and the direction in which the component acts. Opposite sides of the cube have normal stress components that are of equal magnitude and opposite direction. Shear stresses on adjacent planes that act in perpendicular directions have the same magnitude ($\tau_{xy} = \tau_{yx}$, $\tau_{xz} = \tau_{zx}$, and $\tau_{yz} = \tau_{zy}$). Consequently, the stress tensor σ has nine components, three that are normal stresses and six that are shear stresses (Rock Mechanics for Engineers 2016).

$$\sigma = \begin{bmatrix} \sigma_{xx} & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_{yy} & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_{zz} \end{bmatrix} \tag{1}$$

Stress that causes a change in volume without a change in shape results from hydrostatic pressure, which is a scalar quantity because it acts equally in all directions and has the same units as stress (force per unit area). Liquid and gas cannot sustain or transmit shear stress; thus, hydrostatic stress is a scalar quantity referred to as pressure. The reference cube can be rotated to a position such that the shear stresses in the planes of the faces have zero magnitude, which coincides with the normal stresses being principal stresses. The stresses applied to a rock or soil mass include the hydrostatic stress and other stress produced by tectonic forces, external loads, and excavations that may remove earth materials which provide support for adjacent earth material (Pariseau 2012).

Cross-References

► Deviatoric Stress
► Geostatic Stress
► Pressure

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Subsidence

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Definition

Subsidence is any downward movement of the land surface caused by geological changes of the subsurface. The velocity of the land surface subsidence and the extent, depth, and shape of the subsidence depression formed at the surface vary according to the characteristics of the geological setting of the location and the geological processes causing the subsidence. The formation of subsidence depressions may be accompanied by horizontal movements.

Introduction

Subsidence can be caused by various factors that can classify the process in principle as either man-made or natural. Natural tectonic processes or isostatic movements in lithospheric plates or magmatic movements in the upper mantle cause long-term (of several to thousands of years) and very large-scale subsidence of thousands to millions of kilometers squared. But geologic processes in the Earth's crust, causing subsidence lasting up to tens of years, are the most relevant to engineering geology.

Small-scale natural subsidence occurs in environments of specific natural conditions. These can be often developed rapidly. Various sinkholes appear often suddenly without precursor signs at the surface, due to a sudden collapse of an overburden into a void present in shallow depths. They belong to a group of short-term subsidences together with such phenomena as a vertical slope deformation due to erosion or terrain sinking rapidly after an earthquake.

A relaxed rock environment or an underground void, tens to hundreds of meters in scale, is a common source of a subsidence. The development of subsidence in such cases ranges from days to tens of years. This is typical for anthropogenically caused subsidence, such as that due to underground mining, groundwater pumping in semiarid regions, extraction of oil and natural gas, or construction of tunnels. Subsidence can be caused also by an intense human activity at the surface, including surface mines, or by building a new dam causing hydro-isostatic changes in its surroundings. Human activities may also speed up a natural subsidence by influencing geologic processes.

Subsidence Due to Extraction of Natural Resources

Extraction of Liquid Natural Resources

The majority of man-made subsidence is caused by extraction of solid or fluid natural resources or by a decrease of amount of groundwater from the subsurface aquifer rocks due to water pumping. Natural events in resource deposits may also cause subsidence, especially in case of resources that are soluble in water (salt, gypsum, limestone, and others) that have properties similar to karst. In such cases, mining or water pumping activities accelerate subsidence processes and increase the total occurrence of such phenomena as sinkholes. For example, since 1900, only 50 sinkholes in Alabama limestone were of a natural origin, whereas in the same period, 4,000 sinkholes appeared due to groundwater pumping (Goudie 2000). Also, lowering of the water table during mining for abstraction, pumping, or minerals dissolution purposes can lead to instability of overlying materials and to subsequent settlement at the ground surface.

In the case of groundwater level changes, the charges and discharges in aquifers are a part of a hydrological cycle. Unless there are external factors such as human activities, the terrain deformations due to the pressure changes have a temporary character and show a seasonal pattern. Continuous deep groundwater pumping, however, reduces the size and number of the open pore spaces in the aquifer and thus causes permanent subsidence.

The expansion of urbanized areas especially in semiarid regions heavily increases water demand. This is the cause of subsidence in Mexico City increasing from less than 10 cm/year to more than 30 cm/year today, due to pumping from the aquifer that is around 50–500 m below ground level. Continuous subsidence of infrastructure in Turkish Konya region is steadily increasing with the decreasing level of groundwater in the whole closed basin at rates similar to Mexico City. The reason of the subsidence is a number of unregulated wells – two thirds of ~100,000 wells are illegal and have enough capacity to disrupt the ecological balance depending on the local natural water regime (Üstün et al. 2014).

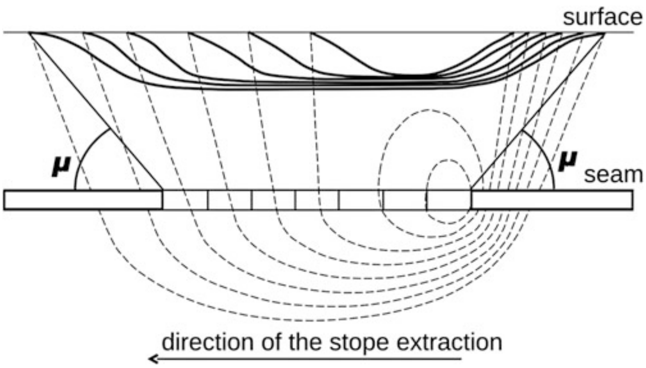
Similarly to groundwater pumping or extraction of any other fluid, the loss of pressure in desaturated material can cause distortions or failures leading to subsidence. The problem is often solved by injection of substitute fluids such as water into the ground. However, in the Wilmington Oil Field in California, an area of more than 75 km^2 subsided of up to 9 m due to oil extraction during 1927–1966, even though the subsidence was successfully ended by water injections in 1962 (Hawkins 2005).

Subsidence Caused by Mining of Solid Natural Resources

Subsidence due to extraction of natural resources such as coal or stone often changes the landscape character significantly over a relatively short period. For example, due to underground mining of a multi-seam deposit of black coal in the Czech region of the Upper Silesian basin, the subsidence of some (previously urbanized) areas reached up to 39 m during intensive mining periods over the last 60 years that included intensive mining periods (Jiránková 2010).

In the case of mines, subsidence depressions commonly take the form of subsidence troughs. Mining voids are very often left unfilled. After some period of time, which differs according to the geological settings and the mining technique used, the void roofs collapse, and the void migration can result in the land subsidence (Younger et al. 2012). The amount of subsidence s due to mining activities depends on the thickness of mined-out areas m, extent and depth of the extracted seam characterized as a coefficient of efficiency e, type of extraction technology that can be quantified in a coefficient of exploitation a (with respect to previous mining activities), and temporal dynamics of the geologic environment z, as $s = m \cdot e \cdot a \cdot z$ (Neset 1984).

Within the subsidence process, three stages are identified – during the first stage, around 5% of subsidence occurs. After the collapse of the roof, the main stage begins and encompasses around 80% of subsidence. The last stage is decay subsidence – this can last several years. In the previously mentioned example of the Czech black coal mines where the coal seams of few meters height are extracted at a



Subsidence, Fig. 1 Scheme of formation of a subsidence trough due to longwall mining

depth of more than 600 m, the main stage easily exceeds a subsidence rate of 0.5 m/month, and the decay subsidence lasts about 2–5 years.

The extent of the subsidence trough is estimated based on an average marginal angle of mining impact μ, calculated as depending to the heights and types of the overburden layers. The formation of a subsidence trough is demonstrated in the schematic example of a longwall mining in Fig. 1. The progress of seam extraction causes a subsidence wave developing on the surface, limited in extent according to the μ. Knowing the depth of the void H, the radius r of the subsidence depression can be estimated as $r = H \cdot \text{cotg}\ \mu$ (Neset 1984).

The behavior of subsidence depends on the geological setting of the layers that form the overburden of the subsurface void. The overburden can be considered as a layered inhomogeneous "girder" composed of variably thick and differently rigid layers that can be disturbed by a complicated system of tectonic fractures, especially in case of other voids at greater depths. Different deformations may occur depending on the different mechanical properties of the overlying rocks. Rigid layers have a large bearing capacity but a small capability of deflection. When the strength limit is exceeded, brittle deformation occurs. This happens also within layers that adapt elastically to changes of deposition and are capable of a large deflection and by those layers that can adapt to respond to changed conditions plastically (Jiránková 2010).

At the time of a breakthrough of the rigid roof of the void, brittle deformation occurs only in those layers having a small capability of deflection. Deformation of elastic layers at the time of the breakthrough is not brittle; brittle fracturing of these layers occurs only subsequently with the development of the void, for example, by the advance of mining. The variable dynamics within such a heterogeneous geological environment explains differences to the expected development of the subsidence trough, by causing subsidence delays, lower amounts of total subsidence, or a displacement of the subsidence trough center.

Specific Types of Subsidence

On the Earth, nature provides various and very specific environments. Modified by natural processes, including a currently increasing rate of climate change, and a higher demand for natural resources or other human activities in areas previously untouched, the original environments undergo significant changes, including subsidence.

Karst Subsidence

Karst landscapes formed on carbonate rocks (limestone, dolomite) are very susceptible to subsidence. Karst subsidence refers to landforms resulting from long-term destructive subsurface processes. In general, subsidence occurs when hydrodynamic destruction (suffosion, liquefaction, erosion, etc.) takes place in permeable loose sediments, whereas more coherent sediments or solid rocks support a void which can be subsequently destroyed by gravitational forces. The resulting sinkholes can be isolated but often are spread to wider areas to form dolines.

Thermokarst Subsidence

In permafrost areas, ground subsidence is associated with development of thermokarst terrain, that is, irregular, hummocky terrain produced by the melting of ground ice especially where ground ice is abundant within unconsolidated sediments (French 2007). The development of thermokarst is primarily due to the disruption of the thermal equilibrium of the permafrost and an increase in the depth of the active layer. This can be caused, for example, by clearing of vegetation (for agriculture or constructional purposes), by heat from buildings on permafrost, or by an installation of oil, sewer, and water pipes into or on top of the active layer (Goudie 2000). Typical thermokarst landscapes occur in central and northern Siberia and in western parts of the Arctic region of North America.

Thermokarst subsidence is associated with a loss of water upon thawing and its removal by either evaporation or drainage. Together with the process of thermo-erosion, subsidence has a considerable importance for thermokarst development.

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Subsidence Due to Water Infiltration or Reduction

Saturated loess deposits worldwide (covering about 5% of Earth's surface) tend to collapse during heavy loading at high moisture levels. The resulting subsidence entails extensive settling and cracking of the soil along ditches.

Soils susceptible to hydrocompaction are generally geologically immature with high void ratios and low densities. The amount of certain clay minerals that are present in soil affects its capacity for shrink-swell due to the water content changes. Variations of ground moisture are affected by weather conditions, the presence of vegetation, and a man-made activity (drainage). The ability of soil to change its volume results in damage to various structures and roads on the ground surface. Expansive soils are especially problematic in regions with a (annual) cycle of wet and dry periods as such arid and semiarid regions over the world are causing a periodic subsidence and uplift of structures every year.

Subsidence of Organic Soils ("Peat" Subsidence)

Peat soils are composed of organic material and water. When the water is removed (e.g., by drainage), peat oxidizes in contact with air and reduces in volume producing consequent subsidence. Subsidence rates vary substantially around the world and depend mainly on drainage and on climate. For example, in the Fenlands of England, approximately 3.8 m of total subsidence occurred between 1848 and 1957, with the fastest rate occurring soon after drainage had been initiated (Goudie 2000). Today's expected rate of subsidence averages approximately 1 cm per year, for instance, in large areas in the Netherlands. Recently in Southeastern Asia, large areas of peat swamp forest have been reduced through deforestation, drainage, and conversion to agricultural lands and other activities. Local recorded subsidence of peat soils reaches up to tens of centimeters per year (Hooijer et al. 2012).

Subsidence Associated with Earthquakes

One of the causes of an earthquake is the sliding movement of two blocks of the Earth's crust against each other. A movement of blocks on a so-called normal or thrust fault involves a vertical component, resulting in raising or sinking of the ground (subsidence).

Another effect of an earthquake resulting in a subsidence can be a soil liquefaction. Earthquake vibrations cause the loss of strength or stiffness of the soil, with increased effect on specific sediments, such as sand and clays. The weight of the overlying sediment (or structures and buildings) causes the settlement of sediments causing the ground surface to subside.

Large earthquakes can also provoke unrest in volcanic areas hundreds of kilometers away from their epicenter. This can also result in ground deformations (including subsidence), thermal anomalies, additional earthquakes, hydrological changes, or eruptions in volcanic regions. A coseismic volcanic subsidence was observed using GPS and InSAR techniques at some Japanese volcanoes following the 2011 Tohoku earthquake and at some Chilean volcanoes induced by the 2010 Maule earthquake (Pritchard et al. 2013; Takada and Fukushima 2013). The authors suggest that the subsidence is a response to stress changes associated with the earthquake along with the surrounding, thermally weakened host rocks in the first case, and to a coseismic release of hydrothermal fluids from beneath the volcanoes in the latter case. The volcanic regions subsided by up to 15 cm, forming elliptical depressions with horizontal dimensions of up to 20 km.

Other Causes of Subsidence Induced by Man-Made Activity

The load of large masses of water impounded in reservoirs can result in subsidence (sometimes accompanied by earthquakes). The process where a mass of water causes a coastal depression is called hydro-isostasy. This type of subsidence can reach few tens of centimeters in large reservoirs. Hydro-isostatic subsidence was detected, among other locations, in Lake Mead in the USA, Koyna in India, Kariba in Zambia/Zimbabwe, and Bratsk and Krasnoyarsk in Russia (Goudie 2000).

Various man-made sources of vibration can produce subsidence by compaction due to settling of underlying Earth materials especially in big cities or along highways (Demek 1984).

In Nevada, subsidence craters were created as a consequence of collapse of the cavity roof during underground nuclear explosions (Demek 1984).

Methods for Monitoring and Modeling Subsidence

Spatio-temporal evolution of subsidence is modeled by convenient techniques. These usually take basic characteristics of the area of interest into account as model parameters, at least the geological configuration of the subsiding location, and size and depth of the underground void causing subsidence on the surface. These characteristics are determined using various geological or geophysical techniques. However, the knowledge of the parameters is often limited, or the parameters are simplified for use in a model. A proper monitoring of subsidence using various geodetic surveying or modern remote-sensing techniques is important to verify the subsidence model and to detect deviations from the expected behavior.

Periodic measurements by geometric leveling or using GNSS receivers offer very high accuracy measurement at specific points. Precise leveling instruments can measure within a standard deviation of better than 1.5 mm/km. Stations combining theodolite and electronic range-finder measurements can be used for trigonometric height measurements. These are used in areas with relatively large-scale subsidence and steep terrain since the measurements of points are made from a distance, however, with lower accuracy than more direct measurements.

Modern remote-sensing techniques are valuable unique tools offering new possibilities to precisely evaluate subsidence trough development by evaluating movement of a large number of points in the area of interest. The area is observed remotely from different platforms – from (elevated) ground stations, aircraft (including UAVs), or satellites. Photogrammetric, LiDAR, and InSAR analyses provide precise spatiotemporal measurements of large areas, but they focus on specific issues that leave their application often experimental and site specific.

Summary

Subsidence is a phenomenon occurring widely on the Earth's surface, from a variety of causes and with variable rates and magnitudes, not restricted in rate or magnitude. The presence of subsidence causes complications to the human activities and needs of a stable living environment. The variability of natural and artificial causes and subsurface environments often result in unexpected subsidence behavior. Modern technologies offer possibilities for regular and repeated observations of subsidence in any range, supporting studies of engineering geologists.

Cross-References

▶ Aquifer
▶ Clay
▶ Collapsible Soils
▶ Earthquake
▶ Hydrocompaction
▶ InSAR
▶ Karst
▶ LiDAR
▶ Liquefaction
▶ Mining
▶ Sinkholes
▶ Surveying

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Subsurface Exploration

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Synonyms

Ground investigation; Ground reconnaissance; Reconnaissance engineering; Reconnaissance investigation; Site investigation; Site reconnaissance; Soil investigation; Soil reconnaissance; Subsoil exploration; Subsurface investigation; Subsurface reconnaissance

Definition

Activities onshore, nearshore, or offshore, planned by a specialist to obtain a geological, geotechnical, or geoenvironmental model of the ground, as close as possible to reality, to enable an environmental or engineering (civil or mining) project to be undertaken with inherent economy and safety.

Introduction

Engineering geology is founded on the definition of the geological model of the ground and it's engineering properties. This is achieved through desk studies and field reconnaissance prior to comprehensive investigations and testing of the ground, to the required depths for a programmed intervention, providing engineering geological information essential for the suitable design of structures, infrastructures, ground improvement, resource exploitation, or soil rehabilitation.

Therefore subsurface exploration is the cornerstone of the Engineering Geologist's activity. The specialist has a set of methodologies, investigation techniques, and field tests that enable him/her to define ground conditions (rock mass or soils). To get it right is a function of his/her professional experience, available techniques at the site, geological or engineering complexity, of the ground or of the intervention respectively, and also of the amount of time and money available.

The financial costs of all these engineering geological activities are almost negligible when compared with the cost of the engineering intervention to be undertaken – usually 1–2%. But costs can grow significantly if subsurface investigations are shortened during design stages, postponed (sometimes *sine die*) by the Contractors, or the homogeneity of the ground is optimistically presumed but not verified. The consequences always affect the safety of the intervention, which depends largely on reliable information for establishing the nature, time, and costs of the construction, exploitation, or rehabilitation works.

Other concerns about adequate planning and the acquisition of reliable information of the ground are a consequence of globalization – sometimes the specialist that plans the subsurface exploration program is in another part of the world and unable to accompany, in real time, the development and the results of the on-going investigation program in a remote area. Therefore, they are unable to swiftly review and update their initial programming. Another problem is keeping up to date with the rapid evolution of some types of engineering structures, such as those for producing renewable energy at, or from, the sea.

Knowledge and understanding of the influences that any structures impose on the ground is essential for adequate programming of the exploration methods. One needs to know what to look for to study it properly. To achieve sustainable development, the Engineering Geologist must be able to encompass efficiently the constant improvements in electronics and telematics; both in equipment and the interpretation of the results. Finally, the Engineering Geologist must deal with the requirements of studying complex sites that must be developed such as brownfield sites, areas of complex geology, and all sites in highly previously developed regions.

But, if data from direct assessment in the exploration program are georeferenced and summarized in an adequate format and according to the technical standards or recommendations published and recognized around the World, there is a resource of valuable information that can be available, if suitably stored, for any future development or interventions that might be needed. Thus, an adequately programmed subsurface intervention can also provide future benefits.

Programming Guidelines

Inadequate engineering results are not an outcome of a lack of *in situ* exploration methods or tests, but, rather, of poorly conducted subsurface characterization because of lack of proper planning of the reconnaissance programs, although sometimes because of imposed financial constraints. Of course, occasionally, the direct information acquired may be incorrectly interpreted, leading to inadequate engineering solutions.

Oliveira (1992) supports the view that subsurface reconnaissance works should be conducted in stages, considering the potential to multipurpose uses of components, progressively using more sophisticated and costly methods and techniques, according to the design stage and its requirements. These studies and exploration works should be conducted to acquire a significant number of results for analysis of each property and should be distributed throughout a representative volume of ground, thus allowing statistical analysis of these results and the definition of characteristic values for relevant parameters. This will lead to optimization of cost and time. Finally, the combined interpretation of all the geological, hydrogeological, geoenvironmental, or geotechnical parameters should be used to zone the ground.

One can identify three major steps: preliminary studies, to develop a conceptual model of the ground; followed by planning and development of the exploration program that may encompass several stages, especially in respect of engineering design studies; and finally, a controlled investigation, during the implementation of the project, to study problems that emerge during construction or remediation works.

It is important to emphasize the role of desk studies, including information from geo-databases and old topographical maps in urban areas, which are fundamental for understanding the ground morphology and hydrology and also the distribution of utility services. Photointerpretation of remote sensing images of the target area is also a key to understand the structure of the ground. This is essential information for adequate design and location of the exploration methods and for identification of accesses to remote sites – Fig. 1.

It is during this initial study stage that data, such as the previous use of the ground (mining, chemical industries, etc.), the main geological constraints (the presence of gases such as radon, methane or carbon dioxide, susceptible ground to volumetric changes, squeezing, compressible or collapsible soils, high water table and karst morphology) and hazards (landslides, mining galleries/shafts, subsidence, running sands, etc.) that may occur at site, can be identified. The desk study allows a first conceptual model of the site to be sketched.

After assessing the type of surveying techniques that should be mobilized, the next stage is selection of appropriate non-intrusive/intrusive methods of surveying according to the geographical location of the site (rural or urban area, flat or mountainous area, etc.) and the ground features. A cost/benefit evaluation should be made before an investigation plan can be implemented. The type and location of methods to be used depend on several factors such as types of geological structures and conditions. The amount and location of

Subsurface Exploration, Fig. 1 Drilling in a remote Central Africa mountain

nonintrusive and intrusive methods should be planned to reflect the types and levels of the required information on, for example, adverse discontinuities in the ground such as important faults and shear zones, weathered dykes, and cavities. These should be subjected to specific geotechnical investigations.

Geophysical Methods

Ground investigation usually begins with the use of nonintrusive methods, usually ground surface geophysics, or less commonly airborne methods, allowing a general insight to subsurface conditions. Geophysical exploration aims to gather information about the subsurface over a large area at an acceptable cost. The resulting information helps to exclude unsuitable sites, but the most important purpose is to better select the positions for boreholes and to reveal variations in the geological conditions between them. Therefore, in the case of uniform ground it prevents unnecessary drilling.

Typically, the main geophysical methods used by Engineering Geologists are electrical resistivity (horizontal and vertical profiling, dipole-dipole) and seismic (refraction and reflection) methods. Also for special circumstances like dissolution or mining cavity detection, studies of water flow/contaminant plumes or identification of buried metallic objects, other techniques (electromagnetic, ground penetration radar (GPR), or microgravity) may be used successfully. The principle of geophysical methods is that the transmission and reflection of magnetic, gravity, seismic, and electric fields are affected by the physical properties of the investigated media such as density, elasticity, electrical conductivity, magnetic susceptibility, and gravitational attraction, or forces in the subsurface.

Some of these methods may be difficult to apply in urban areas where the background vibrations (for seismic techniques) or airborne or buried utilities (for electrical resistivity and electromagnetic techniques) may interfere severely with responses. The main attraction of all of these methods is that they cover a large area/depth quickly and, if appropriately selected and, especially if used in combination, they can give a fairly accurate model of the subsurface, thus allowing a potential saving in subsequent intrusive investigations undertaken to validate the geophysical findings.

Table 1 summarizes the main features relevant for selecting the most current geophysical methods (McDowell et al. 2002).

Another type of non-invasive technique that may be used in preliminary reconnaissance of sites is LiDAR (light detection and ranging), a remote terrestrial or airborne data acquisition technique that can be associated with ortho-corrected images. This technique is especially suitable for study of remote areas or sites having a thick vegetation cover, allowing the specialist to have clearer understanding of the ground surface morphology and structural features and providing a 3D high-resolution digital elevation model of the area. It has been particularly used to map landslides in difficult terrains and in mining to allow study of the geological and geotechnical conditions and contributing to more rapid implementation of mitigation measures.

Subsurface Exploration, Table 1 Characteristics of main geophysical methods

Method	Major applications	Limitations
Seismic refraction	Weathering profile; depth of overburden Rock mass rip ability Depth to groundwater *In situ* dynamic parameters of the ground (E, G, ν)	Requires a constant increasing of compressional wave velocities with depth in the area to be studied Thin strata are sometimes undetectable
Seismic reflection and profiling	Structure of ground and thickness of different acoustic formations Study of the underwater subsurface Bedrock profile	Not useful for in situ determination of dynamic parameters of the ground
Electric resistivity	Variation within the drift/man-made ground cover Relative position of the strata and estimation of their thickness Moisture content variability with depth; soft soils thickness Anomalies like faults, cavities, soil lenses, fluids flow, etc.	Depth determination may be erroneous
Electromagnetic	Detection of buried ferrous objects Water bearing formations/contaminant plumes Anomalies like faults, cavities, soil lenses, fluids flow, etc.	Influenced by some airborne or buried utilities
GPR	Detection of buried ferrous objects Near surface groundwater contours and presence of natural or man-made voids Utilities services detection	Presence of high water table will limit drastically the penetration depth
Microgravity	Natural or man-made voids Variation within the drift/man-made ground cover Information about the geological structures for engineers	Data reduction and interpretation are complex (interference by topography, strata density and presence of man-made structures) and require calibration on site
Magnetic	Location of buried ferrous objects Geological structures, like anticlines, synclines, etc. Location of abandoned mine shafts and other similar structures	Data interpretation also highly specialized and requires calibration on site; influenced by some airborne or buried utilities or the presence of metallic objects – trains, cars, etc.

Exploratory Excavation

The use of direct, or intrusive, techniques usually starts by digging trenches or trial pits (Vallejo and Ferrer 2011), or boring using light augers – Fig. 2, depending the length/area/depth to be studied and the purpose of the study. The first two are the main techniques for direct observation and sampling of the top layers of the ground – Fig. 3, and for sampling large amounts of soils, for laboratory tests for instance when preparing relevant earthworks.

Trenches are linear investigations and their direction can be changed easily if necessary. Pits are excavations whose section is relatively small compared to the whole area of the investigation study, so they are dug to solve localized issues during the preliminary study phase. The dimension usually depends on the method of excavation and the planned tests, but usually are 3–4 m deep, although some shafts, to study specific issues like a concrete dam abutment, may be sunk for a dozen meters.

Significant civil engineering works may also require the excavation of galleries during site investigation as in pilot galleries or adits for large concrete dams or tunnels, or if *in situ* tests need to be executed at important depths or in deep zones. But these are very expensive methods and, therefore, they are only used in those types of works where the benefits they bring to the design/construction are basic to the safety

Subsurface Exploration, Fig. 2 A trench dug for an earth dam axis study

Subsurface Exploration, Fig. 3 Performing a light auger

Subsurface Exploration, Table 2 Main characteristics of exploratory excavation methods

Method	Major applications	Limitations
Trenches	Help geological mapping, namely of formation contacts and structure and to give a prior assessment of the near subsurface To complete and help the interpretation of the results from the geophysical surveys Paleoseismicity studies To provide advice on further investigation	Intersection of groundwater/hard layers dictates the maximum excavation depth Safety rules for excavations must be strictly implemented
Pits/ shafts	Vertical investigation of the lithology, stratigraphy, jointing, weathering profile Core samples may be obtained and *in situ* tests can be performed on them	
Galleries/ adits	Geological structure and weathering of the rock mass Large volume in situ tests can be performed inside them	Expensive and sometimes require support, namely at the portal

Subsurface Exploration, Fig. 4 Close up of percussive drilling tools: star bit and bailer

and economy of the civil engineering work. The length of the galleries depends on the local rock mass properties and the expected ground volume to be affected by the stresses induced by the future structure.

Usually, galleries and boreholes are not used during the reconnaissance phase of the investigation, since they are expensive and take a long time. The more complex the structure is, the more detailed an investigation program (Table 2).

Drilling Methods

Boreholes are small diameter drill holes allowing continuous assessment of the nature and location of ground layers even to great depths. Typically, the cost of boreholes depends on the findings of other previously used investigation methods. The borehole spacing depends on the investigation phase and the type of structure and geological complexity. Normally, a large grid spacing should be selected initially and, afterwards according to the information gathered, a more detailed geotechnical investigation gives an opportunity to sink more boreholes wherever it proves necessary. Note that economic circumstances and also the lack of time can limit the number of boreholes that can be sunk and can reduce the borehole program to a single stage.

There are several types of drilling techniques, which determine the quality and the quantity of the gained information. The boreholes can be sunk by percussion (gravel, cemented materials) or rotary (hard soils and rocks). In general, they allow the extraction and identification of cuttings or cores of the ground, may secure samples of high quality (samplers), and allow the carrying out of *in situ* tests. In general, boreholes have diameters between 75 and 150 mm.

Percussion drilling – Figure 4 is used for investigating soils and soft clay at shallow depths, usually down to 40 m. Percussion borings are not economical in strong rock masses but are particularly suitable for heterogeneous soils having gravel/boulder/shelly strata.

In the case of relatively weak soils, with no interstratified hard strata or cobbles, solid and hollow stem augers, Fig. 5 may be used in either stable self-supporting strata above the water table or unstable formations. Hollow stem augers,

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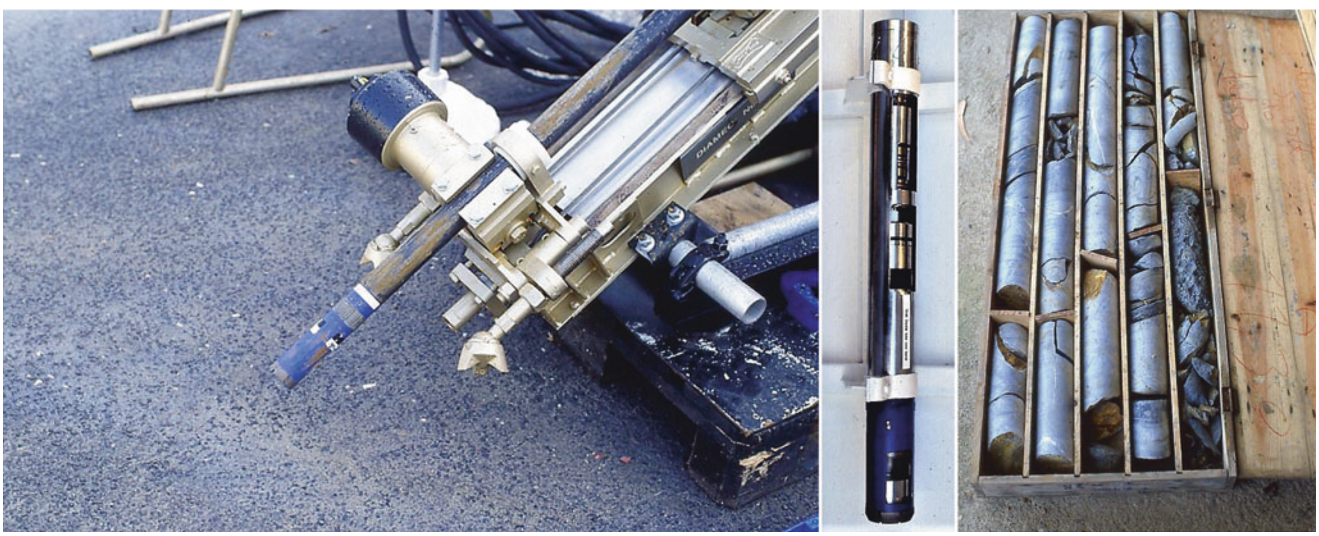
Subsurface Exploration, Fig. 5 Hollow stem auger boring underway and close up of the tip



Subsurface Exploration, Fig. 6 Rotary core drilling: assembly, double tube core barrel cut and cores

particularly used in loose sand and clay below the water table, typically have a diameter ranging from 165 mm (inner hole of 83 mm) and 258 mm (inner hole of 168), allowing undisturbed sampling and testing of soils through the hollow stem and for environmental studies.

Boreholes in soils should not use water to assist drilling, because of flushing away of materials, except in the case of dry granular soils. Whenever a borehole penetrates beneath groundwater and disturbance of the formation is likely, a positive hydraulic pressure head should be maintained in the hole.

Rotary drilling with core recovery – Figure 6 is necessary to study a rock mass or hard soils, up to hundreds of meters deep, by using a double tube core barrel or triple tube core sampler, respectively. Nevertheless, this can also be used to investigate urban solid waste deposits. This method normally uses circulating water to cool the core drilling bit but in solid waste landfills this is not generally advisable, because it will produce leachates. Core recovery in this type of situation should be by dry rotary drilling and the core runs should be smaller – some 0.5 m, maximum. In all the others cases, the core runs should be about 1.0 m maximum for the first stage, and afterwards, the core runs should be increased according to the core recovery, which ideally should be at least 90% in a moderately weathered/fractured rock mass and be as close to 100% as possible, because the core is the prime means for the specialist to assess the rock mass conditions at depth. The minimum diameter of the cores for laboratory tests should be 54 mm (NX boreholes). A core with a large diameter provides a greater amount of valuable information, because small discontinuities can be detected but this is more expensive.

Another type of drilling which recovers high-quality continuous core samples in softer materials (unconsolidated soils, fills, gravels/boulders and porous, weak or weathered rocks) is sonic drilling, utilizing high frequency (150 Hz) resonant energy. As yet, this relatively new technique is only available in a few countries around the world. It can drill efficiently to about 180 m depth and uses a double core barrel assembly with an external diameter ranging between 76 and 305 mm, ensuring 100% recovery in any type of geologic formation with minimal or no fluid usage. Although, it has a rapid advance rate, it is still a very expensive technique, but in special circumstances, sonic drilling may be the best solution to ensure adequate investigation for a given problem.

Rotary drilling without core recovery, by rotary open hole or rotary percussive drilling, is not usually used in engineering geology site investigations by Engineering Geologists. While they are cheaper than standard coring, these are destructive methods and no core samples can be recovered. Exceptions occur when it is necessary to advance a hole to locate some specific feature in the ground such as the presence of a cavity (natural or man-made), the location of some buried structure, or as an auxiliary borehole to do some cross testing. Thus, to obtain some other benefit from this type of drilling, one must consider rate of penetration, loss of flushing medium, etc. to get additional information regarding the deposits.

Finally, mechanical and water absorption tests and geophysical and nuclear logging can be performed within the boreholes. In soils, some sampling, usually undisturbed, can also be made.

The study of groundwater should always be considered during site investigations both by adequate study of its oscillations during drilling and by considering the need to install some type of piezometer to evaluate the water table behavior with time and during construction phase. Thus one can develop a water monitoring plan at a low cost.

After completion of any type of intrusive method, it is necessary to backfill the void/hole to minimize subsequent depression at the ground surface due to settlement of the fill. In places located near environmental sensitive features or in contaminated soils, extra care is necessary not only during drilling, when water from dissimilar layers must be kept out of contact but also at the end of drilling, where particular attention should be given to grouting of the hole Table 3.

Subsurface Exploration, Table 3 Main features associated with drilling techniques in the scope of civil engineering works

Borehole	Ground suitability	Sampling	Limitations
Percussion	Heterogeneous soils and weak rocks	Cuttings	High resistance rocks layers limit significantly the progression depth; drilling equipment is heavy; poor quality of the cuttings below groundwater level
Auger	Relatively soft soils, above (solid stem) or below (hollow stem) water table		The presence of cobbles and boulders as well as hard strata limit the progression depth; problems of stability in uncemented soils (solid auger); poor quality of the cuttings
Rotary	Medium to hard soils and rocks		Poor quality of the cuttings
Rotary core	Hard soils and rocks	Core	Most expensive of all drilling methods
Sonic	Soft materials, like unconsolidated soils, fills, gravels/boulders, and porous, weak, or weathered rocks		

Sampling

Since the main drilling techniques in soils, percussion and auger methods, do not ensure representative samples, these boreholes are complemented with regular sampling, at least each 2.0 m runs, using specific samplers designed to reduce potential soil disturbance during the process. Table 4 summarizes the main types of samplers and the types of soils where each of them performs best. Sampler selection depends on the types of laboratory tests to be performed: mechanical or hydraulic tests require undisturbed samples, or identification tests, on deformed samples.

In situ testing It may sometimes be difficult to determine engineering properties from sampling and laboratory techniques because of lack of sample representativeness or because it is too expensive or time consuming. The Engineering Geologist must then rely on in situ tests, to complement

Subsurface Exploration, Table 4 Main types of soil samplers and their applications

Sampler	Ground suitability	Quality of sample	Limitations
Split spoon	Relatively soft soils without coarse gravel or pebbles	Probably slightly disturbed	Below groundwater, fine particles are washed out
Thin wall (Shelby)	Soft to medium clays/ silts and medium to loose sands with fines	Undisturbed	Presence of coarse particles
Piston	Soft to very soft soils, specially below groundwater		
Rotary barrel	Hard soils and weak rocks		Unbound materials
Pitcher barrel	Soils in general, soft to hard		Very soft soils
Swedish	Poorly cemented loose soils		Expensive

Subsurface Exploration, Table 5 Main hydraulic and mechanical in situ tests

Ground	Test	Location	Parameter
Soil	Infiltration/Lefranc	Borehole, pit (quick test)	Soil permeability
	SPT (standard penetration test)	Borehole	Soil strength
	CPT (static penetrometer), CPTu (piezocone)	Self-drilling	
	Dynamic penetrometer (super heavy, heavy, medium, or light)		
	Vane test (VST)	Borehole/ self-drilling	
	Pressiometer (PMT)		Deformability
Rock mass	Dilatometer (BHD)	Borehole	
	Jack/plate test	Pit, gallery	
	Large flat jack – LFJ	Gallery	
	Small flat jack – SFJ		State of stress
	Hydraulic fracturing/ packer impression test	Borehole	
	Stress tensor tube (STT), CSIRO	Borehole	
	Shear strength of discontinuities or rock pillars	Gallery	*In situ* strength
	Water absorption/packer	Borehole	Equivalent permeability

the site investigation plan. These tests can be used to define: stratigraphy and the location/thickness of specific layers; the presence/absence of certain contaminants/fluids; the quantification of the permeability of the formations; or deformability/ compressibility, strength profiling, etc. Table 5 summarizes the main tests used in soils or rocks. The most costly and time consuming *in situ* tests, embracing larger volumes of the rock mass, and conducted in small numbers (e.g., large flat jacks for deformability tests and state of stress tests), should only be located in the most critical zones of the rock mass in order to obtain more precise values for the geotechnical parameters to be used in final stability analysis of the structures.

It is important to emphasize the need for a careful planning of tests, including the suitable standards to be observed and, afterwards, the necessity for a cautious interpretation of results by an experienced specialist are relevant for the success of the investigation and the adequate solving of site problems.

Some logging techniques can also be used inside boreholes. In the case of acoustic logging, seismic wave velocity profiles can be made between two nearby boreholes or, instead, geophones can be placed on the bottoms of trenches or galleries and used in conjunction with a nearby borehole to locate the sources of seismic wave's source. These approaches can be used for site-specific investigation and to gain more information for the soil or rock characterization, to determine engineering properties of materials and to study deformations.

Reporting

A site investigation is only concluded when a report, including the geological and structural setting, the description and discussion of the exploratory methods adopted for the study, their results and inherent discussion of their implications for the problem to be solved, is completed. This report must be accompanied by the logs of all borings, trenches, and other site investigations, as well as any drawings necessary for specialists and nonspecialists comprehend the model and/or design to be implemented. Any recommendation for the next stage of design must be appropriately pointed out, whether it is for a complementary site investigation study or for a monitoring plan.

Summary and Conclusion

The safety of most large engineering works depends, to a significant extent, on ground conditions that interact with constructions. Engineering Geology is the science which may supply the required geological and geotechnical information to characterization of the sound design of the project.

Desk study, field reconnaissance, site investigations, and testing are the tools which, when properly used, contribute to the understanding of the ground where those structures are to be assigned. Nevertheless, the studies and investigations and testing which provide the geological and geotechnical information for a safe design do not exclude the possible need for follow-up during construction to adjust the final design to the real conditions of the ground and to monitor the behavior of the relevant structural components. It is important to understand that even the most complete site investigation program may not be able to detect all the significant geological and geotechnical features of a site, since nature is seldom homogeneous and isotropic.

Cross-References

- ▶ Aerial Photography
- ▶ Aeromagnetic Survey
- ▶ Boreholes
- ▶ Borehole Investigations
- ▶ Brownfield Sites
- ▶ Characterization of Soils
- ▶ Cone Penetrometer
- ▶ Designing Site Investigations
- ▶ Drilling
- ▶ Engineering Geomorphological Mapping
- ▶ Engineering Properties
- ▶ Excavation
- ▶ Geophysical Methods
- ▶ GIS
- ▶ Groundwater
- ▶ Instrumentation
- ▶ Jacking Test
- ▶ Karst
- ▶ Landslide
- ▶ Mass Movement
- ▶ Piezometer
- ▶ Remote Sensing
- ▶ Risk Assessment
- ▶ Rock Field Tests
- ▶ Rock Laboratory Tests
- ▶ Shear Strength
- ▶ Soil Field Tests
- ▶ Soil Laboratory Tests
- ▶ Soil Properties
- ▶ Waste Management

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Surface Rupture

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Synonyms

Surface faulting

Definition

Surface rupture occurs when displacement on any type of subsurface fault plane propagates upward and displaces the ground surface. Resulting surface displacement (whether rapid or slow) can cause damage to infrastructure and thus is a legitimate concern to engineering geologists. The most common type of surface rupture is created during moderate- to large-magnitude shallow earthquakes (hypocentral depth less than 20 km), when slip on the coseismic fault plane (principal fault) propagates up to the ground surface and displaces it. The surface displacement occurs a few seconds after the start of surface ground shaking (Wilkinson et al. 2017) and generally exhibits the same sense of slip as the underlying fault. Coseismic surface faulting is common in earthquakes larger than about moment magnitude 6.0. With increasing earthquake magnitude, the length of the principal surface rupture zone and the amount of surface displacement increase. At higher magnitudes surface faulting away from the principal fault (distributed faulting) becomes common. Some distributed faults may have a different slip sense than the principal fault, especially if they have propagated upward through unconsolidated sediments. The zone of distributed faulting may range from tens to thousands of meters wide, with zone width and distributed displacement increasing with magnitude. Finally, surface rupture on even more distant faults (triggered faulting) may be induced by strong ground motion and/or Coulomb stress changes during the earthquake. In the late 1990s, efforts began to quantitatively predict the location and size of future coseismic surface ruptures (principal, distributed, and triggered), which resulted in the technique of probabilistic fault displacement hazard analysis (PFDHA) (e.g., Youngs et al. 2003). However, PFDHA is restricted to predicting probabilities and displacements on discrete coseismic surface faults. It cannot predict diffuse folding and bending caused by coseismic shear near the fault nor secondary faulting related to coseismic folding (such as flexural-slip or bending-moment faults).

Hanson et al. (2009) were the first to describe the complete spectrum of surface rupture, spanning all fault types that pose

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Surface Rupture, Fig. 1 Two-meter-high fault scarp (upper left to center) created by the Borah Peak, Idaho, USA, earthquake of October 28, 1983 (Mw6.9). Surface rupture here crossed a low-gradient, gravelly alluvial fan. Scarp was 36 h old when photo was taken. Gayle McCalpin (1.7 m tall) stands in a graben formed on the hanging wall, a typical feature of surface faulting involving normal faults

hazards to surface infrastructure. They defined tectonic faults to include both primary faults capable of producing earthquakes (discussed above) and secondary faults that are produced by earthquakes but are not themselves capable of generating an earthquake. They also defined non-tectonic faults to include those produced by gravitational processes (e.g., landslide features, sackungen), dissolution phenomena (e.g., karst collapse features), evaporite migration (e.g., salt domes and salt flowage structures), sediment compaction (e.g., growth faults, subsidence structures), and isostatic adjustments (e.g., glacial rebound structures). These faults can create surface rupture comparable to those of tectonic faults. Non-tectonic gravitational faults are mainly extensional (landslide headscarps (Gutierrez et al. 2010), sackungen, karst collapse, and subsidence) resulting in normal faults and graben. Less common are non-tectonic reverse faults (landslide toe thrusts, rebound/unloading scarps). Surface rupture hazards from such faults have typically been analyzed at the site scale in engineering geology, using many of the investigative techniques such as trenching developed for tectonic faults (e.g., McCalpin 2009). However, at present no quantitative, PFDHA-type analysis has been developed for these nontectonic surface ruptures (Fig. 1).

Cross-References

- ▶ Earthquake
- ▶ Faults
- ▶ Geohazards
- ▶ Hazard Assessment
- ▶ Probabilistic Hazard Assessment

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Surveying

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Definition

The science and technique of making accurate measurements of the Earth's surface.

Surveying includes data collection using appropriate instruments, data reduction to obtain useful information, and the establishment of location and size based on derived data. Surveying is generally classified based on the purpose of the resulting output or the method and instruments used for the survey. Some of the common surveying techniques based on the purpose relevant for engineering geology include topographic survey, cadastral survey, geodetic survey, engineering survey, and geological survey. Traditionally, surveying was performed using ground systems, which are now supplemented by aerial and satellite surveying methods.

Some of the early historic accounts of surveying are noted in Egypt. The geometrically accurate pyramids and the use of distant control points to replace property corners destroyed by the flooding of Nile River are examples of early Egyptian surveying. Followed by the Egyptians, the Greek and Romans surveyed their settlements (Hofmann-Wellenhof et al. 2012; Leick et al. 2015). Surveying can be classified based on the instrument or method employed as (a) chain survey, (b) theodolite survey, (c) traverse survey, (d) triangulation survey, (e) tacheometric survey, (f) plane table survey, and (g) photogrammetric survey. The chain survey is the simplest form of surveying and is suitable for small areas. Theodolites are precise instruments used for measuring horizontal and vertical angles, and its use for survey is referred as theodolite surveying. Traverse is a method used for surveying to establish control networks. Triangulation survey is a method that uses the measured angles and a known distance to compute the two unknown sides of a triangle in surveying. Tacheometric survey is carried out using a tacheometer that helps to make rapid angular measurements. Plane table survey is a graphical method of survey where the field observation and the data plotting are done at the same time. Photogrammetric survey uses photographs to obtain reliable spatial information.

Over the years, the instruments and methods used for surveying have seen great advancements. However, three critical improvements have significantly improved the speed of data collection and are noteworthy. The introduction of mapping from aerial photogrammetry in the early 1920s provided the opportunity to cover large areas, in the 1960s the electronic distance measurement and alignment as well as the adoption of lasers made rapid data collection possible, and the late twentieth century witnessed the satellite-based geodetic surveying and computerized data processing (Wright and Lyman 2015).

Cross-References

► Aerial Photography
► Aeromagnetic Survey
► Engineering Geological Maps
► Engineering Geomorphological Mapping
► InSAR
► Photogrammetry

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Tailings

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Definition

The waste material separated out during the processing of mineral ores (EPA 2000). Tailings are found as impoundments, piles, or other accumulations in proximity to mills or other processing sites. Processing sites may be near the location where ore is extracted or a short distance away from the mine location (Wills and Napier-Munn 2006).

Characteristics

Commonly, extracted ore is crushed until most of the ore is reduced to particles ranging from sand to silt size (Fig. 1). This process, sometimes referred to as comminution, creates a size range that facilitates separation of the mineral material from the ore by either a physical or chemical process (Wills and Napier-Munn 2006). Roasting, floatation, or centrifuge separation are common physical processes (Wills and Napier-Munn 2006). Chemical processes can involve creating amalgamations or precipitates to more easily separate the ore mineral from the less reactive host rock material. Chemical extraction of gold using mercury (amalgamation) or cyanide (precipitation from gold-cyanide solution) is an example.

Tailings often represent a significant part of the ore volume introduced into the extraction process (EPA 1994). Consequently, waste management is an important part of the mineral extraction effort (Wills and Napier-Munn 2006). Some physical and chemical processes produce tailings in the form of a wet slurry. Slime and sand are terms used to refer to tailing slurries consisting predominantly of silt-size and sand-side particles, respectively. The slurries expedite transporting this waste material out of the mineral extraction process (EPA 1994). However, slurries require discharge to impoundments to allow the material to dry. Such impoundments are referred to as tailings ponds. Tailings produced by processes where water is not needed are removed using conveyor systems, small rail carts, or Earth moving equipment. Tailings moved in this manner accumulate as piles near the processing facility.

Engineering geologists have a role at active ore processing facilities to determine geologic conditions important to designing foundation elements for the facility. They also assess slope stability for facility construction and maintenance including the transportation system supporting mill operations and contribute to safety inspections. For those processing facilities requiring tailings ponds, slope stability assessment is especially important for both the construction of impoundments and their ongoing operations (EPA 1994). Worldwide, there have been notable examples of human-caused disasters due to tailings pond instability (e.g., Luino and De Graff 2012). Both determinations of geologic factors important to foundation design and maintaining slope stability are roles engineering geologist fulfill during reclamation of inactive or abandoned processing facilities. This work can be complicated by tailings that contain hazardous materials due to chemicals used during the mineral extraction process, reactions of minerals in the tailings due to

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Tailings, Fig. 1 Tailings at a World War II-era chromite mine in the Central Coast Range near San Luis Obispo, California (USA). Tailings are visible to the *left* and *below* the flat area where the crushing operation occurred. Annually, eroded tailings are carried into the head of the nearby catchment

exposure to surface conditions, and the presence of residual ore minerals as can be the case with mercury and uranium tailings (DTSC 1998).

Cross-References

- ▶ Catchment
- ▶ Crushed Rock
- ▶ Mine Closure
- ▶ Mining

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Tension Cracks

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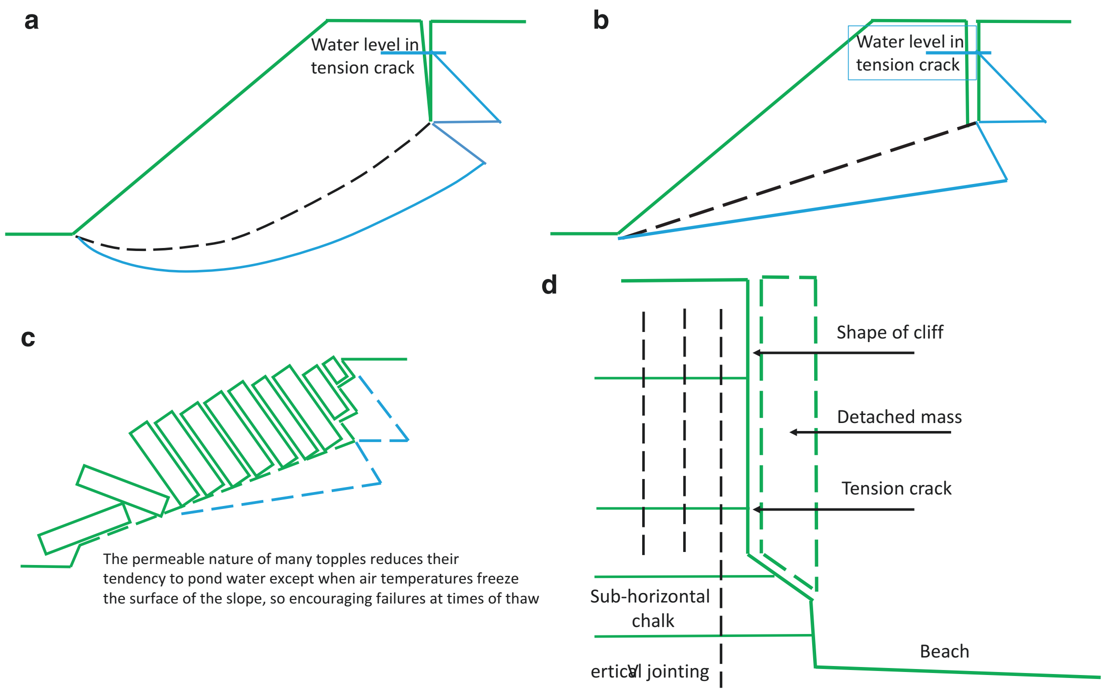
Definition

A vertical gap develops behind the crest of a slope in either soil or rock and forms the back face to the scar left by a landslide when complete failure of the slope occurs. Water within a tension crack can significantly contribute to slope failure.

The shear forces of concern in slopes are those directed out of the slope face because they will cause ground to move toward the slope and, when sufficient, will eventually cause the slope to fail.

Shear stresses are greatest in the region behind the toe of a slope. It is from this area that failure of the ground in shear commences, nucleating from discrete locations where shear stresses exceed the shear strength of the ground to form larger surfaces of shear. Failure progressively radiates outward from these centers, toward the toe and the crest of the slope.

In homogeneous plastic ground of the sort commonly found in unstructured clay, this will result in either a circular or near-circular rotational failure. Most clay are not homogeneous; some contain vertical or near vertical fissures by overconsolidation inherited either from geological history, or from desiccation in hot dry or cold dry climates, including those of



Tension Cracks, Fig. 1 Common examples of tension cracks; (**a**) rotational failure, (**b**) translational failure, (**c**) toppling failure and possible water pressure profiles associated with them, and (**d**) failure seen in Chalk cliffs of Southern England

the Ice Ages, or from desiccation associated with vegetation, especially around the roots of trees. The body of the ground moving toward a slope face utilizes these vertical surfaces and pulls apart from them as it deforms, to form a tension crack. Such cracks seem to be able to extend downward behind the slope crest at the same time as the shear surface of ultimate failure progresses upward toward the crest. Eventually the two meet below ground level.

In more coarsely granular materials such as sand, shear failure tends to be planar, and in rocks it usually occurs by translation on either one or more semi-parallel planes or by toppling, the planes being provided by either bedding or jointing or cleavage. In these slopes, tension cracks may also develop especially if suitably oriented discontinuities provide surfaces for separation that enable developing cracks to meet the major surface of sliding. In toppling, movement of the slope is by rotation which in itself creates a tension crack. These basic characters are illustrated in Fig. 1a–d.

Tension cracks reduce the stability of slopes (i) by shortening the area of sliding surface and thus the magnitude of any cohesion that may operate to resist sliding, (ii) by providing access for water to soften and thus weaken the sliding surface, and (iii) by enabling water ponded in the crack to exert a pressure directed out of the slope and against the normal component of shear on any sliding surface to which it is hydraulically connected.

The development and depth of tension cracks are not easily predicted, and care should be taken when either incorporating them or excluding them from an analysis (Chowdhury and Zhang 1991). Field evidence from previous failures in similar material provides a sound basis for the analyst to follow (Leffingwell 1919; Morgan 1971). However, even here caution is required as back slopes formed from tension cracks are themselves inherently unstable and usually fail, so disguising their former presence and misleading field reconnaissance.

Cross-References

- ▶ Clay
- ▶ Desiccation
- ▶ Failure Criteria
- ▶ Landslide
- ▶ Mass Movement
- ▶ Shear Stress

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Thermistor

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Synonyms

Thermally sensitive resistor

Definition

A thermally sensitive resistor, the primary function of which is to exhibit a change in electrical resistance with a change in body temperature (EIA Standard 1963).

Characteristics

Thermistors are metallic oxide electronic semiconductors which are manufactured from oxides of nickel, manganese, iron, cobalt, magnesium, titanium, and other metals. All are epoxy encapsulated with two leads. The first reported thermistor was attributed to the British physicist Michael Faraday in 1832, based on his study on the semiconducting behavior of silver arsenide (Ag_2S) (Faraday 1832).

The relationship between the resistance of a thermistor and its body temperature may be expressed as

$$R(T) = R_0(T_0) \cdot \exp[\beta \cdot (1/T - 1/T_0)] \tag{1}$$

where β is the "material constant" of the thermistor, $R(T)$ is the resistance at temperature T under self-heated conditions, and $R_0(T_0)$ denotes the resistance at T_0 with zero electrical power dissipation (Sapoff and Oppenheim 1963). T and T_0 are the Kelvin temperatures. For accurate temperature measurements, the Steinhart–Hart equation is a widely used third-order approximation:

$$1/T = A + B \cdot \log R + C \cdot (\log R)^3 \tag{2}$$

where T is the Kelvin temperature and R is the resistance. For each thermistor, the Steinhart–Hart parameters A, B and C, are specified and obtained by high accuracy of laboratory calibration (Steinhart and Hart 1968).

Accuracy, resolution, and stability are specified for each thermistor. After high-accuracy laboratory calibration, the accuracy of a thermistor is generally better than 0.02 °C in the measurement of temperature over a 200 °C range. The resolution of thermistors can be up to 0.001 °C since they exhibit high temperature coefficients of resistance. Thermistors can be effective for several years because finished thermistors are chemically stable and not significantly affected by aging or exposure to strong fields of hard nuclear radiation.

The thermal time constant, defined as the time required for a thermistor to indicate 63.2% of a newly impressed temperature, is usually less than 1 s since the size of thermistor is very small, for instance, high-quality temperature sensors have accuracy of ±0.002 °C, resolution of <0.00005 °C, drift of ~0.002 °C/yr., and thermal time constant of 0.1 s.

Applications

Thermistors have been manufactured commercially since 1930. They are now widely used in a variety of electronic applications, most often as temperature sensors with a negative temperature coefficient (NTC type) in Earth science and engineering areas for *in-situ* temperature measurements such as in boreholes and in sub-seafloor measurements (e.g., Pfender and Villinger 2002).

Cross-References

▶ Borehole Investigations
▶ Geothermal Energy
▶ Instrumentation
▶ Monitoring
▶ Thermocouple

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Thermocouple

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Definition

Thermocouple – consist of two dissimilar conductors joined at one end to form a hot junction which produces a voltage. As the junction is heated or cooled the voltage changes and is converted to return as temperature. The Thermocouple Temperature Sensor is used in engineering geology for measuring temperature in soil and rock. Simply put, thermocouple is a “thermometer” capable of reading the temperature on the rock face or at different depths of the rock mass.

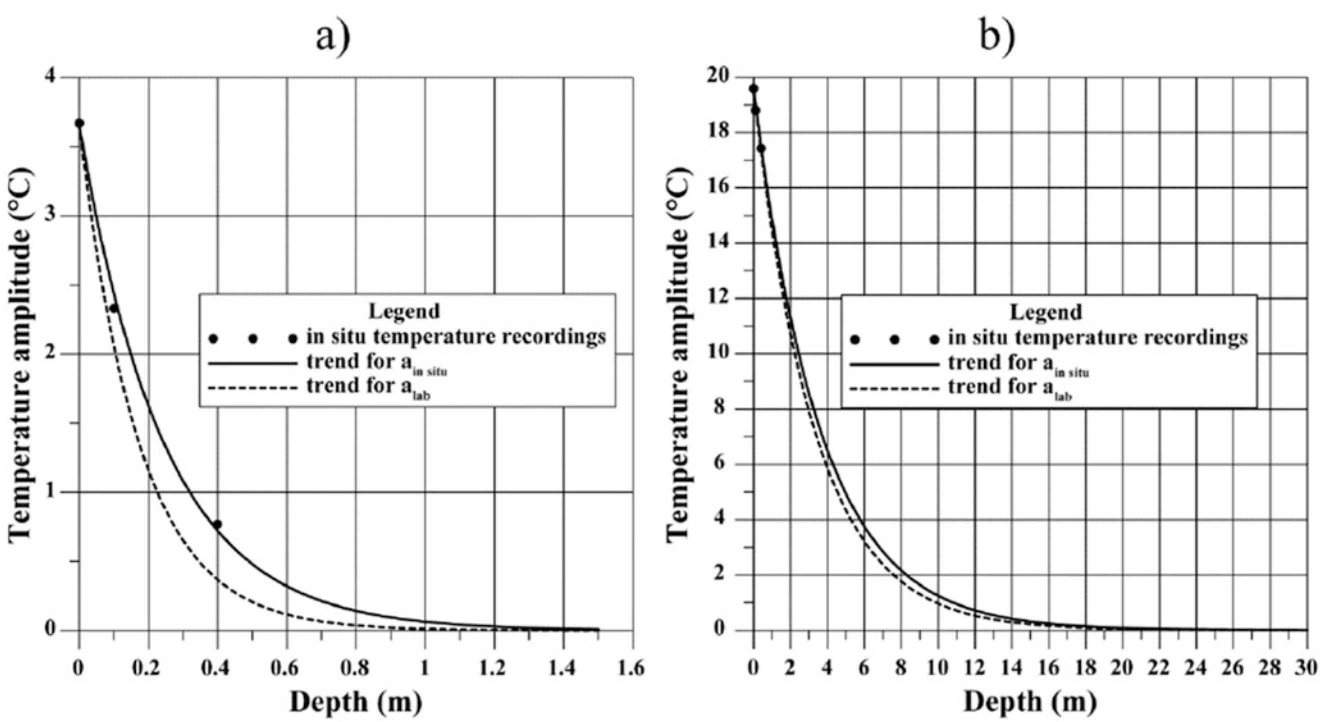
Overview

A large number of slope failures, where no significant trigger has been proposed and the rock failure mechanism remains unclear, have been recognized as hazards triggered by meteorological factors. Temperature oscillations produce cyclic thermal strains in rock masses which result in deterioration processes associated with cracking and jointing, in opening/closing of joints/fractures, and in jointed rock masses which in turn may initiate thermally driven displacements from slow creep (block ratcheting in terms of Pasten et al. 2015), further toppling and tilting to rapid movement in the form of rock cliffs overturning to rock falls.

Use

In engineering geology, measuring rock surface temperature or temperature at shallow depths within the rock mass is generally conducted in two directions to:

1. Quantitatively estimate the rate of temperature oscillations for thermal rock fatigue or thermal shock, respectively. If the rate of temperature change exceeds certain thresholds (Yatsu 1988) regardless of the thermometer scale (subzero or above zero recordings), thermal shock may result and the rock breaks.
2. To confirm the existence of thermally driven non-reversible or plastic deformations of rocks that might be triggers



Thermocouple, Fig. 1 Curves showing the penetration of the diurnal (**a**) and annual (**b**) temperature waves calculated according to 3-year temperature recordings from three different depths. The curves assigned as $a_{in\ situ}$ represent the data calculated from in situ recordings while those assigned as a_{lab} represent the data determined in the laboratory or controlling factors for larger rock volume displacements (Vlcko et al. 2009; Bakun-Mazor et al. 2011; Gunzburger and Merrien-Soukatchoff 2011).

To acquire an insight into the heat balance both at the rock surface and deeper in the rock mass, it is necessary to install a monitoring system comprising several thermocouples inserted at different depths in the rock mass. The time series of temperature data facilitate the interpretation of the heat transfer inside the rock body, maximum or minimum time of temperature reached at any point below the surface, actual temperature, short and long-term temperature range, time lag, etc., and therefore establish several thermomechanical parameters needed for further calculation or establishing the thermomechanical model (Fig. 1).

Cross-References

- ► Engineering Geology
- ► Geotechnical Engineering
- ► Rock Mass Classification

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Tiltmeter

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Synonyms

Accelerometer transducer tiltmeter; Clinometers; Electrolytic level transducer tiltmeter; Inclinometer; Mechanical tiltmeter; Vibrating wire transducer tiltmeter

Definition

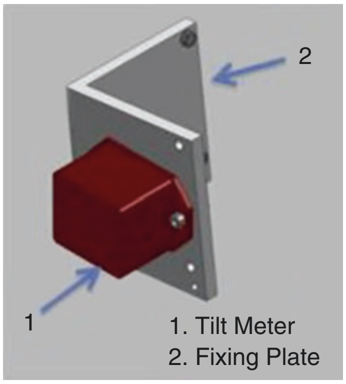
A tiltmeter is a sensitive inclinometer designed to measure very small changes from vertical orientation, either on the ground or in structures.

Context

Tiltmeters are used extensively for various types of monitoring, such as the response of dams to filling, small movements of potential landslides, inclination of towers or rotation of retaining walls, movement of piers and piles, changes to the orientation and volume of hydraulic fractures, morphological changes to volcanoes, or the response of built structures to various influences such as loading and foundation settlement.

Tiltmeters may be purely mechanical or they may incorporate vibrating-wire or electrolytic sensors for electronic measurement. A sensitive instrument can detect changes as minute as one arc second. Precision is expressed in arc seconds, or gons. Tiltmeters have a long history of development and use, somewhat comparable to the history of seismometers. The very first tiltmeter consisted of a long-length stationary pendulum. These early instruments were used in the very first generation of large concrete dams, and are still in use today, but often augmented with newer technology such as laser reflectors. Although they had been used for other applications, they have certain distinct disadvantages, such as their considerable length and sensitivity to air currents. As applied to dams, they are slowly being replaced by modern electronic tiltmeters. The modern electronic tiltmeter relies on a simple bubble level principle, as used in the common carpenter level. Small changes in the level are recorded using a standard data logger. This arrangement is quite insensitive to temperature, and can be fully compensated using built-in thermal electronics.

Among the wide range of available tiltmeters the most notable is the *Beam Sensor Tiltmeter* (the sensor is mounted on a rigid beam with a defined gauge length, typically 1–2 m long, anchored to the structure) and the *Portable Tiltmeter* connected from tilt plate to tilt plate to obtain measurements. Most tiltmeters today adopt a newer technology: the micro-electromechanical systems (MEMS) sensor that enables tilt angle measuring tasks to be performed conveniently in both single and dual axis mode (measurement range of +/− 15°). Ultra-high precision 2-axis MEMS driven digital tiltmeters are available for quick angle measurement applications and surface profiling that require very high resolution and accuracy of one arc second. Modern digital tilt meters may be read locally using portable readouts, or centralized with a data logger for remote wireless monitoring solutions. The 2-axis MEMS driven tiltmeters can be digitally



Tiltmeter, Fig. 1 Example of dual axis digital tiltmeter (MEMS) with is fixing plate

Tiltmeter, Fig. 2 Detail of the tiltmeter installed in the wirless monitoring system in the siq of Petra, Jordan (2013)

compensated and precisely calibrated for nonlinearity and operating temperature variations, resulting in higher angular accuracy and stability performance over wider angular measurement ranges and broader operating temperature ranges (Figs. 1 and 2).

Cross-References

- ▶ Deformation
- ▶ Foundations
- ▶ Geohazards
- ▶ Inclinometer
- ▶ Instrumentation
- ▶ Mining Hazards
- ▶ Retaining Structures
- ▶ Rock Bolts
- ▶ Rock Mechanics
- ▶ Subsidence
- ▶ Surveying

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Tropical Environments

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Synonyms

Tropics

Definition

The Earth's tropical environment is geographically defined as the region of the globe lying between the latitudes of the Tropic of Cancer, North latitude 23.5°, and the Tropic of Capricorn, South latitude 23.5°, including the Equator (Fig. 1).

Introduction

About 40% of Earth's population resides in a tropical environment (Fig. 1). People's lives in the tropical environment are shaped by inherent climatic, geologic, tectonic, and geomorphic features. The tropics are classically described as a hot and humid region. However, this geographic zone is a diverse climatic-geographic region characterized by marked variation in rainfall manifested as arid and humid tropics. Tropical landforms include mountain ranges, islands, coral reefs, river valleys, deserts, coastal features, and karstic landscapes. Coral reefs are a unique feature of the tropics. Some of the key mineral resources, for example, bauxite, owe their existence to hot and humid tropical climate weathering. With average temperatures near 25 °C and scenic landscapes, especially in coastal areas, tourists are attracted to the tropics.

Globally, rain forests in the tropical environment are critical *"carbon sinks"* to maintain a cool Earth. Sea-surface

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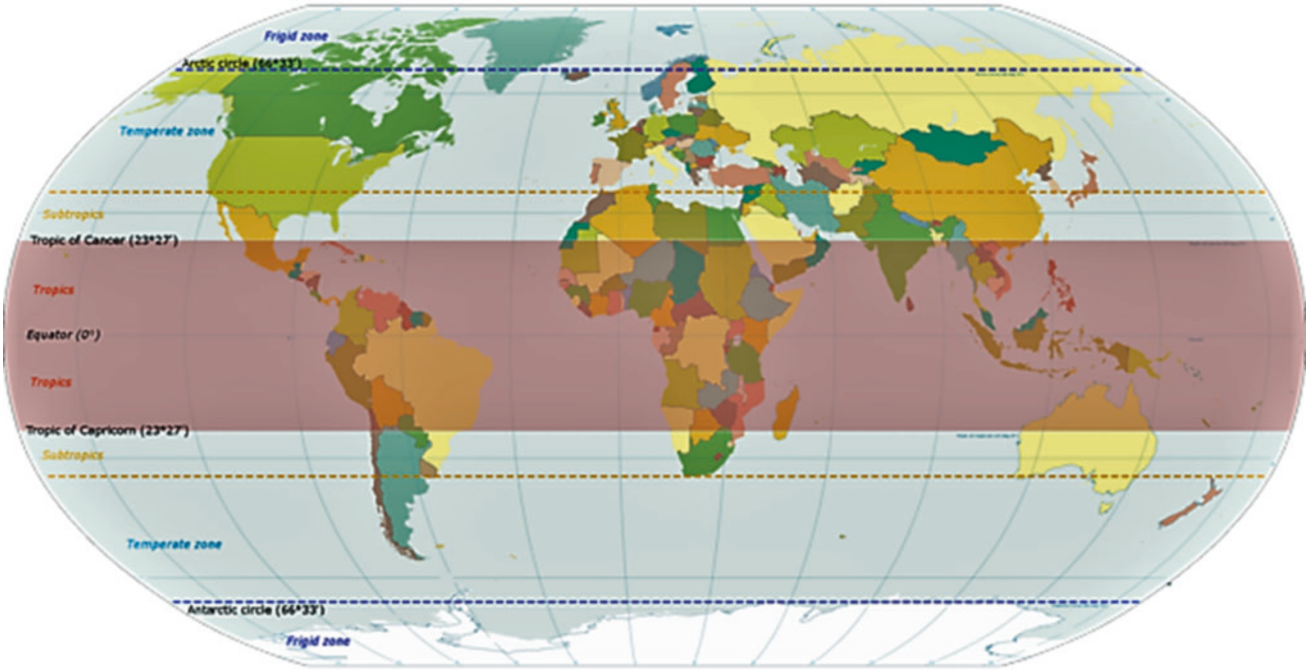


Tropical Environments, Fig. 1 Tropical environment, tropics comprise Australia and Oceania – Small Island Oceanic States, South East Asia, South Asia, Northern Africa and Middle East, Central and South Africa, South America, Caribbean, and Central America. Tropics, from Wikipedia, the free encyclopedia: https://en.wikipedia.org/wiki/Tropics. Accessed on 25 Dec 2017

temperatures in tropical oceanic areas of the equatorial Pacific control climate and support world weather systems, for instance, El Niño and La Niña. With abundant sunshine all through the year, solar energy is a boon to the tropical environment.

Tropical environments can have negative attributes such as multiple natural hazard processes related to earthquakes, volcanism, tropical storms, landslides, weathering, riverine flooding, coastal processes – tsunami and storm surge, and arid tropic processes. However, it is these processes that have partly created the beautiful tropical landscapes.

Countries with a tropical environment have witnessed rapid urbanization and industrialization over the last two decades. According to Gupta and Ahmad (1999a), the urban population in less developed countries between 1950 and 1990 increased by about 100%. By 2000, cities with a population more than ten million, termed megacities, were mainly in the tropics (Gupta 2012). However, except for Singapore and Hong Kong, most of the countries in the tropics are classified as "low-income countries" (World Bank 2017) and remain underdeveloped.

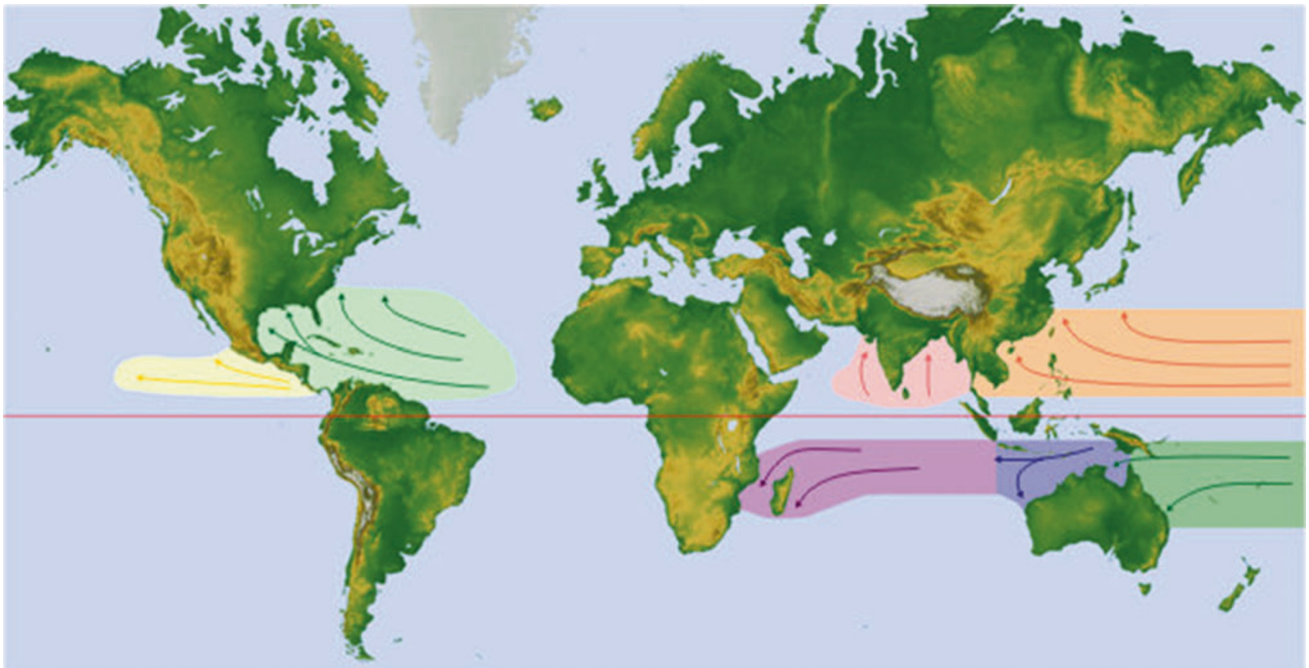
Tropical Climatic Zones and Characteristics

Fully understanding the tropical environment requires appreciation of the variety of climatic zones present as defined by the Köppen-Geiger (KG) climate classification system based on temperature and precipitation and updated by Kottek et al. (2006). The world map of the Köppen-Geiger climate classification map is publicly available at http://koeppen-geiger.vu-wien.ac.at/pdf/Paper_2006.pdf (Accessed on 30 November 2017). According to the KG climate classification system, the tropical zone comprises equatorial climates (A) and arid climates (B). Each of these climatic zones is further subdivided as shown in Table 1 and described in the following paragraphs.

Tropical Environments, Table 1 Description of the Köppen-Geiger tropical climate zones, modified from Kottek et al. (2006)

Type	Description	Criterion
A	**Equatorial climates**	$T_{min} \geq +18\ °C$
Af	Equatorial rainforest, fully humid	$P_{min} \geq 60$ mm
Am	Equatorial monsoon	$P_{ann} \geq 25\ (100 - P_{min})$
As	Equatorial savannah with dry summer	$P_{min} < 60$ mm in summer
Aw	Equatorial savannah with dry winter	$P_{min} < 60$ mm in winter
B	**Arid climates**	$P_{ann} < 10\ P_{th}$
BS	Steppe climate	$P_{ann} > 5\ P_{th}$
BW	Desert climate	$P_{ann} \leq 5\ P_{th}$

T temperature in °C, T_{ann} the monthly mean temperatures, the warmest and coldest months T_{max} and T_{min}, respectively, *P* precipitation in mm/year, P_{ann} the accumulated annual precipitation, P_{min} the precipitation of the driest month, P_{th} dryness threshold in mm



Tropical Environments, Fig. 2 Tropical cyclone formation regions with mean tracks and digital elevation model of tropics; adapted from National Oceanic and Atmospheric Administration (NOAA) Tropical Cyclones, Tropical Cyclone Formation Basin, National Weather Service JetStream – Online School for Weather: http://forecast.weather.gov/jetstream/tropics/tc_basins.htm. Accessed on 25 Dec 2017

Equatorial rainforest, fully humid (Af), wet climates are characteristic of the equatorial regions between 10° North and South latitudes, and typical of the South-East Asia Archipelago, parts of Central Africa, North-Central South America, and Small Island states. Rain forests are critical *"carbon sinks"* to maintain a cool Earth. The hot and humid conditions and annual precipitation more than 2000 mm is an environment which sustains high weathering rates and rainfall-induced sediment flows including mudflows and debris flows.

Equatorial monsoon climate (Am) is characterized by the "monsoon wind system" that flows from sea to land in the summer months and the reverse flow in the winter. There are dry and wet seasons with high and intense rainfall that triggers devastating monsoon floods in South Asia, Bangladesh, and northern parts of India. Monsoon rains sustain agricultural crops in areas with high population and recharge aquifers.

Arid and semiarid climates in some areas in the tropics experience low annual rainfall, about 10–30 cm, with high temperatures. Equatorial savannah with dry summers (As) and equatorial savannah with dry winters (Aw) are typical of Australian tropics, South East Asia, South Asia, much of Central Africa and Indian Ocean island states; Central and Northern South America, the Caribbean, and Central America. Wet and dry seasons are characteristic of African extensive grassland ecosystems, such as the Serengeti Plain. The dry and wet seasons experience precipitation averaging less than 60 mm monthly and 1000 mm, respectively. Droughts and floods characterize this zone.

Also, this climatic region is affected by tropical storms from June 1 to November 30, identified as typhoons in the Pacific, cyclones in the Indian Ocean, and hurricanes in the Caribbean (Fig. 2) (National Oceanic and Atmospheric Administration (NOAA), 2013). A very important global issue is the effect of climate warming on tropical storms. According to the Geophysical Fluid Dynamics Laboratory (2017), "we conclude that at the global scale: a future increase in tropical cyclone precipitation rates is likely; an increase in tropical cyclone intensity is likely; an increase in very intense (category 4 and 5) tropical cyclones is more likely than not; and there is medium confidence in a decrease in the frequency of weaker tropical cyclones. These global projections are similar to the consensus findings from a review of earlier studies in the 2010 WMO Assessment."

Salient Physical Features and Processes of the Tropical Environment

Accounting for approximately one-third of the Earth's landmass, the tropical environment comprises a set of diverse landforms and landscapes. As far as safety of citizenry and sustainability of built environment are concerned, most of the land area in the tropics lies in active convergent tectonic plate boundary zones and along tropical storm tracks (Fig. 2). Landforms in the convergent zones include ocean deeps, volcanic islands, and Small Island states in South East Asia, Oceania, and the Caribbean, mountain chains in South Asia, the Himalaya, associated with a large alluvial plain of major rivers, the Andes and active volcanoes in Central American. In the plate interiors, important landforms are volcanic hotspots, Hawaiian Islands, and cratonic areas. The passive margins include major delta regions and submarine fans.

These areas are characterized by pervasive Quaternary tectonic movements, earthquakes, volcanism, mass movements, riverine and coastal flooding, and atmospheric storms producing extreme weather events. Environmental degradation and vulnerability to climate change in the tropics is related to anthropogenic changes that influence geomorphic processes and their rates. An acute shortage of domestic water supplies in the tropics directly impacts sanitation, hygiene, and health. In general, food security is threatened by lack of water resources which are stressed due to over exploitation and pollution.

Many societies located in the tropical zone are shaped by a variety of geophysical, geologic, atmospheric, and hydrogeological hazardous processes. Following are a few areas to illustrate this point: Luzon province in Philippines (1991 Mt. Pinatubo eruption); Banda Ache in Indonesia and circum-Indian ocean areas (2004 tsunami); Bangladesh delta region and North Bihar in the Indo-Ganga Plains (recurrent cyclones and floods); State of Eritrea (desertification); Island of Montserrat (volcanic eruption); Vargas state in Venezuela (debris flows); Armero in Colombia (volcanic mudflow); and Mexico City (earthquake).

Given their locations at convergent plate boundaries, much of the mountainous landscape of the tropics is in areas of high seismic activity. Slope movements are ubiquitous over much of the mountainous tropics triggered by earthquakes, and rainfall associated with tropical storms. Landslides in tropical zones continue to cause extensive damage to road networks and property, including injuries and fatalities. Landslide hazard assessment in seismically active wet tropics is challenging since both seismic shaking and moisture in slope materials are important factors in triggering landslides. Active Quaternary volcanoes dominate the landscape of South East Asia, the Caribbean, and along the Pacific coast of Central America. Many a city and village live in the shadow of an active volcano.

Liquefaction and landslides are a widespread and significant earthquake-induced ground deformation hazard in the tropical environment. Bommer and Rodriguez (2002) studied earthquake-induced landslides in Central America spanning 1447 to 2001, and listed that Mw 5.6 was the lowest earthquake magnitude that triggered landslides. Most of the areas affected by earthquake-induced landslides may experience rivers dammed by landslide debris, changed river courses, and stripping of hill slopes of vegetation. Also, earthquake-induced submarine landslides trigger tsunami along the Pacific coast of Central America and South-East Asia. A destructive submarine landslide near Aitape coast, Papua New Guinea, was triggered by the Mw 7.0, July 17, 1998, created a tsunami which impacted a 30 km stretch of the coast and killed more than 2000 and injured thousands more.

Examining Human Influence Within the Tropical Environment

Kingston, Jamaica, is used here to illustrate inherent geophysical-geomorphological-meteorological-hydrogeological constraints on sustainable development of tropical urban environment and governance and the practice of urban planning and engineering geology in a geomorphically sensitive landscape. Kingston (Jamaica) is located in a tectonic and geomorphic unstable environment with multiple hazards; a geomorphologically sensitive terrain which makes it difficult to manage. This metropolitan area illustrates the inherent physical environment constraints in a tropical environment and the dependence of urban planning and engineering practice for resilient city development on a basic data set defining: (1) basement geology, geologic structure, and tectonics; (2) fluvial, volcanic, seismic, mass movement processes; (3) hydrology, rainfall, and tropical storms, (4) drainage analysis results; (5) Quaternary geology (landforms in upland, slopes, transition zone uplands and lowlands, alluvial fans, river valleys, deltas, and coastal zone), (6) geotechnical characters of earth materials, soils present, and depth of weathering, (7) distribution of vegetation, (8) ground water regime, and (9) available earth resources and extraction (Gupta and Ahmad 1999b). The following discussion is drawn from Gupta and Ahmad (1999a, b) and Gupta (2012).

In countries located in plate boundary zones within the tropics, neotectonics, bedrock characters, seismicity, and duration and intensity rainfall control geomorphic processes and diverse landforms. The island of Jamaica is located in a 200 km wide, seismically active strike-slip plate boundary zone between the Caribbean and North American tectonic plates. Geologically, young landforms in Jamaica are a mosaic of montane areas with steep hillsides, faulted mountain fronts, highly faulted and deeply weathered bedrock, fault-controlled river valleys, karst land, coastal plains and gravelly paleofans. High annual precipitation, short-duration and intense rainfall associated with tropical storms trigger widespread slope failures as well as floods. Recurring seismic activity produces damage and additional landslides through earthquake-shaking. Montane rivers carry high sediment loads. Coastal flooding related to storm surge and tsunami is common. Population pressures and urbanization contributes to slope destabilization following encroachment of hill slopes, deforestation, and vegetation conversion. Global warming may increase cyclone risks and inundation of low-lying coastal areas. This is a typical description of geomorphically sensitive areas in the tropics.

The capital city of Jamaica, Kingston, was founded on the Liguanea gravel fan in 1692 following the MMI X June 6, 1692, earthquake destruction of Port Royal, the buccaneer city located on the Palisadoes Tombolo which hosts the historical site of Port Royal and the N.M. Manley Airport. Urbanization of Kingston is expanding into the surrounding low limestone hills and the highly dissected Port Royal Mountains that rise to a maximum height of 1353 m. The Port Royal Mountains are comprised of highly fractured (joints and faults) and deeply weathered volcaniclastics, andesites, and granodiorite.

The typical rainfall pattern for the Kingston area is seasonal tropical depressions and storms and hurricanes. These rainfall events trigger riverine flooding and deep and shallow slope failures dominated by debris flows. Destructive hurricanes include 1951 Charlie, 1963 Flora, 1988 Gilbert, 2004 Charley and Ivan (damage US$360 m), and 2005 Dennis and Emily.

The Ms 6.5 Kingston 14 January 1907 earthquake destroyed most of the Kingston business district. The M5.4 earthquake, 13 January 1993, caused widespread damage. All significant earthquakes that affected the Greater Kingston area and the Palisadoes Tombolo triggered landslides, liquefaction-related ground failures, submarine landslides (which damaged cables), and localized tsunami. "The parishes of Kingston including Port Royal, and lower St. Andrew were probably the worst possible location to choose for the capital city of Jamaica" (Shepherd 1971, quoted in Gupta and Ahmad 1999a). Seismic hazard maps for the Kingston Metropolitan Area were prepared in 1999 and 2013.

Landslide landforms are widespread on the hillslopes in the Port Royal Mountains. Landslide triggers thresholds are generally (i) >M5 earthquakes and (ii) about 200–300 mm of rainfall in 24 h. Seasonal tropical storms initiate widespread rainfall-induced shallow landslides that quickly turn into debris flows and commonly affect most of the roads in the area. It is common for old landslides to be reactivated during subsequent rainfall events. These slope failures are difficult to manage. This problem is exacerbated by deforestation, vegetation conversion, and anthropogenic activities. Shallow landslides cause accelerated soil erosion in the uplands that are responsible for the siltation of two water reservoirs in Kingston. Landslides in the Kingston Metropolitan area and controlling factors were mapped in detail and landslide susceptibility maps for deep and shallow landslides were prepared in 1999 including guidance to use maps and hazard reduction.

The natural drainage on the Liguanea Fan has been extensively modified and channeled into seven major gully systems to facilitate seasonal rainfall runoff. Kingston is affected by urban flooding due to insufficient discharge capacities of the gully system. The ever-increasing impervious cover due to urbanization causes increased overland flow and flash flooding. Gullies crossing roads are one of the most dangerous urban flood hazards. Riverine flooding is confined to the Rio Cobre and Hope River. Flood hazard maps (100-year) are available for these rivers.

Recurrent hydrogeological disasters in the Kingston area, and all over the island, are costly. Riverine flooding and landslides destroy and damage housing, businesses, agricultural crops, critical facilities and infrastructure, particularly road transport corridors (totalling more than 15,000 km), and the water sector. Following 1988 Hurricane Gilbert hydrogeological hazard events, the costs of island-wide damage to roads and the water sector was estimated to be more than US$ 19 m and US$ 86 m, respectively.

Summary

It appears that geomorphic factors and multiple hazardous processes, particularly extreme weather and hydrogeological disasters in the tropical environment are a significant factor in their underdevelopment (UNISDR 2015). Sustainable development and built landscape of tropical environments should be underpinned by a comprehensive multiple hazard evaluation underscored by the Sendai Framework for Disaster Risk Reduction 2015–2030 (UNISDR 2015). The best solution for sustainable development of tropic built-up areas of course is to match the use of the land with inherent physical constraints, but that is seldom possible. Thoughtful use of engineering geologic principles and practices seem the next best means of supporting sustainable development.

Cross-References

▶ Climate Change
▶ Coastal Environments
▶ Desert Environments
▶ Earthquake
▶ Engineering Geology
▶ Erosion
▶ Floods
▶ Geohazards
▶ Hazard
▶ Karst
▶ Landforms
▶ Landslide
▶ Liquefaction
▶ Mass Movement
▶ Mountain Environments
▶ Physical Weathering
▶ Tsunamis
▶ Volcanic Environments

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Tsunamis

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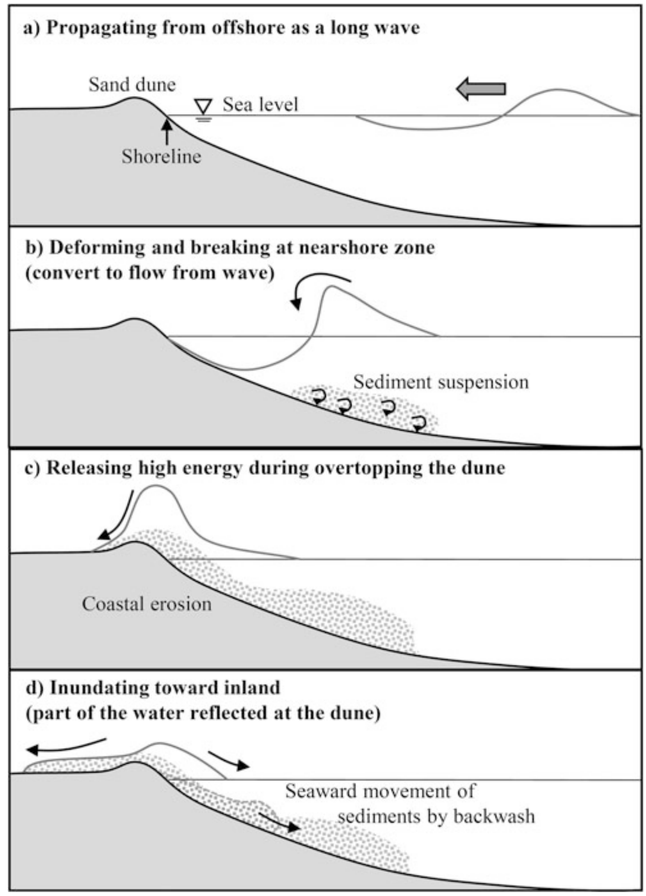
Definition

A series of travelling waves of extremely long length and period, usually generated by disturbances associated with earthquakes occurring below or near the ocean floor... (Intergovernmental Oceanographic Commission (IOC) 2016)

Although often associated with earthquakes, tsunami can also be generated by any types of physical disturbances of seawater by landslides, rock fall, submarine volcanic eruptions and associated pyroclastic flows, and meteorite impact but waves that are generated by climatic and tidal effects, are not included in the tsunami definition (e.g., Intergovernmental Oceanographic Commission 2016). Long wavelength and long period waves generated offshore convert into a series of destructive waves once they approach shallow coastal areas (Fig. 1). The waves then break and flow destructively on to the onshore area. Tsunami erosion and sedimentation on the sea floor or on land causing severe damage to the natural coastal geomorphology, marine and terrestrial ecosystems, and man-made facilities (e.g., ports, power plants, wave

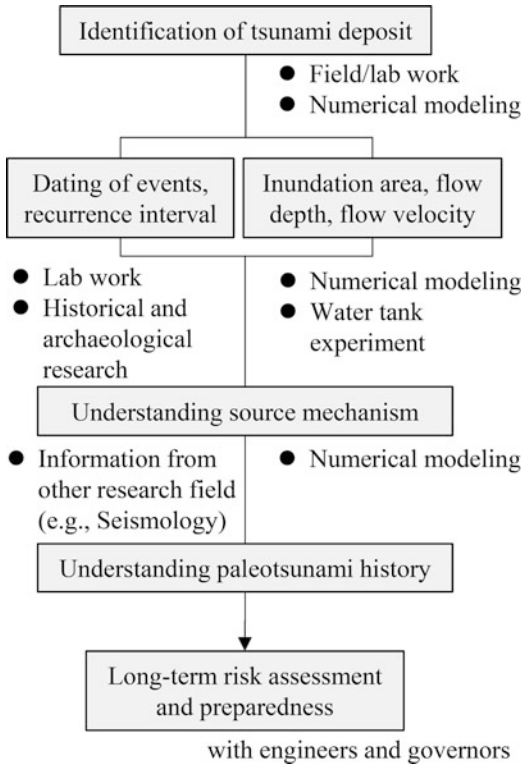


Tsunamis, Fig. 1 (**a**) Propagation of a long period long wavelength tsunami wave from the offshore. (**b**) Deformation of the wave due to shoaling. Tsunami develops a shorter wave length and higher amplitude. Then, the wave breaks and converts into a flow. (**c**) Flow hits and overtops the coastal dune zone. (**d**) Water inundates while part of the water is reflected from the coast

breaks and cities). Affected coasts and ecosystems sometimes recover quickly, but recovery may take a long time or even not occur at all depending on the coastal situation.

Tsunami-reworked allochthonous sediments, ranging from mud to boulder size are called tsunami deposits (e.g., Goff et al. 2012). On land, tsunami deposits cause chemical contamination since saltwater as well as allochthonous minerals are deposited, and it may be important to remove these from certain places such as agricultural fields. On the other hand, it is well known that tsunami deposits are useful in understanding paleo-tsunami history for hundreds to thousands of years or even further into geologic time. Therefore, tsunami geology or paleo-tsunami research (Fig. 2) has been conducted in many coastal areas since the late 1980s.

For example, the AD2011 Tohoku-oki tsunami, which was generated along the central part of the Japan Trench, had possible predecessors in the AD869 Jōgan tsunami and older events which were recognized by geologists well before the 2011 event (e.g., Minoura et al. 2001). Clarifying paleo-tsunami



Tsunamis, Fig. 2 Flow chart of paleo-tsunami research

histories using tsunami deposits is crucial for future tsunami risk assessment and making preparations in 'at risk' areas. The deposits specifically allow estimation of the recurrence intervals and of local tsunami size (inundation area, flow depth, and flow speed). If the tsunami source can be specified, then it is also useful to estimate the mechanism and location of the fault and magnitude of the associated earthquake (e.g., Nanayama et al. 2003). However, identification of a tsunami deposit is not easy because similar deposits can be formed by other events such as flood and storm surges or waves (e.g., Goff et al. 2012).

Sedimentary features indicating strong flow (e.g., erosional contact, upward fining), mixture of allochthonous material (e.g., microfossils, chemical signature), and variations of spatial thickness and grain size are the key to identifying tsunami deposits geologically. Forward and inverse numerical models are recent advances in identification of the origin of tsunami deposits and also to estimate local tsunami size (e.g., Sugawara et al. 2014).

Cross-References

▶ Contamination
▶ Earthquake
▶ Geohazards
▶ Landslide
▶ Volcanic Environments

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Tunnels

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Synonyms

Mine; Subway; Underground passageway

Definition

Tunnels are artificially constructed passageways beneath barriers such as a stream or other bodies of water, hill or other earthen structures, or a building. In all cases, a tunnel has a minimum of two openings, one for entry and another for exit, depending on the point of ingress and egress. A tunnel can convey fluids either by gravity or under pressure through either a lined or unlined excavation. Other uses are for motorized vehicular transport from one point to another where horizontal curvature or cultural restrictions prevent removal of overburden or the barrier in question. Simple uses include human passage. Tunnels can be built through rock, soil, or a combination of both in either an open-cut/cover arrangement or by means of mining using drill and blast, tunnel boring machine, or sequential excavation methods.

Introduction

Tunnels have their origin in the early industrialized world in urban settings where conventional transport ways were too crowded or too circuitous. The first large-scale tunnels appeared in the eighteenth century in Great Britain and France. These tunnels were excavated partly by hand and partly using explosives. Improvements in drilling procedures and tools, manufactured high-energy explosives, and safe practices allowed more conventional drill-and-blast tunnel method to develop. This method was state of the art for over 150 years and was perfected in particular by the Chinese, who were instrumental in constructing rail tunnels in the mid- to late nineteenth century in California.

Tunnels were seldom lined in competent bedrock, but in the early days of water and railway tunnels in soils under Greater London, Paris, New York City, and Chicago, brick lining was the preferred material.

In the 1960s, new methods for excavation and support were developed by necessity to accommodate ever greater rock strengths, mixed ground conditions, and larger diameter bores. In addition, the use of drill-and-blast mining in older, developed cities was not considered desirable due to ground vibration. Mechanized mining using shields was designed by British engineers in the early to mid-nineteenth century as a way of tunneling beneath the River Thames and quickly became the preferred method as it allowed for a relatively protected and safe working environment for miners. The design of the tunnel boring machine or TBM is attributed to The Robbins Company for hard rock mining in the mid-1950s. Rapid improvements have been made in TBM technology such as cutters, the placement of temporary and sometimes final support simultaneously with mining and disposal of tunnel muck via trailing gear. Where mixed face conditions or cost was critical, a simple yet effective method of mining termed the New Austrian tunneling method (NATM) or the more conventionally accepted sequential excavation method (SEM) was developed in Austria. It has the unique advantage of allowing sequential mining and installation of temporary support using the stress from the inner tunnel acting on sprayed shotcrete.

In the last 20–30 years, computer design has allowed for sophisticated modeling of soil and rock using finite element analysis programs (i.e., FLAC). These are critical to evaluating subsurface stress fields and the impact on surface structures susceptible to settlement. In addition, new techniques have developed to assess seismic risk from ground shaking (which underground is not as big an issue as on the surface), tunnel crossings of faults, and control of groundwater inflows.

Exploration

Before a tunnel can be designed and built, the geologic and geotechnical team must decide whether a tunnel is feasible, necessary and cost-effective. Various approaches on technical feasibility can be made and include factors such as tunnel diameter and length, lifespan of the structure, whether tunnels have previously been built in the area, and most importantly what influence the ground conditions would be on design and construction. A phased approach is generally the best method for tunnel feasibility. The phased approach process may begin with a desktop study and geologic reconnaissance, proceed to simple subsurface investigation and preliminary design, and conclude with final design and preparation of bid documents. A discussion of these steps follows.

Desktop Study and Field Reconnaissance

Typically, a geologic study for a new tunnel will include a review of existing geologic, structural, and groundwater reports. If little geologic information exists, then a simplified field program including surface geophysics, outcrop, and surface geologic mapping is imperative. These geologic data are best stored, managed, and displayed in a geographic information system (GIS). New data can be added to the GIS in later phases of the design process.

The outcome of the desktop study and field reconnaissance is to determine potential route alignments and consideration of tunnel invert depths with respect to depth of weathering, groundwater, and for location and investigation of tunnel-related structures (shafts, pump stations, and portals).

Subsurface Investigation for Preliminary Design

The main objective of subsurface geotechnical investigations is to obtain the *in situ* properties of Earth materials and identify geologic structures to facilitate the tunnel design and construction methods (USNCTT 1984). Effective planning of the investigation is critical to allow for understanding the risks of advancing a tunnel through unknown ground conditions, areas susceptible to settlement or disruption, and the costs associated with these factors.

A good starting point is determining the best locations for tunnel portals. The use of horizontal or angled directional drilling at these locations is advantageous as it mimics the tunnel boring process. A good rule of thumb for vertical borings is to extend the depth of borings two times the tunnel diameter below the invert and to position each boring on approximately 1,000 ft (305 m) of the center if the geology is consistent along the tunnel alignment. If it is not consistent, then it is best to position borings to intercept geologic contacts or faults that hopefully are apparent from the field reconnaissance mapping above.

The subsurface program should also include *in situ* testing (pressuremeter, Goodman Jack, and packer permeability) to determine moduli hydraulic conductivity as well as borehole geophysics (Televiewer, P- and S-wave seismic, and E-logs). Obtaining high-quality core of soil and rock will allow appropriate laboratory testing of material properties.

The objective of the subsurface investigation is to develop a geologic model to be used by the tunnel designer and to modify to incorporate additional data and interpretations.

Subsurface Investigation for Final Design

Refinement of the tunnel design, permitting, and cost considerations may require additional geotechnical investigations. The scope of the investigations may include installing a series of groundwater monitoring wells to measure fluctuations, installation of instrumentation to measure potential ground movement, or additional borings to obtain more samples or to address changes in tunnel size, alignment, or depth. Refining the geologic model will provide better interpretation to include in the geotechnical baseline report (GBR) (UTRC 2007).

Types

Several types of tunnels have developed over the years to better serve the public. Geologic and cultural factors that have influenced these types include comprehension of the ground conditions, corridor restrictions, mechanized methods, economies of scale, and vertical/horizontal curvature.

Transportation Tunnels

Highway tunnels are unique in that they require positive ventilation and interior lighting and have speed restrictions related to horizontal curvature. Multilane highway tunnels can reach an open roof span of 60 ft (18 m) which can accommodate four lanes, provided tunnel lining, rock bolts, or other supports are used. Tunnels can be combined with overwater structures, as evidenced by the Øresund Bridge in Denmark and Chesapeake Bay Bridge and tunnel in the USA.

Rail tunnels may also require positive ventilation if they accommodate diesel-type locomotion. Large tunnels with long consists of cars typically displace a large amount of air, which can require ventilation shafts. The Gotthard base tunnel in Switzerland is the world's longest tunnel 57 km (35.4 mi), has a cover of over 2.3 km (1.43 mi), and required special consideration of rock bursting during the TBM drive.

Commuter rail lines generally are in congested urban areas. They are usually in a twin tube or dual track within a horseshoe configuration. Some commuter-rail tunnels are built in sections and placed into dredged channels in the sea bottom, including the Bay Area Rapid Transit (BART) Transbay tube between San Francisco and Oakland, California, in the USA.

Water Tunnels

Water tunnels are challenging from a geologic perspective that they have a tendency to leak into the formation surrounding the bore unless they are lined. There are other internal pressure and erosion factors to consider in unlined water tunnels.

Some water tunnels are relatively short and seldom used. These might include outlet structures and diversion tunnels from dams. These tunnels may require short term very high pressure and velocity to perform satisfactorily.

Specialty Tunnels

Historically, tunnels have provided a place of refuge, such as air raid shelters during times of wartime bombing raids. Tunnels can also provide conduits for other means of conveyance of goods and services beyond their original intended use. These can include conveyor belts for transporting mining products, pipelines, and electrical utilities, as well as storage of sensitive materials that are susceptible to atmospheric disturbance (moisture, light, humidity). Radioactive waste storage has generally been confined in caverns which may or may not have ingress or egress accommodations. Tunnels can also assume the role of drainage galleries for dewatering of rock structures and for providing drill rig access for dewatering operations.

Design Considerations

The tunnel design incorporates a number of factors that must be considered for completion of a successful project with the understanding of the geology being the most important factor affecting the design and construction of a tunnel. Anticipated soil, rock, and groundwater conditions allow the owner and engineer to plan and design alignments, assess the feasibility of tunneling methods, design tunnel support systems, and prepare bid documents. Accurate understanding of geologic and geotechnical conditions also enables contractors to submit appropriate bids, select proper tunnel equipment, and plan and schedule the construction work. This section provides an overview of some of the design considerations for tunnels including ground classification systems, groundwater and fault impacts, and gassy ground.

Ground Classification

A number of classification systems have been developed over the years to help correlate ground conditions and anticipated ground behavior for tunnel design. Ground classification systems are generally divided into two categories, soft ground and hard rock, which are discussed in the following sections.

Soft-Ground Classifications

For soft-ground tunnels, that is, tunnels in soil or soil-like sedimentary rock, the primary controlling factors are soil type, index properties (grain size and plasticity), and engineering properties (strength, modulus, and permeability). The two classification systems commonly used for tunnel applications in soils are the Unified Soil Classification System (UCSC), which provides a description of the soil particles, and the Tunnelman's Ground Classification System, which describes soil types and their anticipated behavior.

Tunnels, Table 1 Tunnelman's ground classification system

Classification		Behavior	Typical soil type
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move	Loess above water table; hard clay, marl, cemented sand, and gravel when not highly overstressed
Raveling	Slow raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, exposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff-fissured clays may be slow or fast raveling depending upon the degree of overstress
	Fast raveling		
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity and without perceptible increase in water content. Ductile, plastic yield, and flow due to overstress	Ground with low frictional strength. Rate of squeeze depends on the degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depth behind surface
Running	Cohesive running	Granular materials without cohesion are unstable at a slope greater than their angle of repose. When exposed at steeper slopes, they run like granulated sugar or dune sand until the slope flattens to the angle of repose	Clean dry granular materials. Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive raveling
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous liquid. The material can enter from the invert as well as the face, crown, and walls and can flow for great distances, completely filling the tunnel in some cases	Below the water table in silt, sand, or gravel with enough clay content to give significant cohesion and plasticity. May also occur in sensitive clay when such material is disturbed
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel	Highly pre-consolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite

In the UCSC system, soils are classified by grain size and divided into two major categories, coarse-grained soils (sand and gravels) and fine-grained soils (silt and clay). Coarse-grained and fine-grained soils are further classified according to grain size distribution and plasticity which, along with engineering properties, influence how soils behave for tunnel applications.

In addition to the UCSC system, the Tunnelman's Ground Classification System, developed by Dr. Karl Terzaghi (1950), further describes representative soil types and their predicted behavior for tunneling. The classification system, later modified by Heuer (1974), is shown in Table 1. In the Tunnelman's system, ground classifications range from firm to swelling and correspond to typical soil types above and below groundwater and their anticipated behavior.

Rock Mass Classification

As with soils, several characterization systems have been developed for tunnel applications in rock. Unlike soils, however, the primary controlling parameters that influence behavior include rock type and strength, spacing, condition, and orientation of discontinuities and *in situ* stresses. The following sections describe four commonly used classification systems used for tunnels in rock: (1) Terzaghi's rock mass classification, (2) rock-quality designation (RQD), (3) rock mass rating (RMR), and (4) quality index (Q).

One of the earliest classification systems for rock was developed by Terzaghi (1946). The classification system was developed as a method of classifying rock masses and evaluating rock loads based on qualitative assessments. Today quantitative systems are more widely used, but Terzaghi's system is still useful in describing general rock mass behavior.

The rock-quality designation (RQD) developed by Dr. Don U. Deere (1963) and Deere and Deere (1988) in the 1960s is a method of logging sound-drilled rock core to calculate and quantify the percentage of "good" rock in a core run. The percentage of good rock was used as an indicator of rock mass quality for tunneling and to assess tunnel support requirements. RQD today is used worldwide as a quantitative method of evaluating rock quality and is also widely used as one of the parameters in other more numerical rock classification systems. RQD can be defined as the percentage of rock core pieces 4 in. (10 cm) or greater in length divided by the total length of a core run expressed as a percentage:

$$\text{RQD} = \text{Sum of length of core pieces 4 inches or greater}/ \\ \text{Total length of core run} \times 100\%$$

Correlations between RQD, qualitative rock quality, and general tunneler's descriptions are provided in Table 2.

Tunnels, Table 2 Rock quality vs RQD

Rock quality	RQD (percent)	Approximate general tunneler's description
Excellent	90–100	Intact rock
Good	75–90	Massive, moderately jointed
Fair	50–75	Blocky and seamy
Poor	25–50	Shattered, very blocky, and seamy
Very poor	0–25	Crushed

Tunnels, Table 3 Rock mass classifications based on total RMR

Rating	Class	Description
81–100	I	Very good rock
61–81	II	Good rock
41–60	III	Fair rock
21–40	IV	Poor rock
Less than 20	V	Very poor rock

The rock mass characterization systems that are most commonly used in tunnel practice today include the rock mass rating (RMR) and the quality index (Q). The RMR system developed by Bieniawski (1989) and the quality index (Q) developed by Barton et al. (1974) provide overall comprehensive indices of rock mass quality for the design and construction of excavations in rock, such as tunnels.

The RMR system incorporates rock mass data regarding rock strength, RQD, discontinuity spacing, discontinuity condition, groundwater, and an adjustment for discontinuity orientation with respect to the excavation. These parameters are assigned numeric values based on their conditions, and the summation of the numeric values for all the parameters is the rating of the rock mass. The rock mass classifications based on the total RMR are shown in Table 3.

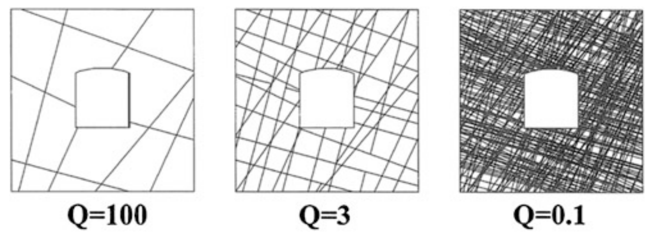
The quality index (Q), developed to estimate the roof support pressure that is required in an underground working, uses parameters similar to the RMR system to evaluate the stability that can be expected for excavation within the rock mass. One of the differences between RMR and Q lies in the assessment of the *in situ* stress state in the Q system by the use of the "stress reduction factor." The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is estimated from the following expression (Barton 2002):

$$Q = (\mathrm{RQD}/\mathrm{Jn}) \times (\mathrm{Jr}/\mathrm{Ja}) \times (\mathrm{Jw}/\mathrm{SRF}),\ \text{where}$$

Jn = joint set number
Jr = joint roughness number
Ja = joint alteration number
Jw = joint water reduction factor
SRF = stress reduction factor

Tunnels, Table 4 Q vs rock quality

Q	Rock quality
400–1,000	Exceptionally good
100–400	Extremely good
40–100	Very good
10–40	Good
4.0–10	Fair
1.0–4.0	Poor
0.1–1.0	Very poor
0.01–0.1	Extremely poor
0.001–0.01	Exceptionally poor



Tunnels, Fig. 1 Q-index rock quality

The general relationship between Q and rock quality is provided in Table 4 and illustrated in Fig. 1.

Results from both RMR and Q systems are used to evaluate parameters such as tunnel span, tunnel support, rock mass deformation, rock mass strength, and stand-up time.

A relatively new classification system, termed the geotechnical strength index or GSI (Marinos et al. 2006), captures the variability in geologic materials associated with faulting and extreme deformation associated with tunnel in rock. It is meant to provide reliable input data related to rock mass properties required as input for numerical analysis or closed form solutions for designing tunnels.

Groundwater

Groundwater is a major concern for tunnels contributing to loading on tunnel support and final lining and impacting ground behavior and ground stability. In soils, groundwater occurs within the pore spaces of the soil particles, and in rock, it occurs within the rock mass fractures and joints. Groundwater impacts can include settlement related to ground loss, inflows during construction, and sometimes hazardous working conditions. Groundwater also influences selection of appropriate methods and equipment for tunneling. Un-anticipated high loads due to water pressures can exert additional loads on tunnel support systems causing remedial actions and delays (Goodman et al. 1965). Inflows in tunnels can range from nuisance water to continuous or sudden inflows of thousands of gallons per minute. Figure 2 shows accumulation of groundwater from inflows within the invert of a TBM during excavation.

Tunnels, Fig. 2 Groundwater inflows during tunneling

Methods to control groundwater include dewatering, pre-excavation grouting, the use of watertight tunnel liners, and ground freezing, although ground freezing can be very costly and not economical in most cases.

Fault Considerations

Both active and inactive fault crossings can have impacts on design and construction of tunnels, (Goricki et al. 2006). For obvious reasons, it is best to plan around crossing active faults for tunnels. However, in many cases, active fault crossings cannot be avoided. Design issues to be considered where active faults are involved include:

- Avoidance – can tunnel alignments be planned away from active fault crossings?
- Displacement – what is the anticipated fault displacement and can the end product use (highway, water/sewer line, etc.) tolerate anticipated displacements?
- Recurrence interval – what is the probability of fault rupture over the life of the project and are owners and designers willing to accept the risk of fault rupture and damage to facilities?

Non-active faults can also impact tunnels. Fault crossing is often associated with weak, highly sheared, and broken rock, which may impact tunnel excavation and tunnel support. Groundwater is also commonly associated with faults, and fault zones can both act as groundwater barrier with differing pressures and groundwater levels on either side of the fault or a conduit for groundwater flow (Heuer 1995).

Gassy Ground

Gassy ground refers to potentially explosive or hazardous gases that can be encountered during tunneling. The two most common gases that affect tunnels are methane and hydrogen sulfide (H_2S). Methane is a naturally occurring gas associated with decaying organic matter or within sedimentary deposits. Methane forms an explosive mixture when mixed with air with approximately 5–15% of the mixture being methane. Hydrogen sulfide (H_2S) is a potentially deadly colorless gas often associated with methane. It is very poisonous, flammable, and explosive with a characteristic rotten egg odor.

Both of these gases require high-volume ventilation to dilute concentrations to safe levels and continual testing to determine if explosive or hazardous gases are present.

Tunnel Construction

Tunnel construction generally involves three operations: excavation, support of excavations, and muck removal. For the engineering geologist, the primary concerns are with excavation and support of excavations.

Tunnel Support

Based on the ground conditions, various ground characterizations, and project needs, tunnel designers can consider various options for tunnel support. Tunnel support refers to initial and final lining support systems used prior to, during, and after tunnel excavation. For tunnels in soft ground, both initial and final lining tunnel support systems are common, with the initial support intended to provide temporary support during excavation and the adequate support afterwards. Typical initial or temporary support systems used for soft-ground tunnels include:

- Steel ribs and wood lagging
- Steel liner plate
- Lattice girders
- Shotcrete
- Precast concrete segmental liners

Steel ribs and wood lagging have been used for initial tunnel support for decades and can be used for both circular and horseshoe-shaped tunnels. Steel ribs with or without lagging can also be used for rock tunnels where rock reinforcement is not required but some initial support is needed based on rock conditions. Figures 3 and 4 show a typical horseshoe-shaped tunnel supported with steel ribs and wood lagging and a circular tunnel in rock with steel ribs without lagging.

Precast concrete segmental liners are also commonly used for tunnels in soft ground and where tunnel boring machines (TBMs) are also required for excavation. Because concrete segmental liners are large and bulky, mechanized segment erectors or hoists are used to install the liners in thetail shield of a TBM. Precast segments can be used as single-pass or double-pass lining systems to support soil or rock.

Tunnels, Fig. 3 Typical steel ribs and lagging tunnel support in soft ground

Tunnels, Fig. 4 Steel ribs with no lagging tunnel support in rock

Rock tunnels have different support requirements than tunnels in soils. In some cases where tunnels are excavated in massive high-strength rock masses with little or no discontinuities, little or no tunnel support may be required. However, most tunnels in rock require some kind of support or reinforcement to support rock loads or mitigate against slab, wedge, or block failures. Common support systems in rock include:

- Steel ribs with or without wood lagging
- Rock reinforcement (rock bolts, rock dowels, rock anchors, etc.)
- Shotcrete
- Precast concrete segments
- Lattice girders

Steel ribs and lagging were discussed in the preceding section on soft-ground support. For rock tunnels, steel ribs are often installed in conjunction with shotcrete instead of wood lagging.

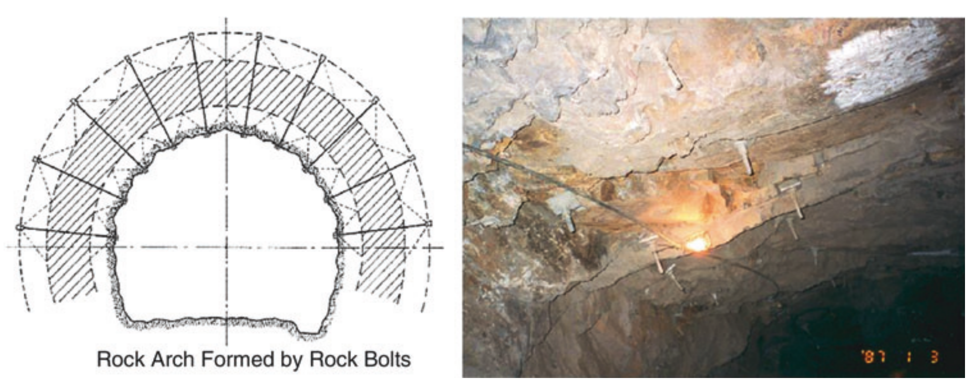
Rock bolts or dowels are used to hold loose slabs, wedges, or blocks in place. Rock bolts differ from dowels in that they are tensioned as soon as they are installed as opposed to dowels which are passive elements that require some ground movement to be activated. Figure 5 shows a typical rock bolt schematic and installation.

For rock tunnels supported by steel ribs, rock reinforcement bolts or dowels, or lattice girders, shotcrete can also be sprayed using a nozzle and can be applied around and between the support members.

Construction Excavation Methods

Tunnel excavation methods have been evolving over thousand years of years. Hezekiah's tunnel was excavated around 700 BC beneath Jerusalem with the use of simple hand tools. Since that time, many modern developments have occurred including the use of blasting in rock with explosives, the development of the tunneling shield and tunnel boring machines, and other mechanized tools such as road headers

Tunnels, Fig. 5 Typical rock bolt



Tunnels, Fig. 6 Miner loading explosives into a drill-and-blast tunnel heading

and hydraulic drills. Some of the more common methods of excavation are discussed in the following sections.

Drill and Blast

The modern drill-and-blast methods involve drilling a pattern of small holes in a tunnel face, loading them with dynamite, and detonating by controlled blasting. The blasted and broken rock is then removed from the heading. Figure 6 shows a miner loading explosives into drilled holes at the heading of a tunnel in rock.

Shield Tunneling

Shield tunneling methods were first used by Marc Isambard Brunel who patented the tunneling shield in 1818. The shield provided a protective compartment for miners working underground during tunnel construction. Since its first use, tunnel shields have evolved into sophisticated tunneling machines capable of rapid excavation in soft ground.

The main components of a typical tunnel shield include a cutting edge, a cylindrical shield, and a tail section in which tunnel support elements are assembled. The shield may be equipped with a digger at the front of the machine for excavation, or it may be open for the use of hand mining or other excavation methods. The cutting edge is often slanted to provide a canopy of support at the front end, and breasting tables or plates are often located at the front end of the machine for control of the ground at the face. Figure 7 shows a typical open-face "digger" shield with cutting edge, backhoe-like excavator, breasting table, and breasting boards.

Tunnels, Fig. 7 Typical open-face "digger" shield

Tunnels, Fig. 8 Rock TBM, Los Angeles redline tunnels

Tunnel Boring Machines

Tunnel boring machines (TBMs) have been used for decades and, like the tunnel shield, have continuously evolved for larger and more complex applications. For soft-ground applications, all TBMs are equipped with cylindrical shields similar to soft-ground shields. Depending on rock conditions, TBMs for rock can be equipped with or without shields. The main components of a TBM are the rotating cutter head, shield (or no shield), and hydraulic jacks or grippers for advancing the TBM.

Many different cutter head designs are available depending on the rock or soil conditions anticipated. For rock, harden steel disk cutters are designed based on a number of parameters including rock strength and spacing and condition of discontinuities. For sedimentary bedrock or soil conditions, the cutters are designed more like small spade-like cutters. A typical rock TBM is shown in Fig. 8.

NATM/SEM

The New Austrian tunneling method (NATM), also referred to as sequential excavation method (SEM) or shotcrete method, was developed in the 1950s when shotcrete was first used systematically to stabilize squeezing ground in a water diversion tunnel at the Runserau Hydroelectric Power Project in Austria (Sauer 1990). NATM/SEM enhances the self-supporting capacity of the rock or soil by mobilizing the strength of the surrounding ground. Typically, SEM tunnels, as the name implies, utilize a sequence of headings and benches that are excavated after adjacent surfaces are shotcreted to provide temporary support (Rabcewicz 1964, 1965). The excavations often utilize a road header, as shown in Fig. 9, or mechanized cutting head that can reach and cut various size faces and benches. Unique advantages of the method are flexibility in configuration and the advance of top headings, benches and inverts, ease of the use of spot bolts, spiling or steel sets in poor ground, and modifications of shotcrete mix design and enhancements. SEM or NATM also is preferred in areas of high seismicity and urban areas where access, vibration, and short tunnel lengths are common.

Tunnels, Fig. 9 Road header excavator head used in SEM tunneling, Caldecott fourth bore

Cut and Cover

The cut-and-cover method is used where there is shallow cover over the tunnel invert, but where an open cut is not desirable due to noise, exhaust, or aesthetic reasons. Cut-and-cover sections of tunnels frequently are utilized as options for portals or where utilization of space requires putting rail, highway, or drainage facilities underground. What is unique to cut and cover when compared with bored tunnels is that there are no headroom restrictions and the tunnel roof is fabricated with steel and concrete in management segments supported on vertical walls.

The normal construction sequence of cut-and-cover tunneling comprises establishing a rough invert subgrade to drill and set a series of soldier piles, driven sheet piles, secant piles, or CIDH piles to support the tunnel walls. In areas of high groundwater or soft ground, cement-bentonite panels can be placed between supporting vertical members, otherwise lagging or formed concrete walls, shown in Fig. 10, can be placed between the vertical members. Additionally, the walls can be

Tunnels, Fig. 10 Cut-and-cover tunnel under construction, Presidio Parkway main tunnel

supported and braced by wailers and struts. The roof of the tunnel can then be placed and final interior lining installed.

Another type of cut-and-cover tunnel is the immersed tube tunnel. These types of tunnels require dredging of a channel or bed along the seafloor, then floating tunnel segments to a position above, and then lowering them by a controlled sinking into place. The individual segments are then welded together, covered with fill, and then pumped dry. The Bay Area Rapid Transit (BART) tunnel in California is a well-known immersed tunnel, with the world's deepest being the recently opened Busan-Geoje Fixed Link in Seoul, Korea.

Monitoring

Tunneling involves the removal of Earth material to leave an opening useful for conveyance of materials, equipment, or people. The resultant openings, if left unsupported, would cause vertical settlement and lateral movement. The purpose of monitoring is to measure the rate and amount of movement during construction and to compare these to calculated and/or allowable values. The sophistication and density of the monitoring is directly related to the sensitivity and location of adjacent existing structures, that is, urban settings. Tunneling near an active rail transit tunnel or beneath an historic district or heavily loaded buildings requires a combination of surface settlement markers, extensometers, and tiltmeters.

Typically, a baseline survey of surface monuments is established. Subsurface measuring points, groundwater piezometers, tiltmeters, and slope inclinometers are also installed, the system is tied to a network, and a monitoring frequency is established based on tunnel advance rates and alignment crossings. Thresholds for movement are established, and if these are exceeded, tunneling is either halted, or mitigative measures to reduce movement are implemented. Additional monitoring points can be established inside the tunnel to measure any divergence or convergence of tunnel inverts and lining. Recent laser technology has leaned to very high precision of movements from within the tunnel, in particular on shotcrete lining application.

Gaseous Conditions

Tunneling in ground containing shallow petroleum, in either liquid or gaseous phases, can result in high concentrations of explosive volatile compounds. A similar condition can occur in oil fields where wells, either active or abandoned, are crossing or are intercepted. Even shale contains measurable concentrations of hydrocarbons that can cause explosivity hazards or displacement of oxygen in the breathing zone.

Equipment used to monitor dangerous levels of gas in the tunnel is deployed within the tunnel and sometimes on personnel working in the tunnel. Warnings of concentration exceedance are then communicated to all personnel in the construction zone. Mitigation for gaseous conditions includes restrictions on equipment with open combustion or possible sparking and having personnel wear oxygen-rescue devices and having rescue chambers close by.

Summary

Tunnel construction involves understanding of the geologic ground conditions before and after tunnels have been constructed. The benefit of tunneling versus surface conveyance is directly related to the diameter, depth, length, and sensitivity of the route the tunnel alignment takes. All of these factors need to be evaluated by engineering geologists, tunnel engineers, and tunnel contractors during design and construction.

Many methods of tunnel excavation have been developed and include drill and blast, cut and cover, mechanized boring, and sequential excavation. The equipment and personnel that operate them need to consider groundwater, mixed ground conditions, settlement, and gaseous conditions. Many guidance documents and handbooks have been developed that provide established procedures and protocols for tunnel design and construction (Bickel et al. 1996; Maidl et al. 2013, 2014).

Tunnel support, both temporary and permanent, requires a well-written Geotechnical Baseline Report to understand the range of ground conditions that will be encountered. Adapting to changing geologic conditions requires experience and contractual flexibility to allow safe construction and long-term performance.

Cross-References

▶ Blasting
▶ Cut and Cover
▶ Dewatering
▶ Drilling
▶ Engineering Properties
▶ Excavation
▶ Extensometer
▶ Faults
▶ Field Testing
▶ Gases
▶ Geology
▶ GIS
▶ Ground Anchors
▶ Groundwater
▶ Grouting
▶ Instrumentation
▶ Liners
▶ Mining

▶ Modelling
▶ Monitoring
▶ Piezometer
▶ Pipes/Pipelines
▶ Pressure
▶ Residual Soils
▶ Rock Bolts
▶ Rock Mass Classification
▶ Rock Mechanics
▶ Sand
▶ Sedimentary Rocks
▶ Shotcrete
▶ Stress
▶ Tiltmeter
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▶ Wells

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V

Vegetation Cover

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Synonyms

Vegetation; Vegetative community

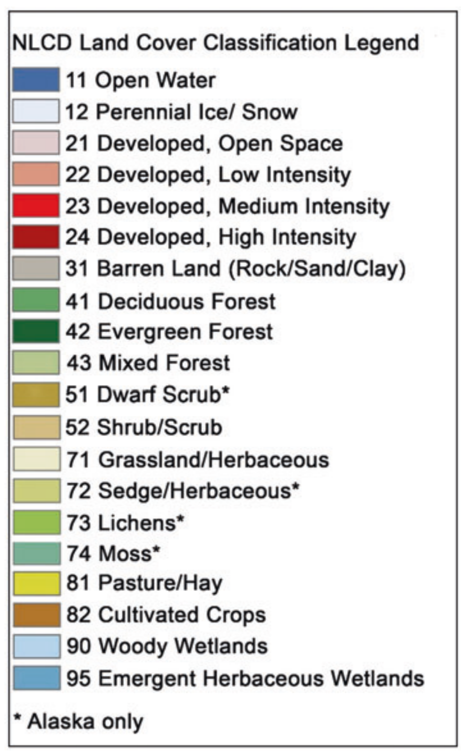
Definition

Vegetation cover is a land cover type that describes the predominant vegetation present for a particular area of the Earth's surface (NOAA 2015).

Classifications of land cover types, developed nationally and internationally by both public and private entities, predominantly describe the types of vegetation that may be present. For example, the legend for the 2011 land cover map for the national atlas of the United States has twenty subdivisions (Fig. 1) (Homer et al. 2015). Thirteen of the land cover types are defined by specific vegetation types representing the canopy vegetation at a spatial resolution of 30 m.

Vegetation cover is generally derived from multispectral imagery obtained using space-based platforms (Homer et al. 2015). Both the capability of the imagery and the purpose for the land cover classification will constrain the degree to which vegetation and other land cover are subdivided for a particular classification. Vegetation land cover data are stored and manipulated in a database format or within geographic information system (GIS) programs for the generation of various map products (Homer et al. 2015). Repeated classification of the same area over a period of time can provide information about how specific vegetation cover types are increasing or decreasing and permits calculation of rates of change (Homer et al. 2015).

At its simplest, vegetative cover is a factor that must be addressed when accessing and constructing engineering works. The application of different site investigation techniques may be hampered by the presence of a particular vegetation cover. Vegetative cover or its absence influences a variety of geomorphic processes affecting the function or implementation of engineering works. For example, vegetation influences surface erosion rates and water runoff into natural channels and artificial drainage systems. For this reason, documenting the



Vegetation Cover, Fig. 1 Cover types, including many vegetative ones, identified in 2011 national land cover database by the Multi-resolution Land Characteristics Consortium (MRLC) (http://www.mrlc.gov/ Accessed December 16, 2015)

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vegetation cover change due to wildfire can be important to protecting engineering works and public safety (Clark 2013). Woody vegetation is especially important to slope stability through its influence on soil moisture and soil strength provided by root networks (Sidle and Ochiai 2006). The response to vegetation change documented from a major typhoon in Thailand reflects the important link between vegetation, slope stability, and public safety (De Graff et al. 2012).

Cross-References

- ► Databases
- ► Land Use

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Velocity Ratio

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Definition

The relationship of elastic longitudinal wave (Vp-waves) velocity to shear wave (Vs-waves) velocity or the relationship of elastic shear wave (Vs-waves) velocity to longitudinal wave (Vp-waves) velocity.

Characteristics

In engineering seismology, the velocity ratio Vp/Vs (TAU parameter) is considered as an indicator of the changes in stress state of rocks during the process of developing strong tectonic and volcanic earthquakes. It is well-known that deep fault zones, named gradiental zones, display rapid changes in the ratio Vp/Vs with a change of the anomalies' sign (positive or negative). These can be considered as seismogenic zones. Relatively low values of Vp/Vs ratio indicate increases in activity in the magmatic chambers of volcanoes during their activization. The ratio of Vp-waves velocity to Vs-waves velocity (Vp/Vs) is used in engineering seismology and engineering geology (Slavina et al. 2015).

In engineering geology, the velocity ratios Vs/Vp and Vp/Vs are used for definition of physical-technical characteristics of rocks. If the values of both ratios are known, it is possible to define dynamic modules of elasticity – Poisson's ratio μ, Young's modulus E, and the shift modulus G. These characteristics are of interest for carrying out engineering surveys for construction. To define the Poisson's ratio μ, it is necessary to know Vs/Vp or Vp/Vs ratios only; for the definition of Young's modulus E and the shift modulus G it is necessary to know also the density of rocks. For the definition of static modulus necessary for engineering calculations, empirical ratios or special diagrams are used (Nikitin 1981; Bondarev 1997).

Cross-References

- ► Earthquake
- ► Poisson's Ratio
- ► Young's Modulus

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Vibrations

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Synonyms

Chatter; Concussion; Flutter; Shaking; Trembling; Tremor; Vibrating

Definition

(*latin.* Vibratio – oscillations)

1. Continuous slight to strong shaking movement: the vibrations from earthquakes;
2. Continuous mechanical oscillations with frequencies from 0.5 Hz up to 40 kHz generated from various sources and passing into soils and rocks.

Modes of Vibration

Vibrations are caused by earthquakes (Longman Dictionary of Contemporary English 1978) but also by human activities. Intentional vibrations generated by vibrating machines are used in construction, mechanical engineering, during extraction of hydrocarbons, etc. Harmful vibrations are produced by movements of vehicles, working of engines, turbines, piling etc. Strong vibration can lead to malfunction and destruction of machines and constructions. Effects of vibrations on organisms depend on frequency and amplitude fluctuations leading to either salutary (therapeutic applications) or harmful (vibration-induced disease) outcomes.

Vibration is one of the components of technogenic physical (power) emissions on the geological environment in cities. Sources of technogenic vibrations are units and equipment of industrial enterprises, construction, and mining as well as vehicles, surface and underground rail transport, etc. Acting on the geological environment, these sources cause mechanical oscillations propagating for distances up to 80–100 m in the ground (Zhigalin and Lokshin 1987, 1991) when the vibration source is situated at the ground surface the types of oscillations are similar to Rayleigh surface waves. The depth of propagation of a "vibrating wave" is 10–15 m. In the case of buried sources, the greater part of the energy is transferred by volumetric longitudinal and shear elastic waves.

Vibration in Engineering Geology

Vibration is an important factor in engineering geology in respect to the mechanical stability of the soils that form the base or the containing environment of engineered constructions. The influence of vibration on soil thickness can lead to reduction of friction and coupling forces keeping soils at the initial equilibrium state. As a result, adverse geological processes may be activated. For instance, strong vibrations can disrupt the structure of thixotropic soils with loss of their strength.

Dynamic impacts on the geological environment lead to compaction of unconsolidated and weakly compacted soils. This is especially important in areas occupied by artificial (technogenic) soils. In cities, buildings located along highways with intensive traffic experience high vibrational impacts. As a consequence non-normative and continuous subsidence of the ground with damage to buildings and destruction of soil structure may be observed (Table 1).

Land Improvement

During land improvement, vibration is used to make artificial changes to the properties of natural soils (compaction and strength increase) to prepare them for industrial, civil, transport and other types of construction (Voronkevich 2005). Induced vibrations cause the modification of ground properties *in situ* by mechanical compaction. This is achieved by a mechanical compaction. Mechanical compaction can be used in both natural and artificial (technogenic) soils.

Vibrations, Table 1 Vibration impacts on the geological environment and engineering constructions (Zhigalin and Lokshin 1991)

Level of vibration				Consequences of vibration impacts
Vibration velocity		Vibration acceleration		
10^{-3} m/s	dB	m/s^2	dB	
0.4	78	0.05	44	Insignificant subsidence (up to 2 mm/year) of the foundations of buildings in weak soils
1.2	87	0.15	54	Insignificant subsidence (up to 2 mm/year) of the foundations of buildings in solid soils; minor damage to old buildings is possible
2.4	93	0.30	59	Continuous subsidence (3–5 mm/year) of the foundations of buildings in weak soils
4.0	98	0.50	64	Significant (> 5 mm/year) continuous subsidence of the foundations of buildings in weak soils; continuous subsidence (3–5 mm/year) of the foundations of buildings in strong soils
8.0	104	1.00	70	Above this level of vibration, damage to stone buildings with overlapping concrete structures is possible
12.0	108	1.50	74	Above this level of vibration, damage to buildings constructed with reinforced concrete is possible
15.9	110	2.00	76	Maximum compaction of dry sands (without static loading) is possible
78.0	124	9.80	90	Maximum compaction of saturated sands (without static loading) is possible
117.0	127	14.70	93	Destruction of the structure of dispersive soils is possible

Cross-References

- ► Compaction
- ► Earthquake
- ► Ground Preparation
- ► Ground Shaking
- ► Liquefaction

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Viscosity

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Definition

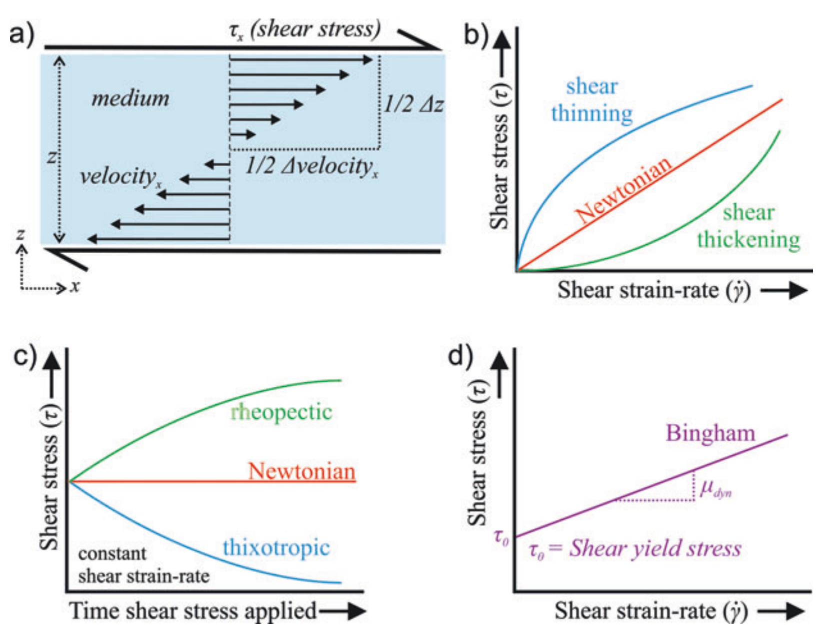
Properties of deformation and flow of materials under stress.

The science of rheology, concerning the deformation of the ground, has three basic aspects: viscosity, elasticity, and plasticity. Viscosity is discussed here. Elasticity and plasticity are considered in Mechanical Properties.

Viscosity is the resistance to gradual deformation of a medium under shear stress. The viscosity can be independent ("Newtonian" or "ideal-viscous" media) or dependent ("Non-Newtonian" media) on the shear strain-rate and time. Temperature and confining pressure may also be of influence. Gases, fluids, and many ground materials behave viscously and also "hard" rocks which may (seem to) behave viscously over long timespans under high confining stress or temperature.

Ground materials are diverse and may be gases, fluids, or solids (i.e., minerals, grains, and aggregates of grains or minerals or some combination), and any mixture of these and also include man-made ground, such as fills and waste dump material. Ground is commonly differentiated into soil and rock; soil being an aggregate of loose or weakly bonded particles, and rock consisting of particles cemented or locked together, giving rock a tensile strength. Soil and rock are, by some, differentiated based on a compressive strength difference with soil being weaker than 1 MPa and rock being stronger. A differentiation is made between "intact" and "discontinuous" ground, that is, ground without, respectively, distinct planes of mechanical weakness (discontinuities) such as faults, joints, bedding planes, fractures, schistosity. A groundmass consists of (blocks of) intact ground with discontinuities, if present.

Viscosity is the resistance to gradual deformation of a medium and relates to the difference in shear strain-rate (also "*shear velocity*" or "*rate of shear deformation*") in a flowing medium (Fig. 1a). Viscosity is due to the friction between neighboring particles that move with different velocities and is dependent on temperature and confining pressure for most media.



Viscosity, Fig. 1 (**a**) Viscosity in laminar flow; (**b**) viscosity dependent on shear strain-rate; (**c**) viscosity dependent on time of shearing; (**d**) Bingham plastic; solid under low and liquid under high shear stress (**b**–**c** modified from Barnes (2000) and Rao (2007))

Newtonian or Ideal-Viscous Media

Equation 1 gives the mathematical formulation for the "*dynamic viscosity*" (with symbol μ_{dyn}, μ or η) of Newtonian media in which the viscosity is independent from the shear strain-rate and from time, which is valid for air, gases, and some liquids such as light-hydrocarbon oils and water under "normal" conditions, i.e., engineering conditions on or near the Earth surface (Barnes 2000) (Fig. 1a–c).

$$\tau_{zx} = \mu_{dyn\ x} x \frac{\Delta velocity_x}{\Delta z} = \mu_{dyn\ x} x \dot{\gamma}_{zx} \tag{1}$$

τ_{zx} = shear stress in x direction on plane with normal in z direction [Pa]
$\mu_{dyn\ x}$ = dynamic viscosity in x direction [Pa · s]
$\Delta velocity_x/\Delta z = \dot{\gamma}_{zx}$ = shear strain–rate = velocity difference in z direction due to shear stress in x direction [1/s]

The "*kinematic viscosity*" (with symbol μ_{kin} or v) is the dynamic viscosity divided by the density of the medium (ρ):

$$\mu_{kin} = \frac{\mu_{dyn}}{\rho} \tag{2}$$

μ_{kin} = kinematic viscosity [m^2/s]
μ_{dyn} = dynamic viscosity [Pa · s]
ρ = density [kg/m^3]

Units for viscosity in Eqs. 1 and 2 are in SI units, but also often used are the non-SI units "*Poise*" (= 0.1 Pa·s) for dynamic viscosity and "*Stokes*" (= 1×10^{-4} m^2/s) for kinematic viscosity. The viscosity of Newtonian media mostly decrease with increasing temperature and increase with increasing confining pressure. Water is an exception as the viscosity of water with a temperature below 32 °C, and a confining pressure less than 20 MPa decreases with confining pressure. Liquids lose their Newtonian behavior and become non-Newtonian for very to extremely high shear strain-rates, for instance, mineral oils at more than 1×10^5 s^{-1} and water likely at more than 5×10^{12} s^{-1} (Barnes 2000). Table 1 gives indicative viscosity values for various materials.

Non-Newtonian Media

Most fluids, many Earth materials, and mixes of ground materials with gases and liquids behave as non-Newtonian media. Non-Newtonian media have a viscosity that is dependent on the shear strain-rate, history of shear strain-rate, shear stress, rate of stress change, or on other factors related to the stress environment apart from being dependent on temperature and confining pressure. Non-Newtonian media are subject to extensive research and new features are regularly reported; a very brief and therefore necessarily incomplete description follows. The different behavior is illustrated with typical examples; however, various examples behave differently under different conditions or depending on details of constituents, for instance, yoghurt may behave with the characteristics of a thixotropic or as Bingham plastic medium and many drilling muds behave as shear thinning, thixotropic or Bingham media.

In "*shear thinning*" media, the viscosity is independent of time, but the viscosity decreases with increasing shear strain-rate, for instance, quick sand, some high-temperature volcanic lavas, some drilling muds, even tomato sauce (Fig. 1b). In "*shear thickening*" also independent of time, the viscosity increases with increasing shear strain-rate, for example, cornstarch with water (oobleck). In "*thixotropic*" and "*rheopectic*" (or "*antithixotropic*") media, the viscosity is dependent on the length of time of shearing (Fig. 1c). The longer the medium is agitated or stressed either the more the viscosity decreases or increases. Some clay bentonite and montmorillonite), Portland cement, some drilling muds, and yoghurt are examples of thixotropic media. Some bentonite mixes, gypsum paste (and even the cream on top of an ice-cream) exemplify rheopectic media. "*Bingham plastics*" behave solid under low shear stress and fluid under high shear stress, the boundary being the "shear yield stress" or "yield-point" (Fig. 1d). Low-temperature lavas, some pyroclastic flows, some drilling muds, cement slurry, yoghurt, and tooth-paste are examples.

Effective Viscosity

Effective viscosity (also "apparent viscosity") of a fluid is often defined as the viscosity following a Newtonian medium (Eq. 1) whatever the character of the viscosity, that is, the effective viscosity is then the viscosity of an imaginary Newtonian fluid that gives the same shear stress at the same shear strain-rate under particular conditions and at particular values or ranges of values of shear stress and shear strain-rate. Note that the effective viscosity value is then only valid under the same conditions and the same values or ranges. This definition is commonly used for drilling fluids (Baker Hughes 2006), but also used for other fluids, semi-solids, and solids.

Effective viscosity of a Newtonian medium is also used for semi-solids and solids for which it is not known whether the behavior is really viscous or is (partly) non-viscous such as the flow of ground and deformation of geological materials over long geological time spans. These show a behavior analogous to viscosity even if the materials are not viscous in a strict physical sense (Petford 2009).

Viscosity, Table 1 Indicative (effective) dynamic viscosity values

Medium	Dynamic viscosity (effective) (Pa•s)	
Gases		
Methane[1, 2, a]	11.1	$\times 10^{-6}$
Carbon dioxide[1, 2, a]	15.0	
Radon[1, 3, b]	18	
Air[1, 2, a]	18.5	
Pyroclastic flow[c]	0.015[4] to 90,000	$\times 10^{-3}$
Fluids		
Gasoline[3, d]	0.5	$\times 10^{-3}$
Water (fresh)[3, a]	1.002	
Water (sea)[3, a]	1.085	
Crude oil (light to heavy)[3, e, f, g]	5 to 1,000	
(Semi-) solids		
Mudflow[5, g] (soil with LI: 0.3 to >6)	0.007 to 500,000	$\times 10^{0}$
Municipal solid waste[6, h]	0.1 to 10	
Crude oil (Canada; extra heavy to tar sand/bitumen)[3, i]	5 to >1,000	
Lava flow[7, j]		
Basalt (temperature 1,400 to 1,100 °C)	0.01 to 0.1	$\times 10^{3}$
Andesite (temperature 1,300 to 1,000 °C)	0.1 to 10	
Rhyolite (temperature 1,000 to 800 °C)	10 to 10,000	$\times 10^{6}$
Ice (Greenland)[8, k] (temperature 0 to −30 °C)	1 to 100,000	$\times 10^{12}$
Tin[9, l]	0.1	$\times 10^{15}$
Marble[9, l]	0.1	$\times 10^{18}$
Rock salt[10, 11, m, n]	0.1 to 100	
Steel[9, l]	10	
Glass[9, l]	100	
Rock mass[11, 12, o, p]	1 to 100,000	

Notes: [1] At 100 kPa, [2] At 27 °C, [3] At 20 °C, [4] Hot air/gas, [5] Depending on Liquid Index, [6] Back calculated from waste dump flows, [7] Indicative only, heavily depending on composition, temperature, water, and gas content; values per range of temperatures typical for the type of lava, lower values for higher temperatures per range, [8] Depending on ice type and burial time, etc., [9] Room temperature, [10] Depth <5 km at 20–200 °C, [11] Indicatiove only, [12] Depending on rock type and stress configuration; high values for 3 km depth below Earth surface, lower values depth >30 km at temperatures 100–900 °C

Data: [a] CRC (2013), [b] Akbari and Mahmoudi (2008), [c] Yamamoto et al. (1993), [d] Green and Perry (2008), [e] Crude Quality Inc (2017), [f] Meyer and Attanasi (2003), [g] Widjaja and Hsien-Heng Lee (2013), [h] Huang et al. (2013), [i] Dusseault (2001), [j] Lesher and Spera (2015), [k] Marshall (2005), [l] Barnes (2000), [m] Chemia et al. (2009), [n] Van Keken et al. (1993), [o] Bürgmann and Dresen (2008), [p] Bills et al. (1994)

Ground and Viscosity

Viscosity governs the flow of gases and liquids through the ground via the small pores and narrow channels between the pores. Gases with very low viscosity flow generally very easily, water considerably less easily because of the higher viscosity and crude oil, with a greater viscosity than water, flows much less easily (Table 1). Ground consisting of loose particles ("soil") can also flow, for example, a soil with relatively little water and high viscosity on a slope slowly moving downwards under gravity or more quickly moving with more water and lower viscosity as a "mudflow." A high-temperature mix of gases and particles may move very fast on a slope of a volcano such as a pyroclastic flow with very low viscosity. Semi-solids and solids such as ice, rock salt, and rock masses have very high to extremely high (effective) viscosity and are immobile in day-to-day perception. However, over long time periods (e.g., ice in glaciers) or millions of years (rock masses) flow occurs. Semi-solids and solids are likely to have a strong anisotropic viscosity due to orientation of minerals, particles, and discontinuities (Hansen et al. 2016; Petford 2009).

Summary

Viscosity is an important feature in engineering geology. Historically elastic and plastic deformation of solids and viscous behavior of gases and fluids were standard parts of engineering geology. However, it is now evident that many processes on and in the Earth's surface can be (better) described with viscous behavior. This new, improved understanding of processes has opened new areas of study in the Earth sciences.

Cross-References

► Mechanical Properties

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Voids

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Definition

Underground open spaces or cavities may be of natural or man-made origin. Natural structures include caves, dissolution and collapse cavities in soluble rocks, cambering fissures (or gulls), open fault cavities, and lava tubes. Man-made voids include all the different types of mines, habitation, religious and storage spaces, military excavations, tunnels, and shafts.

Introduction

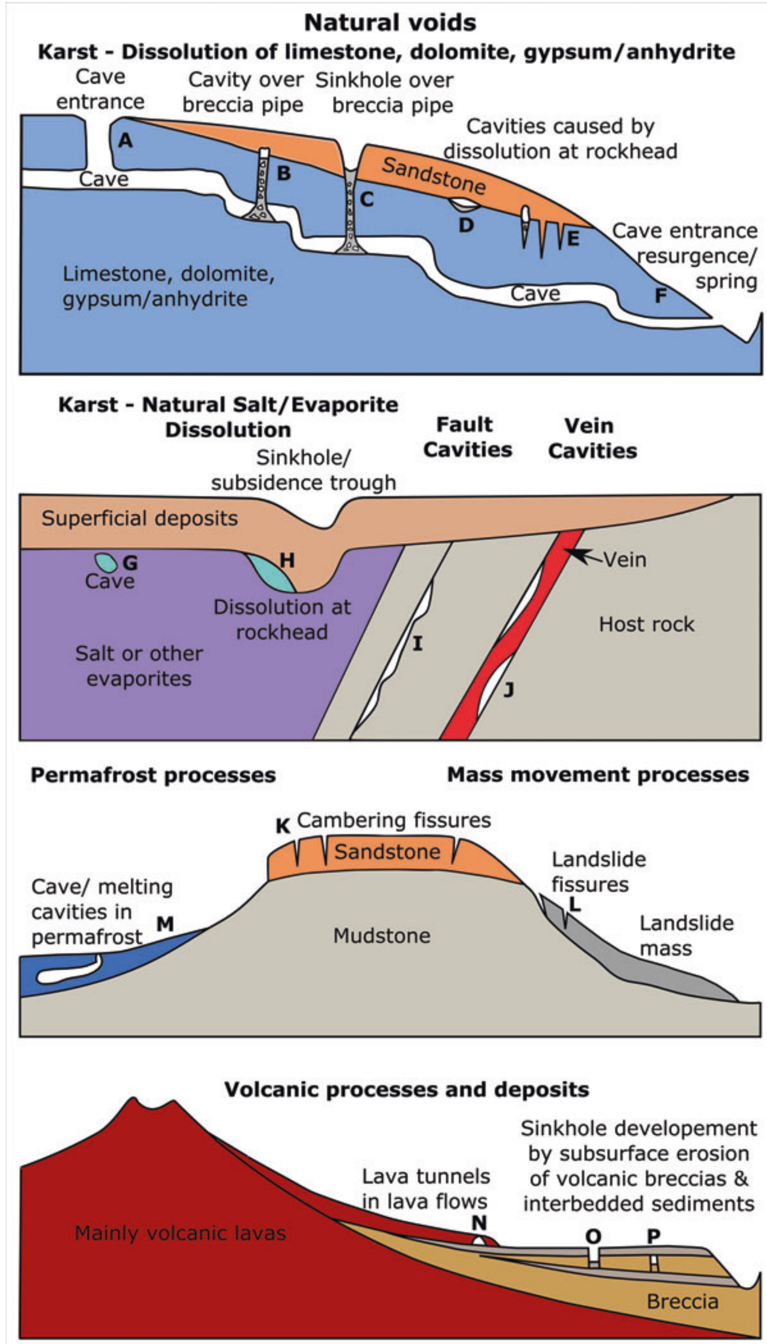
Voids or cavities are open spaces in ground that are commonly encountered as unforeseen ground conditions in engineering geology. In 2012 Donnelly and Culshaw proposed a method for the classification of natural and man-made voids, based on their mode of formation which was presented and published in BS5930 (2015). When voids are not foreseen in engineering geology and construction, they can pose a hazard. In tunneling and mining, they can represent an inrush, flooding, or gas explosion hazard. Where they are present on construction sites, they may result in unacceptable subsidence or collapse. In hydraulic structures (dams and tunnels), they can lead to structural compromise or failure. Voids may form naturally (Fig. 1) or be man-made (Fig. 2 and Table 1) (British Standards Institution 2015, Table F.1). Understanding the differences between natural and man-made voids helps to characterize their geometries, distribution, and likely associated hazards. The void type, size, evolution, engineering geology and geotechnical behavior of the rock mass, hydrogeology, hydrochemical setting, and depth determine the best investigation, remediation, or mitigation methods.

Natural Voids

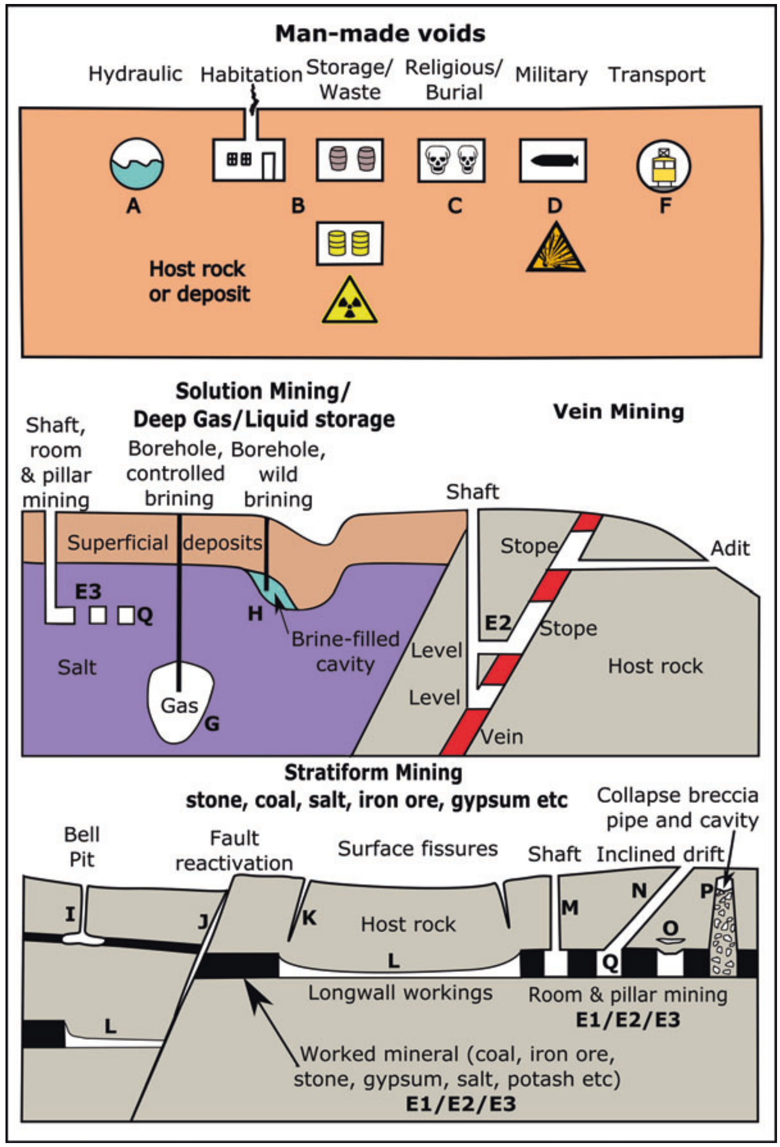
The most common natural voids occur in soluble or karstic rocks (Fig. 1 A–F). In order of increasing solubility (and dissolution rates), from low to high the common soluble rocks are dolomite ($MgCa(CO_3)_2$) and limestone ($CaCO_3$), plus the evaporites including gypsum/anhydrite ($CaSO_4.2H_2O/CaSO_4$), halite, or rocksalt (NaCl) (Gutiérrez et al. 2008; Warren 2016). Natural karstic voids range in size from those widened by dissolution generating fissures a few millimeters wide to enormous cave systems with volumes of millions of cubic

V

Voids, Fig. 1 Types of natural voids in various host situations



meters (Ford and Williams 2007). The most common are cave systems caused by the downward passage of water in unconfined conditions flowing to springs/resurgences. However, deep acidic and/or hydrothermal water flow in confined conditions can produce hypogene cave systems. Various origins and host rock structures (including rock mass discontinuities such as bedding, jointing, faulting, and folding) produce complex cave systems both in plan and profile. In addition, cavities can form by cave roof failure and upward migration of breccia pipes (Fig. 1B, C) forming sinkholes (Waltham et al. 2005). Roof failures in coal mines can show breccia pipe propagation of up to 20 times the cavity height (Dunrud 1984), but the migration height is variable. It can be less, but in soft materials, or with basal erosion, a cavity can continue upward through much greater thickness and produce very large sinkholes (e.g., Xiaozhai Tiankeng in China). Cavities also occur by dissolution



Voids, Fig. 2 Types of man-made voids. Designations *A, B, C, D, E1, E2, E3*, and *F* are noted after Parise et al. (2013) in Table 1; other letters are for identification only in the present text Investigation of voided ground

of the rock surface at rockhead beneath superficial deposits (Waltham et al. 2005) (Fig. 1D, E) or by downward washing of soils into cavities. On a larger scale, salt dissolution at rockhead can produce voids beneath a thick cover of surficial deposits or a bedrock aquifer (Fig. 1H). In arid areas, caves may also develop by downward movement of water through salt deposits (Fig. 1G). In addition to dissolving, some rocks also expand; anhydrite expands considerably on hydration to gypsum forming near-surface swelling caves.

Voids related to tectonic structures are relatively uncommon but are most likely within faults and mineral veins (Fig. 1I, J). At the surface, such cavities are generally not a problem, except where they are opened by fault reactivation (Fig. 2J). Underground, they are a problem forming conduits for water and gas into tunnels.

Coastal sea caves are common in all rock types due to wave erosion. Voids are also common in breccia and boulder conglomerate deposits. The washing out of the matrix from a volcanic breccia to local springs produced large sinkholes in Guatemala City (Fig. 1O, P). These were triggered by surface water, leakage from water, and sewerage infrastructure. Soil piping and voids can also be caused by the natural or induced washing away of sand deposits, both in natural situations and especially in hydraulic structures such as dams. Pipes and voids may also occur in peat (Donnelly 2008). Lava tunnels on the flanks of volcanoes can also pose a hazard to construction (Fig. 1N). Landslide mass movement and cambering may cause fissures and voids (Fig. 1L), which may be open or covered. Other rock types, including sandstone, can dissolve producing pseudokarst cavities and sinkholes (Ford and Williams 2007; Waltham et al. 2005); loess, lateritic, gypsiferous, and saline soils and permafrost (Fig. 1M) can also develop voids and be problematical.

Voids, Table 1 A–G classification of man-made voids based on Parise et al. (2013) and the Donnelly and Culshaw classification in British Standards Institution – BS5930 (2015, Table F.1), [1]brine cavities after Cooper (2002), and [1,2] not in Parise et al. (2013)

Classification of Parise et al. 2013 A–F and subcategories	Man-made voids
A – hydraulic underground excavations	A1 drainage; A2 water inception structures; A3 underground water ducts; A4 cisterns; A5 wells; A6 hydraulic distribution works; A7 sewers; A8 ship and boat canals; A9 ice wells/snow houses; A10 tunnels or ducts of unknown function
B – hypogean civilian dwellings	B1 permanent dwellings; B2 temporary shelters; B3 factories; B4 warehouses, stores, cellars; B5 underground silos; B6 stables/animal shelters; B7 pigeon houses; B8 apiaries; B9 any other kind of civilian settlement
C – religious excavations	C1 temples, wells, shrines, churches, etc.; C2 burial places
D – military and war excavations	D1 defensive works; D2 galleries and passages; D3 mine and countermine tunnels; D4 firing stations; D5 stores; D6 accomodation and command infrastructure; D7 civilian war shelters
E – mines	E1 quarries ([+]also called stone mines) – stratabound
	E2 metal mines – vein and stratabound
	E3 other mines, including coal mines, salt, gypsum mines etc. – mainly stratabound
	E4 nonspecific exploration tunnels
	E5 underground vegetable production
	[2]Former mines used for storage
	[2]Mines created for waste disposal and radioactive waste disposal facilities
	[2]Mining-induced fault reactivation
F – transport excavations	F1 tunnels for vehicles or pedestrians
	F2 transit works, nonmilitary
	F3 railway and tramway tunnels
	F4 non-hydraulic wells and shafts
G – other works	[2]Telecommunications tunnels
	[2]Boreholes
	[1]Dissolution caverns: brine caverns for salt, oil, gas storage, plus pressurized air power plant storage
	[1]Dissolution channels, near-surface brine runs

Man-Made Voids

The near surface in many urban areas is riddled with known and forgotten man-made voids (Table 1 and Fig. 2) classified into broadly similar categories by Parise et al. (2013) and by Donnelly and Culshaw in British Standards Institution – BS5930 (2015, Table F.1). They range from water supply tunnels (Qanats) dating back 3000 years to catacombs, storage and habitation excavations, military tunnels and stores, transportation tunnels, sewers, and infrastructure tunnels (Table 1 and Fig. 2A, B, C, D, F).

Many urban and rural areas are undermined by mines for metalliferous deposits and industrial minerals including sandstone, sand, limestone, gypsum, pozzolana, chalk, flint, stone, coal, iron ore, and salt to mention a few (Fig. 2E1, E2, E3). Clearly, the list of minerals and geographic locations is extensive; if there is a useful mineral or rock, even near surface beneath a town, it is likely to have been worked. This has left a legacy of voids and potentially unstable ground that needs to be identified, investigated, and mitigated. The majority of these industrial rocks, minerals, and fuel minerals are stratiform and were (or still are) worked by shafts, inclined drifts, and near-horizontal adits (Fig. 2M, N). Older small workings were commonly extracted from bell pits or dene holes (Fig. 2I). Larger workings are extracted mainly by room and pillar working, though longwall working is generally the favored method for deep coal working (Fig. 2L, Q). Longwall workings tend to produce subsidence bowls, which may have marginal fissures and fault reactivation causing open fissures and steps in the ground surface (Fig. 2K, L) (Donnelly 2009). Room and pillar workings may collapse by roof failure causing a breccia pipe to migrate toward the surface and break through as a crown hole (Fig. 2Q, O, P). In deeper mines, these collapses may choke before reaching the surface. (Fig. 2P).

Mining for valuable or precious metals also dates back to antiquity. Many of the metalliferous deposits occur in veins, whose extraction is largely by shafts or adits with removal of the vein from above and below to form stopes; these may emerge at or be very close to the surface forming a subsidence hazard (Fig. 2E2).

Early salt extraction used natural brine springs. Later, boreholes and pumps were installed (wild brining) producing underground cavities (Fig. 2H) and significant subsidence (Cooper 2002). Modern salt extraction is by pillar and stall mining (Fig. 2E3, Q) or by controlled brining from large deep dissolution cavities, generally at many hundreds of meters depth (Fig. 2G). Commonly, these cavities are reused for gas and waste storage, though some are made specifically for gas storage. Not all "controlled" cavities have proven successful and notable collapses have occurred in the USA (New Mexico and Texas) (Johnson 2003) and UK (Preesall and Teesside).

Triggering Mechanisms for Void Collapses

Ingress of water is by far the most common triggering mechanism, and spates of collapses forming sinkholes (over natural and mining cavities) have been induced by heavy rainfall and flood events. Burst water pipes or leaking sewers also trigger collapses. Road and hard-standing drainage into gulleys and French drains can allow water infiltration and subsidence along the drainage system. Engineering and irrigation can also induce voids to collapse by changing the groundwater levels due to groundwater abstraction, dewatering, and recharge. Vibration is another triggering mechanism particularly during engineering or earthquakes, or next to roads and railways, but also during borehole investigation and subsequent construction (e.g., piling) and mitigation.

Many voids have a pattern related to their underlying cause, such as a joint-controlled cave system, a vein or the pattern of room, and pillar workings. The related subsidence pattern gives information about the geometry of the cause and the area's susceptibility to collapse. The pattern can commonly be determined from topographical surveys, air photograph analysis, and LiDAR interpretation. Boreholes and probing have traditionally been used to detect and locate voids, but drilling stands very little chance of encountering the target without knowledge of the likely void location derived from previous studies or a geophysical investigation (Table 2). For borehole investigations, the automated recording of penetration rates can help with the assessment of voided ground. It is important that investigation holes are properly grouted; otherwise, they can aggravate the problem they are investigating.

Mitigation of Voids

Where potentially unstable voids are found near the surface, induced collapse and filling is a simple cost-effective mitigation method. Where coal (or other mineral) remains at shallow depth as pillars surrounded by partially collapsed ground, complete excavation of the site can be cost-effective with the mineral value offsetting excavation costs.

Where accessible caves or man-made caverns are likely to be unstable, walls of brick or concrete can be used to support the roof (Waltham et al. 2005). Piling through cavities can

Voids, Table 2 Investigation techniques for voids

Method	Problems/benefits	Uses
Topographical and field survey	Shows void pattern if partly collapsed	Site characterization
LiDAR	Shows void pattern if partly collapsed	Site characterization
Boreholes	Drilled on a random pattern or on a grid, there is great scope to miss the target	Mine workings, caves, man-made voids
Boreholes and void sonar scanning	Good range underground if cavity is penetrated	Mine workings, brine cavities, etc.
Probing	Inexpensive compared with boreholes, limited depth of penetration	Collapsed ground, cavity fill, and voids
Boreholes and borehole camera	Short range dependent on clarity of water/air in cavity	Mine workings, caves, fissures, faults
Resistivity tomography	Good on greenfield sites, poor on previously developed sites	Voids, breccia pipes, and rockhead
Microgravity	Good on greenfield or previously developed sites	Large voids and breccia pipes
Ground-probing radar (GPR)	Shallow depth of penetration attenuated by clay	Near-surface voids and fissures
Seismic refraction and/or reflection and multichannel analysis of surface waves	Suitable for mine workings; confused by irregular karstic features and steep dips	Mine workings and salt dissolution voids
Cross-hole seismic	Requires an array of boreholes	Voids, shafts, caves
Passive seismic tomography (PST) (MASW)	Good for large cavities at depth	Voids, karst, mine workings, man-made cavities
Electromagnetic (EM) conductivity	Images of near-surface voids	Caves, shafts
Natural potential (NP) profiling	A complementary technique	Caves, mine workings, man-made cavities
Self-potential tomography (SPT)	Depth penetration to about 20 m	Shallow air or water filled caves; shafts
Internal surveying/LiDAR scanning	Requires access to void	Caves, any dry accessible and safe cavity

support overlying structures, but care is needed to prevent obstruction of the natural water flow (Waltham et al. 2005). Small voids can be mitigated with appropriate foundations to span any likely failure.

Grouting is the common method of dealing with voids (Warner 2004). Depending on their size, strong cement grouts may be utilized, but for large mine workings, low cement grouts with a filler of pulverized fuel ash (PFA) or sieved colliery spoil have been successful. For abandoned salt mines, cement, PFA, and brine mixtures have been used to prevent further dissolution. For voids in hydraulic structures, grouting can be difficult, and where highly soluble rocks such as gypsum and anhydrite are present it may be impossible, though some chemical grouts have been used with some success. Grouting voids in cave systems may be problematic and can cause ethical issues for cave conservation. In some cases, the voids and related caves may be too big to grout, or grout may be lost. Furthermore, grouting may affect the natural groundwater flow and induce dissolution in adjacent areas; this may be highly problematic in the more soluble rocks such as gypsum, anhydrite, and evaporites. Foam grouts can also be utilized to fill moderately large spaces where the roof is stable (Waltham et al. 2005).

Ingress of water into the ground can trigger cavities to develop into sinkholes. Sustainable drainage systems (SuDS) using infiltration, unlined drainage gulleys, and French drains should be avoided where voids are suspected. Reinforced and flexible services should be installed, preferably in lined trenches so that if a failure occurs, the water is directed away from sensitive structures to places of safe drainage. Fluctuations of groundwater levels within voided ground can also trigger collapse; abstraction from boreholes and irrigation can not only lower the local groundwater level but also add considerably to the amount of water infiltration triggering collapse. Similarly, open-loop ground source heat pump systems both abstract and return water to the ground with the likelihood of affecting voids and causing their collapse.

With all these methods, it must be appreciated that, in some places, the natural or man-made voids may be so severe that they are impractical to mitigate in which case complete avoidance of that ground is the only option.

Summary and Conclusions

Voids are commonly present in the near surface where they commonly encountered as unforeseen ground conditions and hazardous ground during civil engineering. Voids may be of natural occurrence, commonly associated with soluble (karstic) rocks including limestone, dolomite, gypsum, anhydrite, and salt. They also occur naturally associated with some volcanic rocks (breccias and lavas) or associated with landslides, cambering, soil piping, and some tectonic structures including faults. Man-made voids are particularly common in the near subsurface, and here they range from ancient to modern excavations for habitation, religious use, military use, transport, or water supply and storage. In addition to these and present at depths from the near surface to the deep subsurface, voids associated with mining are particularly prevalent representing the extraction of coal, iron ore, salt, and a long list of industrial, precious, and metalliferous minerals. Each commodity has its own preferred method of extraction, stratiform deposits being largely worked by room and pillar or longwall working; steeply dipping deposits may have been worked by digging levels and stopeing. Some soluble rocks, such as salt and evaporites, are commonly worked by solution mining. The wide range of natural and man-made voids that could be present need to be considered before any engineering is undertaken. By understanding the engineering properties and geotechnical behavior of the host rocks, local history, and form of any mineral deposits, ground can be characterized and then investigated using techniques including geophysics and boreholes. In most situations, voids can be mitigated by grouting or filling, but in some circumstances, avoidance is the best course of action.

Cross-References

▶ Boreholes
▶ Borehole Investigations
▶ Cambering
▶ Designing Site Investigations
▶ Dewatering
▶ Drilling Hazards
▶ Engineering Geological Maps
▶ Engineering Geomorphological Mapping
▶ Evaporites
▶ Failure Criteria
▶ Faults
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▶ Sabkha
▶ Sinkholes
▶ Site Investigation
▶ Subsidence
▶ Tension Cracks
▶ Tunnels
▶ Vibrations
▶ Volcanic Environments

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Volcanic Environments

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Definition

Environments that are affected either negatively or positively by volcanic processes.

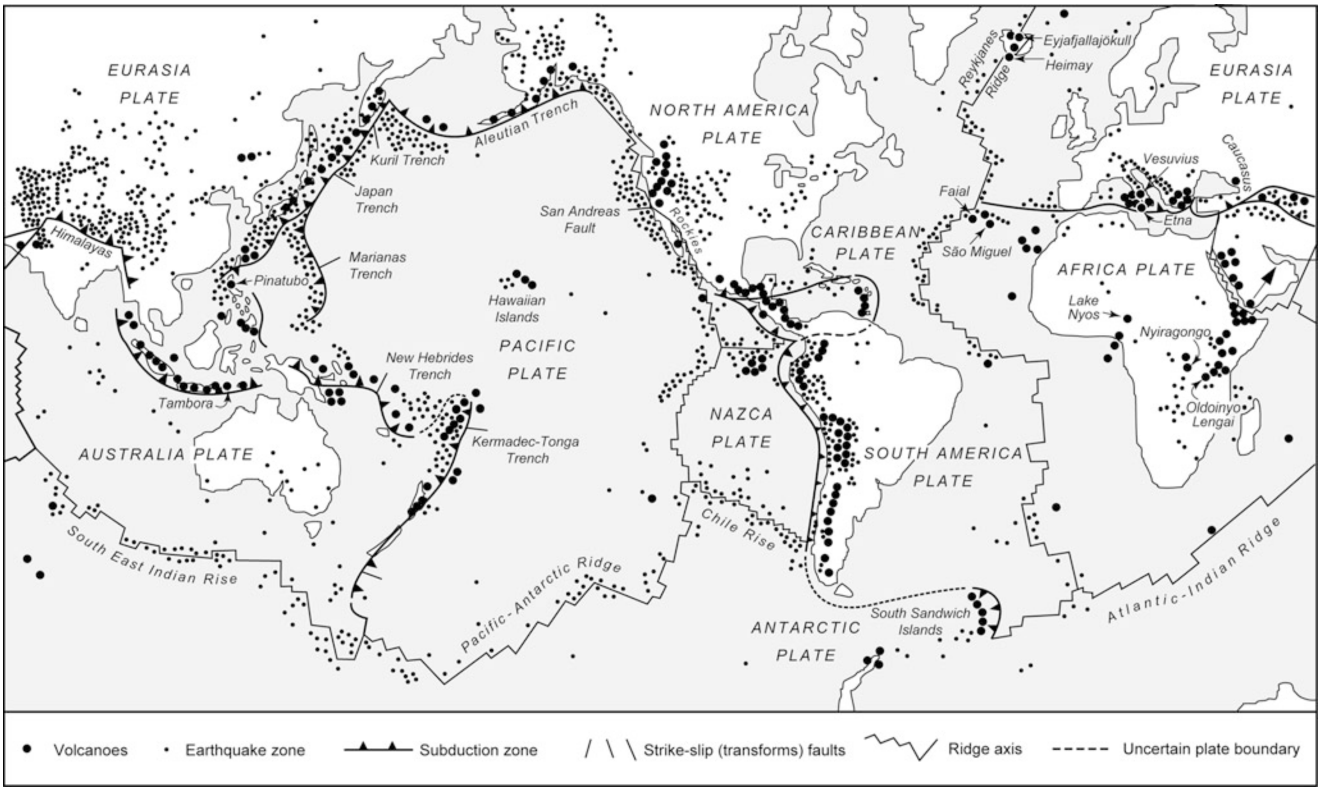
Introduction

During the Holocene (i.e., last 10,000 years), some 500 subaerial volcanoes have been active yet they occur in narrow bands that comprise less than 1% of the Earth's surface, being associated with divergent and convergent plate boundaries and intraplate "hot spots" such the Hawaiian islands (Tilling 2005). The higher the population density and the greater the level of economic development, the more severe will losses be in the event of an eruption. The interface between people and volcanoes also gives rise to resources of minerals, geothermal power, and potentially fertile soils, and it is these points of negative and positive interaction between volcanic processes and human activity that define areas in which there is a valuable role for the engineering geologist.

The Generation and Ascent of Magma

Volcanoes occur where magma is erupted at the surface of a planet. Magma is molten rock material and on Earth is almost exclusively a silicate melt with dissolved gas and typically some crystal content. There are rare examples, however, of carbonate and sulfur magmas. In terms of composition, erupted magmas can be broadly classified on the basis of increasing silica content ranging from *mafic* (basalts), through *intermediate* (andesites and trachytes) to *silicic* (rhyolites). The distribution of volcanoes on the surface of the Earth is related to global tectonics and in particular volcanism is associated with plate tectonic boundaries (Fig. 1, see Sigurdsson et al. 2015, chapter 3). *Divergent plate boundaries*, active ocean ridges, are associated with mafic volcanism erupting submarine basalts. *Convergent plate boundaries*, where oceanic lithospheric plate material is subducted into the mantle, occur as two main types: (1) island

V

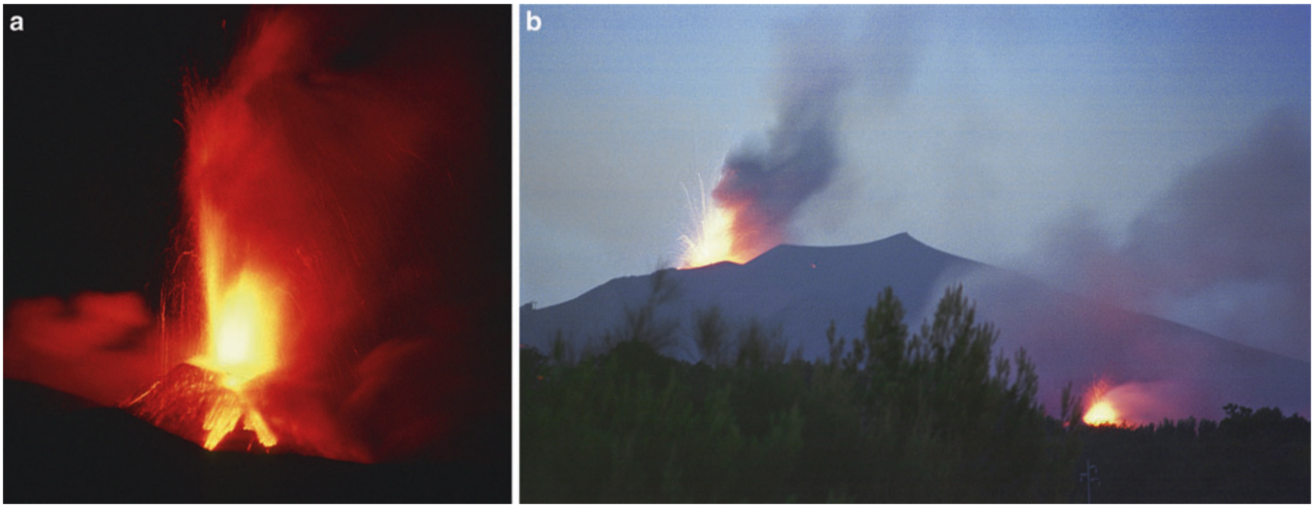


Volcanic Environments, Fig. 1 Volcano distribution and plate tectonics. Note that volcano locations are indicative and where clusters of volcanoes occur not all volcanoes are shown (Based on numerous sources)

arcs such as the South Sandwich Islands where oceanic plate is subducted beneath oceanic plate and characterized by mafic and intermediate magmas (basalts, andesites) and (2) active continental margins such as much of the western coast of South America where the Nazca oceanic plate is subducted beneath continental plate and this is characterized by intermediate to silicic volcanism (andesites and rhyolites). Volcanism also occurs in association with fracturing (rifting) of continental lithosphere with mafic (basalts), alkali-rich intermediate volcanism (trachytes), and occasional carbonate (carbonatite) volcanism. Oldoinyo Lengai in Tanzania in the East African Rift valley is the only example in recent historic times of a volcano erupting carbonatite magmas. *Within-plate* environments can be also associated with volcanism. The Hawaiian islands are basaltic volcanoes in the middle of the Pacific plate formed by generation of magmas from a hot spot which is thought to be caused by a mantle plume. A plume is a hypothesized upwelling of very hot rock within the mantle and is invoked to explain the occurrence of hotspot volcanoes like those of Hawaii and the flood basalts of regions such as Siberia and the Deccan.

Magmas that erupt at the Earth's surface typically derive from melts that originated in the Upper Mantle. Partial melting of mantle peridotite may be initiated through decompression, for instance, at the head of a rising mantle plume or upwelling of upper mantle at an active ocean ridge, or by reducing solidus temperature through influx of H_2O, typically H_2O from dehydration of subducted oceanic crust rising into the overlying mantle wedge (Sigurdsson et al. 2015, chapters 1 and 3). Melt separates from the source region as a primary magma and rises because it is less dense than the surrounding mantle. During ascent magma can stall, for example through encountering a change in density in the surrounding country rock, leading to zones of magma storage. Such magma reservoirs provide sites for fractional crystallization, processes generating secondary magmas with more evolved composition with increasing Si, Na, and K and decreasing Mg, Fe, and Ca (Sigurdssson et al. 2015, chapter 4). Magma systems in the crust feeding active volcanoes are investigated using geophysical techniques such as interferometric synthetic aperture radar (InSAR) and active and passive seismic techniques. It is probable that underneath many volcanoes small reservoirs of magma develop above much thicker columns of crystal-melt mush (Cashman and Sparks 2013; Sigurdsson et al. 2015, chapter 8).

At active ocean ridges where the crust is thin (<10 km), the magma is transported through dykes and there is only modest opportunity for differentiation of the mafic primary magma to occur prior to eruption. At the other extreme at active continental margins, the accumulation of mafic magmas at the base



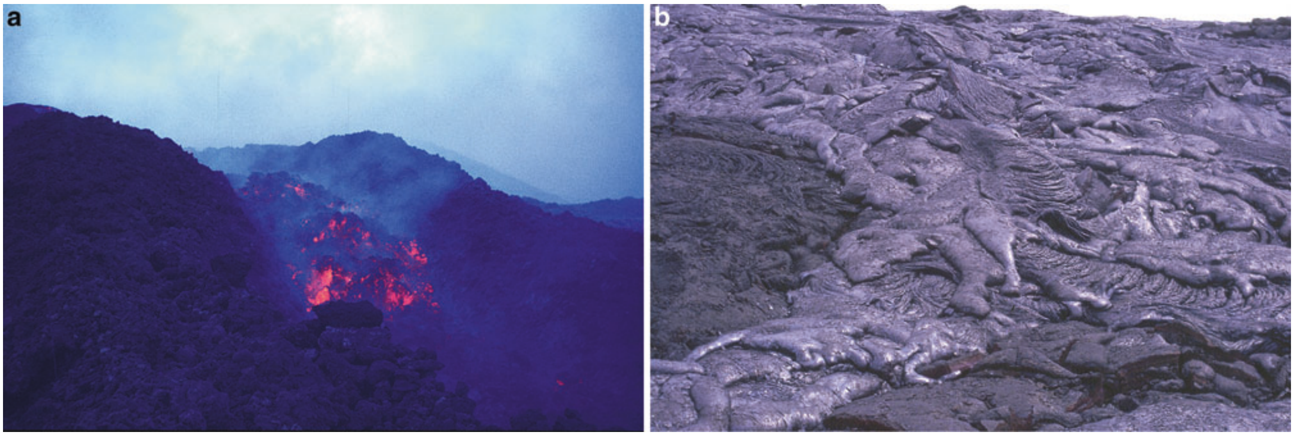
Volcanic Environments, Fig. 2 (**a**) Hawaiian-type lava fountain Etna 2000 (Photograph John Guest and Angus Duncan). (**b**) Strombolian activity Etna 2001 (Photograph Angus Duncan)

of the crust provides heat and the thickness of the continental crust (>40 km) provides space, for extensive differentiation of magma to take place by fractional crystallization, assimilation of crustal rocks, partial melting of crustal rocks, and mixing of magma generating large volumes of andesitic and rhyolitic magma (Sigurdsson et al. 2015, chapter 3).

The Eruption of Volcanic Products

The eruption of magma from a volcano generates three types of product: first, eruption of magma (largely degassed) as *lava* flows on the surface; second *pyroclastics* (tephra), fragmental material formed by explosive release and discharge of gas; and third, the *gas* itself. The nature and distribution of the products produced by a volcano will depend on the style and volume of its eruptions. Mafic magmas tend to erupt effusively, and lava is the main product with gas being readily released from the low viscosity melt and, when sustained as a continuous discharge, can either generate high lava fountains (Hawaiian style Fig. 2a) or – when not so vigorous – frequent small explosions (Strombolian style Fig. 2b). Rheology, the study of the way non-rigid material deforms when subjected to applied stress, is important in understanding the flow of lava. Newtonian fluids will flow when shear stress is applied whereas Bingham fluids will flow only when a critical value of stress is applied, that is, the fluid has a yield strength which must be exceeded before flow will occur. At near liquidus temperatures, basaltic magmas behave as Newtonian flows but, as cooling and crystallization proceed, the flow develops yield strength and most lava flows are emplaced as Bingham fluids.

There are two main types of basaltic lava flow, *aa* and *pahoehoe* (the terms derive from Polynesian words used in Hawaii to describe different types of lava morphology). Active *aa* lava flows typically comprise a surface crust of jumbled clinker spinose fragments rafted on the mobile interior of fluid lava. As the flow progresses the central portion often drains down slope creating an open channel, bound by static rubble levees, that feeds the flow front (Fig. 3a). By way of contrast, *pahoehoe* lava flows have smooth, billowy surfaces, ropy textures, and extruded lobes, with lava being transported to the flow front by tubes developed within the flow (Fig. 3b). Inflation is an important process in the development of large sheets of *pahoehoe* lava. The transformation from *pahoehoe* flow conditions to *aa* requires the crust of the lava flow to be disrupted and this can occur through increased rate of shear when the lava flow encounters a steeper slope. Lava flows tend to slow down rapidly as they move away from the vent and usually advance slowly so people have plenty of opportunity to escape. There are rare examples of fast moving flows such as the eruption of Nyiragongo in 1977, when lava travelled around 6 km in 20 min overrunning a village and killing 70 people (Sigurdsson et al. 2015, chapter 55). Lava flows cause long-lasting damage to agricultural land and infrastructure (e.g., Vesuvius 1944, Etna 1669 and 1928, Heimay 1973). When basalt magma comes into contact with water at low pressure (either at the surface or at shallow depth), this can cause vigorous generation of steam and local explosive activity: hydrovolcanism. In 1957 there was a basaltic eruption just offshore from the Island of Faial (Azores), which built a subaerial tuff cone that eventually became joined to the island by an isthmus of tephra. The eruption lasted just over a year and the fine ash caused by the explosive water-magma interaction (phreatomagmatic activity) had a devastating impact on agriculture and subsequently many people emigrated to the USA (Coutinho et al. 2010).



Volcanic Environments, Fig. 3 (**a**) Active aa flow Etna 1983 (Photograph Angus Duncan). (**b**) Pahoehoe lava Kilauea, Hawaii (Photograph Angus Duncan)

Fine ash generated by volcanic eruptions when ingested into jet engines can cause major damage. The April–May 2010 phreatomagmatic eruption of Eyjafjallajökull volcano, Iceland, generated a low plume of ash at an altitude of ca. 6100–9100 m that resulted in a major disruption of Western European airspace. Explosive eruptions generated by gas-rich intermediate/silicic magmas present the greatest threat to human activity. The hazard in large measure relates to the scale of the eruption, and this is commonly represented by the *Volcanic Explosivity Index (VEI)*, which is based on eruption volume and height of the eruption column (Sigurdsson et al. 2015, chapter 13).

In the last 150 years, there have been a number of explosive eruptions of sub-Plinian/Plinian character with VEI (4–6), one of the largest of which was Pinatubo (VEI 6) in the Philippines (1991). These eruptions last from several hours to a few days and generate eruption columns that can rise to altitudes in excess of 35 km reaching a point of neutral buoyancy and drift downwind with pyroclastic material falling from the plume mantling the ground topography, tephra fall deposits becoming progressively finer grained and thinner away from the volcano and in the travel direction of the stratospheric wind. The ash in the plume posed a hazard to aviation in the region. It is entrainment of air which is heated by the hot gas/pyroclasts mixture of the column that allows the plume to become buoyant. As discharge from the vent wanes, then the process of entrainment and heating of air becomes less effective such that the column is no longer buoyant and collapses under gravity. Collapse of the column from several thousand meters forms flows of hot pyroclastic material and gas that sweep down the flanks of the volcano as pyroclastic density currents (PDCs). These can cause devastation and loss of life, as occurred during the first night and early morning of the AD79 eruption of Vesuvius (Italy). Even when buoyant, eruptive columns can show instability with PDCs being generated from buoyant plumes (Cashman and Sparks 2013). Pinatubo was monitored and closely documented, and details of its precursory activity, eruption dynamics, and immediate/long-term impact and responses to the event are summarized in Table 1. Only one eruption of VEI 7 has occurred in the last 300 years, Tambora (Indonesia) in 1815 and this was large enough for substantial volumes of SO_2 to be injected into the stratosphere to form aerosols of sulfuric acid which had a residence time of 1–3 years before settling. These stratospheric aerosols formed a screen reflecting sunlight and causing global cooling. The poor summer weather in 1816 led to failed harvests and famines in Europe and North America. Although located in the Southern Hemisphere, this eruption was close to the equator and the stratospheric circulation ensured a global impact (Sigurdsson et al. 2015, chapter 53).

Magmatic gas can present a hazard from volcanoes even when they are not erupting, the volcanic system can remain a conduit for gases such as CO_2 even during periods of quiescence. Lake Nyos is a volcanic crater lake in Cameroon that has not erupted for hundreds of years. CO_2 seeps into the lake floor being dissolved in the bottom waters. On 21 August 1986 the water in the lake overturned, possibly caused by a landslide, and a cloud of CO_2 gas escaped from the lake and asphyxiated around 1700 people (Scarth 1999). Even when volcanoes are quiescent seepage of CO_2, a dense gas that collects in depressions and enclosed spaces, poses a serious hazard. In Graciosa in the Azores archipelago in 1992, two tourists died in a lava tube in the caldera from CO_2 asphyxiation: the volcano has not been active for more than 500 years.

Many volcanoes are unstable landforms and may be dangerous places when not in eruption (e.g., landslides, lateral collapse, floods from crater lakes and heavy rainfall on heavily dissected terrain) (see Table 1).

Volcanic Environments, Table 1 Summary of the eruption of Pinatubo (Luzon Island, Philippines). For further details see Scarth (1999)

The 1991 eruption of Pinatubo was one of the best examples since 1900 of the impact of a large gas-rich silicic explosive eruption on a surrounding population. Before the eruption, Pinatubo was not recognized by the authorities as a volcano likely to erupt and was not closely monitored.
Precursory activity
2 April 1991: Small steam explosions occurred along a fissure near to the summit. Fine ash was erupted and a smell of sulfur was reported by local authorities. The *Philippine Institute of Volcanology & Seismology* (PHIVOLCS) investigated, installed seismographs, and recorded 200 small earthquakes which constituted a seismic swarm.
5 April 1991: PHIVOLCS identified a 10 km radius danger zone around the summit of the volcano. Five hundred farmers were evacuated into refugee camps.
23 May 1991: PHIVOLCS, with support from US Geological Survey, completed a preliminary Hazard Assessment Map based on analysis of deposits from previous (more than 500 years old) explosive eruptions. This map identified areas likely to be affected by ash fall, pyroclastic flows, and mudflows (lahars).
7 May–1 June 1991: A total of 1800 earthquakes were recorded with a 2–6 km depth range reflecting the ascent of magma. Increased discharge of sulfur dioxide on 1st June indicated fresh magma at a high level in the volcanic system.
3 June 1991: More earthquakes (seismic activity), small explosions, and the summit area began to bulge.
7 June 1991: A total of 1500 earthquakes were recorded and an explosion of steam and cold ash rose to a height of 8 km above the summit. The radius of the danger zone was extended to 20 km. In the evening, magma reached the surface forming a small lava dome.
The eruption
9 June 1991: Magmatic explosions begin, sending pyroclastic flows down valleys over distances of up to 4 km. Farmers were evacuated from within a 10 km radius danger zone. Around 25,000 people were housed in refugee camps. Some people refused to leave their farms and many were subsequently killed.
10 June 1991: US Air Force evacuated Clark Air Base, located some 25 km from the summit.
12 June 1991: Increase in explosive activity occurred, with the eruption column reaching a height of 20 km and pyroclastic flows devastated some of the evacuated villages. The evacuation zone was increased to a 30 km radius.
15 June 1991: The eruption reached a climax. Ninety percent of the 10 km^3 of material erupted by Pinatubo during the eruption was discharged. The eruption column rose to 35 km. Pyroclastic flows traveled for distances of up to 16 km from the summit and impacted an area of 400 km^2. Thick, wet ashfall (a tropical storm occurred at the same time as the climax) was deposited around the volcano and accumulation of this heavy wet ash resulted in roof collapses: the main cause of death in this eruption. Following the discharge of the large volume of magma during the climax of the eruption, collapse of the summit formed a 2.5 km diameter caldera.
After the climax, activity subsided and the eruption finally ended on 4th September.
Outcomes
Up to 800 fatalities. During the eruption, most deaths were caused by roof collapse as a result of ash accumulation, and fatalities also occurred among those who refused to evacuate. Acute respiratory problems and other illnesses in refugee camps led to more deaths. A large number of people continued to be killed after the eruption as a result of mudflows (i.e., lahars).
The high-altitude plume of ash from the explosive phase of the eruption disrupted commercial aviation, causing engine failure. Fortunately, there were no crashes or fatalities.
The eruption injected a large volume of sulfur dioxide (SO_2) into the stratosphere and this reacted with atmospheric gases to form sulfuric acid aerosols, which had a 1–2 year residence time leading to a small (<1 °C) global cooling in 1992/1993.
After the eruption, there were thick deposits of ash on the flanks, in particular in the valleys, of the volcano and these were prone to being reworked during periods of heavy rainfall. In the typhoon season, heavy rainfall generated lahars that swept down valleys and engulfed villages. A number of measures were implemented to address this problem. A system of "watchmen," located upstream of the villages, was put in place to give advanced warning to the inhabitants. This was effective and saved hundreds of lives. Dams and *levées* were also built to protect villages but were not as effective. In 1995 (4 years after the eruption), the town of Bacolor was struck by lahars and over 100 people lost their lives (Sigurdsson et al. 2015, chapter 56).

Engineering Geology of Volcanic Environments

Positive Benefits Although volcanoes are popularly perceived as agents of destruction (see below), humankind derives considerable benefits from magmatic processes, many of which are only realized through the application of engineering geology. Lava and pyroclastic deposits have been utilized for building since antiquity, and today, volcanic products are not only used directly as freestones but also as constituents in concrete/cinder blocks and as construction aggregates. In the case of the latter, specifications are frequently demanding with laboratory testing being required in order to determine whether particular rocks are suitable. More specialized uses include: highly water absorbent bentonite, a clay formed from altered volcanic ash, employed in foundries and as a "drilling mud"; pumice, low-density bubble-rich volcanic glass, used as an abrasive and in soap manufacture; perlite, hydrated volcanic glass, utilized as an aggregate, as a substitute for sand in some specialized plasters/fillers and, because of its high surface area to volume ratio, it may be adopted as an air and water retentive medium in agriculture. Refractories are materials that are not deformed or damaged by high temperatures and have a variety of uses that include insulation, firebricks, and furnace linings. Pumice, bentonite, welded tuffs (i.e., ignimbrites) are either used directly, or as important ingredients of formulated refractories (Sigurdsson et al. 2015, chapter 74).

As noted by Robert and Barbara Decker (1981 p. 168) "volcanoes are nature's forges and stills where elements of the Earth, both rare and common are moved and sorted," and the elements associated with magmatic processes both direct and indirect include: fluorine, sulfur, zinc, copper, lead, arsenic, tin, molybdenum, uranium, tungsten, silver, mercury, and gold. Magma chambers beneath volcanoes supply both heat and fluids. As silicic magma collects and cools it forms a reservoir within the crust. Minerals then crystallize leaving a residual water-rich melt which generates high temperature hydrothermal fluids (the water being of magmatic origin) enriched in incompatible elements. These hydrothermal fluids then migrate into the surrounding country rock causing sulphide mineralization. Some minerals precipitate over a wide range of pressures and temperatures, whereas others (e.g., gold and silver) have a very narrow range and this leads to ore zonation. An important type of the mineralization associated with volcanoes is porphyry (e.g., copper, molybdenum, and gold) deposits, which can form 2–4 km below calc-alkaline volcanoes in subduction zone environments. Such mineralization provides an important source of these metals in areas such as the Andes and Indonesia (White and Herrington in Sigurdsson et al. 2000, pp. 897–912).

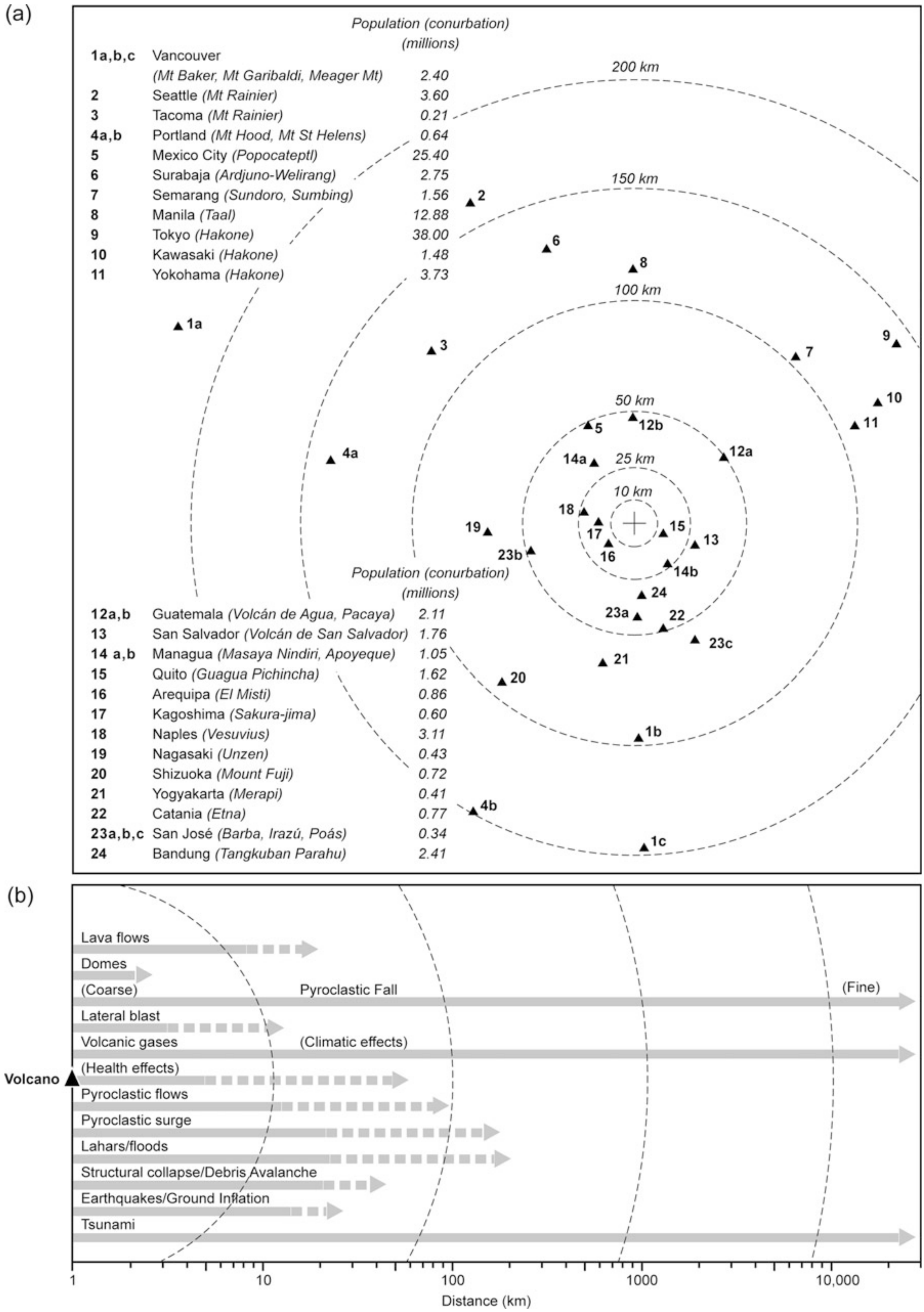
Hydrothermal fluids can also form from meteoric (nonjuvenile) waters in the crust. Submarine hydrothermal systems occur at active ocean ridges where ingress of sea water into the oceanic crust leads to hydrothermal convective systems driven by heat from active basaltic magma reservoirs within the basic igneous country rock. These hydrothermal fluids react with the igneous rocks leaching out metal cations such as Fe and Ca. Where these hydrothermal fluids vent at the sea floor they form minerals including sulfides (Butterfield in Sigurdsson et al. 2000, pp. 857–875). Hydrothermal systems in oceanic crust associated with volcanic arcs and back arc basins have higher concentrations of SO_2 and CO_2, and metals such as K, Mn and Ca which provides the potential for significant mineralization to occur below the sea floor (Sigurdsson et al. 2015, chapter 14). For further information on the origin and emplacement of, and prospecting for, magmatically-derived ore deposits see the review by Gibson (2005).

It is a common assertion that volcanic soils are quite fertile and have supported many agricultural civilizations, but this is only partially true. What are usually considered volcanic soils are *Andisols* (i.e., *Andosols*) developed in tephra, one of the 12 orders of the Soil Taxonomy, but weathering of lava over longer time periods can produce soils across a range of soil orders, not all of which are notable for their fertility. Andisols are generally uncompacted, water retentive, and are able to develop a high organic content, but require careful management. Not only are they easily eroded especially on the steep slopes of a volcano, frequently requiring terracing and other soil conservation measures, but andisols are also notoriously phosphorous deficient. Allophane minerals are an important constituent and possess a high phosphorus fixing capacity with deficiency often limiting crop growth, especially if previous overexploitation has caused depletion of soil organic matter and increased acidification. Traditionally phosphorus (plus potassium and nitrogen) have been obtained from manure, but increasingly it is derived from phosphate-rich rocks many of which also contain fluorine. With long-term use this may cause problems of groundwater contamination and pose a risk to grazing animals. For further details see Sigurdsson et al. (2015, chapter 72; Fig. 4).

The amount of energy stored in the Earth in the form of heat is of the order 10^{31}J, but this is only exploitable where geological conditions favor the development of hydrothermal systems, volcanic and tectonic active areas being the most important. The geothermal energy used in power stations depends on several geological processes: concentration of heat in magma; its transport into the upper parts of the crust; plentiful water and permeable rocks. Heat sources are magma chambers in the upper crust, and the larger, younger and closer a chamber is to the surface then the higher is the geothermal potential. There are many plant designs (Duffield 2005) and examples where geothermal power has been developed include: California; Iceland, generating ca. 50% of the country's energy needs; Italy; São Miguel Island (Azores), accounting for ca. 42% of the island's requirements (Fig. 5); and Wairakei, New Zealand. From an engineering perspective, harnessing geothermal power is challenging. It involves not only identifying or inferring magma chambers by applying geological, geophysical, and petrological methods of investigation often through remote sensing, but also designing the plant, the "plumbing" linking station to reservoir with care being taken to avoid contaminating surrounding land and developing power distribution networks. Some of the engineering geology issues involved in geothermal power development are further discussed by Duffield (2005) and Sigurdsson et al. (2015, chapter 71).

Dealing with the negatives Volcanoes represent a risk to people living in their vicinities and, as Fig. 4a, b shows, not only are many large cities within range of erupted products, but the effects of certain phenomena such as fine ash, gases, and volcano-induced tsunamis may also be felt over thousands of kilometers, though most categories of threat are confined to within tens of kilometers or less of the eruption site.

There are several areas in which the expertise of the engineer and engineering geologist may be utilized. For instance, there are two approaches to prediction and both require technical expertise. *General Prediction* (i.e., hazard mapping or assessment) studies the past behavior of a given volcano in order to understand the frequency, magnitude, and styles of its eruptions, so as to delineate areas of differing risk. Traditionally output has been a hazard map, but increasingly data are



Volcanic Environments, Fig. 4 (**a**) A selection of the world's most vulnerable conurbations, plotted according to the relative distance and direction from the nearest volcano(oes) (Modified and updated from Munich 1984, Fig. 9). (**b**) The influence of distance on the impact of volcano and volcano-related phenomena (Modified and updated from McGuire 1998, Fig. 1b)

Volcanic Environments, Fig. 5 Ribeira Grande geothermal power station on São Miguel Island in the Azores. It is one of the two plants that exploit the resources of a 240°C liquid-dominated reservoir. Together the plants generate 186 GW h, representing 42% of the power production of the island and 22% of the Azores as a whole (Photograph courtesy of Professor José Pacheco)

being analyzed using computer-based *Geographical Information Systems (GIS)*, with information being compiled in layers relating to spatial patterns of varying categories of risk, topography, demography, settlement, evacuation routes, and other features of human vulnerability.

Specific prediction (i.e., monitoring or forecasting) seeks to estimate the time and type of eruption and is based on surveillance of a volcano by monitoring precursory changes in such phenomena as seismic activity, ground deformation, thermal characteristics, infrasound, and the geochemistry of gases and waters. Many of these procedures make use of detailed ground-based measurement and techniques of remote-sensing either from aircraft and/or satellite. In recent years, the use of satellite data from GPS measurements of distance combined with synthetic aperture radar interferometry (InSAR), has led to major improvements in monitoring the ground deformation of volcanoes (Sigurdsson et al. 2015, chapter 66). InSAR uses radar images from satellite scans of a volcano surface, and these are compared with images from previous passes by the satellite. Under optimum conditions, movements of less than a centimeter can be detected (Sigurdsson et al. 2015, chapter 66). Many satellites also collect Thermal InfraRed (TIR) data which are valuable in detecting thermal anomalies which may occur prior to an eruption. Modern satellite sensors can measure temperatures up to 2000 °C without the sensor being saturated, allowing the measurement of active lavas. Airborne light detection and ranging (LIDAR) is a powerful tool in monitoring active lava low fields. In 2006, LIDAR imagery was used to collect high resolution morphological data at time intervals of an active lava flow field. The capacity during an eruption to collect data on lava flow field dynamics, volume fluxes, and emplacement conditions has the potential to support hazard assessment by improving the ability to model and predict flow field growth (Favalli et al. 2010). Ozone monitoring satellites (e.g., TOMS) measuring UV and TIR can be used to measure SO_2 and ash content (Sigurdsson et al. 2015, chapter 66). SO_2 levels in the atmosphere are important in the monitoring of climate change and identifying ash in volcanic plumes is a vital consideration in aviation safety. The engineering geologist may also have an important role in working to mitigate the effects of eruptions once they have begun. There are examples of lava flows being diverted and cooled (e.g., Heimay 1973, Etna 1983 and 1992), small lahars being trapped by dams and advice being given to the aviation industry about the dangers of volcanic materials to jet engines and aircraft structures. For more information reference should be made to Sigurdsson et al. (2015, chapters 51, 52, 53, 54, 55, 56, 57, 58, 63, 64, 65 and 70) and Tilling (2005).

Summary

Engineering geologists have valuable contributions to make at the boundary between volcanism and human activity. More particularly their expertise may be particularly worthwhile in areas which include: assessing the commercial potential of volcanic products; harnessing geothermal power; *general* and *specific* studies of prediction, especially where these involve techniques that include remote sensing and the use of GIS; working to mitigate the effects of an eruption after it has begun, by advising on techniques such as diversion of lava and the trapping of pyroclastic materials in temporary storage sites; furnishing information to the aviation industry about the dangers of ash to aircraft structures and engines and making civil authorities aware that volcanoes are potentially dangerous even when they are not in eruption (e.g., gas flux and the risks presented by gravity-fed mass-movement processes).

Cross-References

▶ Acidity (pH)
▶ Aerial Photography
▶ Aeromagnetic Survey
▶ Alkali-Silica Reaction
▶ Bedrock
▶ Building Stone
▶ Chemical Weathering
▶ Collapsible Soils
▶ Contamination
▶ Earthquake
▶ Earthquake Intensity
▶ Earthquake Magnitude
▶ Engineering Geological Maps
▶ Engineering Geology
▶ Engineering Geomorphological Mapping
▶ Geohazards
▶ Geology
▶ Geothermal Energy
▶ GIS
▶ Hazard
▶ Hazard Mapping
▶ Hydrothermal Alteration
▶ Igneous Rocks
▶ InSAR
▶ Instrumentation
▶ Land Use
▶ Lidar
▶ Mountain Environments
▶ Risk Assessment
▶ Risk Mapping
▶ Shear Stress
▶ Tsunamis

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W

Waste Management

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Definition

Managing wastes resulting from human activities in the most sustainable and least environmentally damaging ways using suitable wastes, as far as practicable, as resources for productive purposes.

All human activities, industrial, agricultural, domestic, or others inevitably produce waste. Some waste is harmful to health. Others, during decomposition, cause nuisance from odor, attract vermin, or cause contamination of soils and pollution of water. This chapter focuses on solid and liquid waste (but not waste water or radioactive waste). Disposal of waste by tipping has been used for thousands of years (early sites are now archaeologically valuable). But landfill has caused increasing problems as large populations in growing urban areas have produced ever greater volumes of waste. Large, numerous landfills divert land from other productive uses and can also cause short and long term hazards.

This has led to an approach to management of waste known as the waste hierarchy. This aims to minimize or make the best uses of the components of waste with the aim of transforming these, as far as possible, from problems to resources (Cheng and Pires 2015). The hierarchy is:

- Prevention – not creating waste in the first place
- Minimization – reducing waste, for example by reducing packaging materials
- Re-use – for instance by retaining or using furniture or equipment
- Recycling
- Energy recovery
- Disposal (through landfill or incineration without recovery) – least desirable

A successful waste strategy requires co-operative forward planning and investment by government, regulators, and industry (Cheng and Pires 2015.). It also requires producers, handlers, and depositors of waste to deliver according to the hierarchy including investment in specialized sorting and recycling plants to provide high-quality feedstock. If potentially useful waste becomes mixed then the default option is usually landfill (Fig. 1). The engineering geologist has a limited role in prevention, minimization, and reuse but makes important contributions to other aspects.

Recycling

Types of waste that can be recycled include:

- Glass – crushed to produce sand-grade material for use as aggregate or, with the addition of some silica sand, can be used to make new glass products
- Metals – refined and recast as new products
- Construction and demolition waste – crushed and graded for use as aggregate or construction landscaping and fill (Figs. 2 and 3)
- Soils – either recovered for use as soil or construction landscaping and fill
- Paper and cardboard – for manufacture of new paper and cardboard
- Fabrics and textiles – for new fabrics or use as fillers in, for example, paper
- Plastics – some of which can be used in new plastic products (Rogoff 2013).

A key factor in achieving high rates of recycling is a market demand for recyclable materials leading to a price that makes the investment in processing profitable. Price

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depends on quality of the feedstock therefore it is essential that different types of materials for recycling are well segregated (Fig. 3). This can be achieved by separation by the home occupier at source or at a municipal facility. In practice, this may be difficult because individuals are variably conscientious at this task. An alternative is collecting household waste with separation only of biodegradable products from other, recyclable materials which can then be mechanically and manually separated at Materials Recycling Facilities (Fig. 4; CallRecovery and PEER Consultants 1993).

Contamination of potentially recyclable materials can make it uneconomic to separate contaminated materials that must then be landfilled rather than used; for instance, the contamination of demolition waste by asbestos if it is not carefully removed before demolition. It is, also, important for some types of waste to recover particularly valuable materials; for instance, end-of-life mobile phones contain elements and compounds that are scarce in nature in viably economic concentrations and are difficult and expensive to secure in the international market. More generally, recycling of construction and demolition waste, glass, and metals also reduce the demand for new mining and quarrying and therefore contribute to environmental protection (Townsend 2001).

Waste Management, Fig. 1 Unsorted mixed waste, London, UK, which cannot be recycled and will go to landfill thus wasting some potential resources (Photograph by the author)

Another type of recycling is composting of garden and food waste to produce dressings for agriculture, horticulture, gardens, and allotments to partly replace the use of natural materials such as peat with consequent environmental benefits (Fig. 5; Epstein 2011).

Energy Recovery

The principal methods of energy recovery are:

- Incineration of combustible materials – organic waste, paper, and many plastics can be burnt at high temperature to generate energy (and some waste, such as clinical items, must be incinerated) but careful monitoring and regulation is required to prevent harmful levels of atmospheric pollution and most incineration residues must be land-filled (Cheremisinoff 2013).
- Anaerobic digestion of organic materials (Mata-Alvarez et al. 2000).
- Landfill gas – methane is generated by landfills containing decomposing waste. This can be burnt by “flaring it off” but, more productively, can be used in generators to provide local power or heating (Fig. 6; Environment Agency 2004).

Disposal

The aim of waste management is to minimize disposal to landfills. Landfills can cause environmental problems during operation (e.g., odors and vermin) or subsequently (e.g., adverse effects of leachates on ground and surface waters

Waste Management, Fig. 2 Construction and demolition waste stacked awaiting crushing, London, UK (Photograph by the author)

Waste Management, Fig. 3 Mobile crusher crushing construction and demolition waste for use as construction aggregate, Yorkshire, UK (Photograph by the author)

Waste Management, Fig. 4 A Material Recycling Facility for separation of clean municipal and commercial wastes for recycling (Photograph by the author)

Waste Management, Fig. 6 Pipework for recovery of methane for energy production from a restored landfill site, Yorkshire, UK (Photograph by the author)

Waste Management, Fig. 5 Composting of organic agricultural, garden and food wastes to produce materials for improvement of soils, Suffolk, UK (Photograph by the author)

and evolution of gases). Also landfills prevent or inhibit other uses of land on the selected sites (Fig. 7). However, there is often no other option for mixed waste and controlled (i.e., potentially harmful) products can only landfilled in specialized facilities (McBean et al. 1995).

Role of the Engineering Geologist

The engineering geologist has an important or essential role in:

- Identification of suitable sites and design and monitoring of landfill sites
- Assessment of suitability of crushed and graded construction and demolition waste for use as aggregates

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Waste Management, Fig. 7 Former quarry lined after installation of a basal drainage system ready for landfilling, Yorkshire, UK. For more information see the entry on landfill (Photograph by the author)

- Ground and site investigations for new buildings for waste management
- Assessment of natural hazards that might adversely affect waste management sites or environmental hazards that might be caused by these

Summary

Waste management is a strategic process to make the best uses of materials that would otherwise be disposed of in landfills with adverse environmental and land-use consequences. It aims to treat most waste as resources rather than problems. Strategies reflect the waste hierarchy. These require collaboration between government, regulators, industry, and commerce. But these also depend on the co-operation of the general public and commerce in directing waste into the correct "waste streams."

Cross-References

► Aggregate Tests
► Landfill

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Water

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Synonyms

Hydro

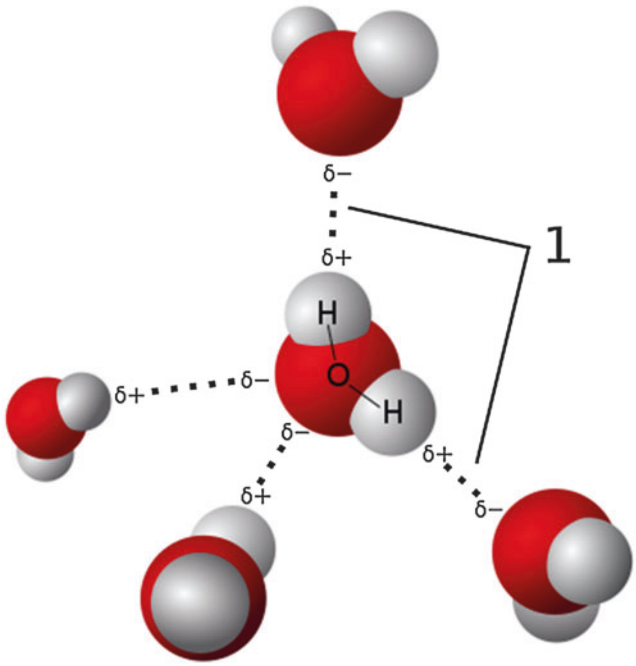
Definition

Water (H_2O) each molecule consists of one oxygen atom and two hydrogen atoms, connected by covalent bonds, forming a polar inorganic compound that is, at room temperature, a tasteless and odorless liquid, nearly colorless with a hint of blue. It covers more than 70% of the Earth's surface. It is the only common substance to exist as a solid, liquid, and gas in normal terrestrial conditions (Reece 2013).

Introduction

The word "water" comes from Old English "wæter," from Proto-Germanic watar (source also of Old Saxon watar, Old Frisian wetir, Dutch water, Old High German wazzar, German Wasser, Old Norse vatn, Gothic wato "water"), from Proto-Indian European wod-or, suffixed form of root wed-... "water; wet." Water is essential for life. With two thirds of the Earth's surface covered by water and the human body consisting of 75% water, it is evidently clear that water is one of the prime elements responsible for life on Earth. However, contrary to the past, our recent developed technological society has become indifferent to this miracle of life. Our natural heritage (rivers, seas, and oceans) has been exploited, mistreated, and contaminated.

Water is a liquid at the temperatures and pressures that are most conducive to life. But in general, there are three states of



Water, Fig. 1 Model of hydrogen bonds (1) between molecules of water (Reference: http://en.wikipedia.org)

water in nature: solid (ice), liquid, and gas (steam or water vapor). Since the water molecule is not linear and the oxygen atom has a higher electronegativity than hydrogen atoms, it is a polar molecule, with an electrical dipole moment: the oxygen atom carries a slight negative charge, whereas the hydrogen atoms are slightly positive (Fig. 1).

This simplest hydrogen chalcogenide is by far the most studied chemical compound and is described as the "universal solvent" for its ability to dissolve many substances (Greenwood and Earnshaw 1997). This allows it to be the "solvent of life" (Reece 2013).

Physical and Chemical Properties of Water

Water has specific physical and chemical characteristics. The important physical properties of water are cohesion (water is attracted to water) and adhesion (water is attracted to other substances). Water quality primarily depends on chemical characteristics and include color, dissolved oxygen, electrical conductivity, hardness, pH, saline water, suspended sediment, and turbidity (Swanson and Baldwin 1965).

Color: Water, even pure water, is in fact not colorless; but has a slight blue tint.

Dissolved oxygen: Another common measurement often taken is dissolved oxygen (DO), which is a measure of how much oxygen is dissolved in the water.

Electrical conductivity: Pure water is an excellent insulator and does not conduct electricity. Water that could be considered "pure" would be distilled water (water condensed from steam) and deionized water (used in laboratories), although even water of this purity can contain ions. Water ceases to be an excellent insulator once it starts dissolving substances. Salts, such as common table salt (NaCl), are ionic compounds composed of cations (positively charged ions) and anions (negatively charged ions). In solution, these ions essentially cancel each other out so that the solution is electrically neutral (without a net charge).

Hardness: The simple definition of water hardness is the amount of dissolved calcium and magnesium in the water. Hard water is high in dissolved minerals, both dominantly calcium and magnesium.

pH: pH is a measure of how acidic or basic water is. The range goes from 0 to 14, with 7 being neutral. pH of less than 7 indicates acidity, whereas a pH of greater than 7 indicates alkalinity.

Saline water: Water that is saline contains significant amounts (referred to as "concentrations") of dissolved salts, the most common is sodium chloride (NaCl).

Suspended sediment: Suspended sediment is generally transported within and at the same velocity as the surrounding fluid, in this case, water. Most sediment particles are not continuously suspended but are continuously settling through the surrounding fluid and may eventually return to the bed.

Turbidity: Turbidity is the measure of relative clarity of a liquid. It commonly serves as an indirect measure of suspended sediment.

Earth's Water

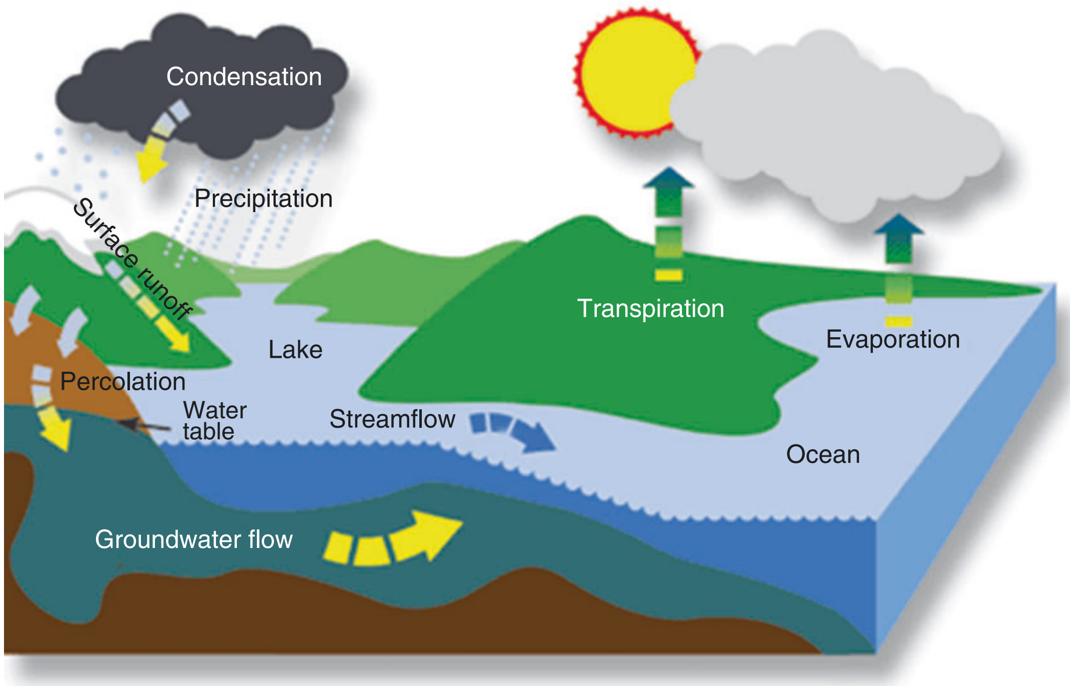
Earth's water is always in movement. The natural water cycle, also known as the hydrologic cycle, describes the continuous movement of water on, above, and below the surface of the Earth. Water changes state from solid, to liquid or gas, depending on variations in temperature and pressure both in the short term and over geological time.

Water Cycle

The hydrologic cycle in nature is a conceptual model that describes the storage and movement of water between the biosphere, atmosphere, lithosphere, and the hydrosphere (Fig. 2).

Water on this planet can be stored in any one of the following reservoirs: atmosphere, oceans, lakes, rivers, soils, glaciers, snow fields, and groundwater.

Water moves from one reservoir to another by way of processes such as condensation, precipitation, evaporation,



Water, Fig. 2 Schematic diagram of the hydrologic cycle (Reference: http://water.usgs.gov)

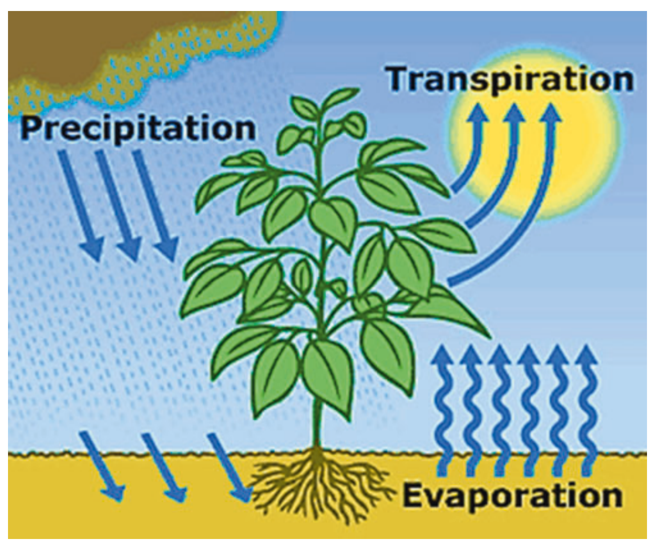
transpiration, evapotranspiration, deposition, infiltration, interception, runoff, sublimation, melting, and groundwater flow. Water is continually cycled between its various reservoirs.

Condensation is the process by which water vapor in the air is changed into liquid water by cooling and is the reverse process to evaporation. Condensation is crucial to the water cycle because it is responsible for the formation of clouds. These clouds may produce precipitation, which is the primary route for water to return from the atmosphere to the Earth's surface within the water cycle. Precipitation is the moisture that falls from the atmosphere as rain, snow, sleet, or hail.

Evaporation is the phase change of liquid water into a vapor. Evaporation is the primary pathway that water moves from the liquid state back into the water cycle as atmospheric water vapor.

Studies have shown that the oceans, seas, lakes, and rivers provide nearly 90% of the moisture in the atmosphere via evaporation, with the remaining 10% being contributed by plant transpiration. Transpiration is the process by which plants return moisture to the air. Evapotranspiration is the combined net effect of evaporation and transpiration in other words (Fig. 3).

The amount of water that plants transpire varies greatly geographically and over time. There are a number of factors that determine transpiration rates: temperature, relative humidity, wind and air movement, soil moisture availability, and type of vegetation.



Water, Fig. 3 Relationship between precipitation and evapotranspiration (Reference: http://water.usgs.gov)

Interception occurs when water reaches the surface in various forms of precipitation directly into bodies of water, by plants, or falls directly onto structures forms of precipitation directly into bodies of water. Precipitation that collects on the leaves or stems of plants is a form of interception. Precipitation on plants may evaporate back into the air or flow down the plant to the ground as stem flow.

When the precipitation rate exceeds the infiltration rate, surface ponding and runoff occurs. Runoff is the movement of water from precipitation across the Earth's surface into stream channels, lakes, oceans, or other depressions and low points on the Earth's surface.

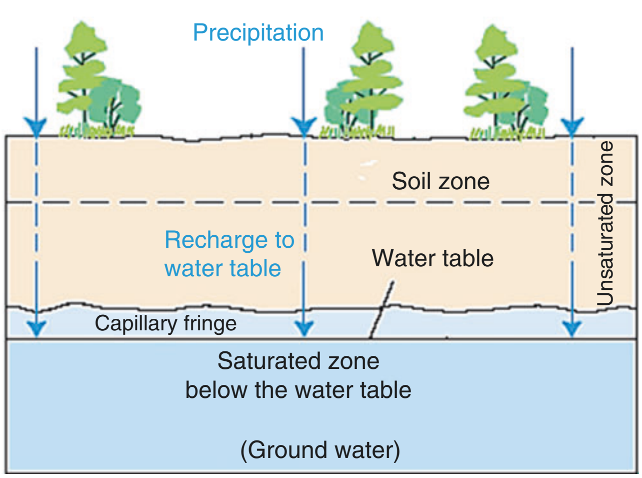
In the worldwide scheme of the hydrologic cycle, runoff from snowmelt is a major component of the global movement of water. Of course, the importance of snowmelt varies greatly geographically, and in warmer climates, it does not directly play a part in water availability except at high altitudes. In colder climates, though, much of the springtime runoff and streamflow in rivers is attributable to melting snow and ice. Until melting takes place, snow and ice represent a temporary storage of water.

Infiltration is the entry of water into the soil surface. Some water that infiltrates will remain in the shallow soil layer, where it will gradually move vertically and horizontally through the soil and subsurface material. In fact, the upper layer of the soil is the unsaturated zone, where water is present in varying amounts that change over time but does not saturate the soil.

Some of the water may infiltrate deeper, recharging groundwater aquifers. (Fig. 4) Groundwater is a part of the water cycle. Some part of the precipitation that lands on the ground surface infiltrates into the subsurface (Toth 1963). The part that continues downward through the soil until it reaches rock material that is already saturated is groundwater recharge. Water in the saturated groundwater system moves slowly and may eventually discharge into streams, lakes, and oceans (Waller 1982).

If the aquifers are porous enough to allow water to move freely wells can be used to make use of the water resource. Aquifers are a huge storehouse of Earth's water, and people all over the world depend on groundwater in their daily lives.

Generally, infiltration and runoff are controlled by soil type, thickness, original water content, and precipitation characteristic.



Water, Fig. 4 Ground water (Reference: http://water.usgs.gov)

Water Balance

The water balance is an accounting of the inputs and outputs of water. This accounting is also referred to as a water budget. The water balance of a place, whether it is an agricultural field, region, or continent, can be determined by calculating the input and output of water. The major input of water is from precipitation and outputs are runoff and evapotranspiration (Willmott and Rowe 1985). The water balance equation is as follows:

$$ET = P + R_1 - R_2 + R_g - F - \Delta S$$

Where ET is evapotranspiration, P is precipitation (e.g., Rainfall), R_1 is input water, R_2 is output water, R_g is deep inactive groundwater, F is infiltration water, and ΔS is change in soil water content (soil storage). If the volume of water is increased, ΔS is positive and if it decreased, it will be negative.

The Ocean, Lake and River, Watershed and Streamflow

The ocean is a storehouse of water. The water in the oceans moves because of waves, which are driven by winds. Ocean currents move massive amounts of water around the world. These movements have a great deal of influence on the water cycle.

Lakes are where surface-water runoff (and maybe some groundwater seepage) have accumulated in a low spot, relative to the surrounding countryside. It is not that the water that forms lakes get trapped but that the water entering a lake comes in faster than it can escape, either via outflow in a river, seepage into the ground, or by evaporation. And if humans live nearby, then water levels can be affected by water withdrawals for human needs (Wetzel 2001).

Climate, atmospheric inputs, underground rock and soils in the watershed, physiography, land use, lake morphometry (such as: size, shape, and depth) are some ofthe most important basic factors that give unique character to each lake ecosystem.

A river forms from water moving from a higher altitude to a lower altitude, due to gravity. When rain falls on the land, it either seeps into the ground or becomes runoff, which flows downhill into rivers and lakes, on its journey towards the seas (Wetzel 2001).

A watershed is an area of land that drains the streams and rainfall to a common outlet such as the outflow of a reservoir, mouth of a bay, or any point along a stream channel. A watershed is a precipitation collector. Not all precipitation that falls in a watershed flows out. There are many factors that determine how much water flows in a stream. Some of these are: precipitation, infiltration, soil characteristics, soil saturation, land cover, slope of the land, evaporation, transpiration, storage, and water use by people.

Streamflow is always changing, from day to day and even minute to minute. Of course, the main influence on stream flow is precipitation runoff in the watershed. Rainfall causes rivers to rise, and a river can even rise if it only rains very far up in the watershed – because water that falls in a watershed will eventually drain by the outflow point (Gohari et al. 2013).

Summary and Conclusions

Water is one of the most important substances on Earth. All plants and animals must have water to survive. Apart from drinking water survive, people have many other uses. Water is necessary to keep people, livestock, crops, ecosystems and the environment healthy and should be valued and protected as a precious resource. What is important is that water for irrigation and food production constitutes one of the greatest pressures on freshwater resources. World agriculture consumes approximately 70% of the freshwater withdrawn per year (UNESCO 2001). Population growth, climate change as a result of greenhouse gases, and inaccurate management are among the factors that cause frequent droughts with long periods of time. These factors exacerbate drought, especially in arid and semiarid regions. Therefore, to increase water use efficiency in agriculture, the irrigation management must be optimized to avoid unnecessary waste of important and sometimes limited water resources (Saccon 2017).

Cross-References

▶ Aquifer
▶ Evaporites
▶ Groundwater
▶ Hydrogeology
▶ Hydrology
▶ Infiltration
▶ Water
▶ Water Testing
▶ Wells

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Water Testing

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Definition

The various procedures used to analyze samples of surface water (lakes, streams, and rivers); groundwater (from springs and wells); and wastewater for physical, biological, and chemical constituents.

Water testing may be undertaken to evaluate the background or environmental water quality, characteristics of water before or after treatment (including water for human consumption – drinking water), and suitability of water for industrial uses such as laboratory, manufacturing, or equipment cooling.

The sampling protocol and methods used are important to overall success of the testing and assessment process. The collection and analysis should be conducted in accordance with established procedures and standards specifically required by local, state, or national laws and regulation (Schulze et al. 2011; USDA-NRCS 2012). The water sampled is representative of the variability of the chemical, physical, or biological systems observed in the temporal and spatial distribution of the water source. Preparations are necessary to describe and run quality control/quality assurance (QC/QA) procedures on all the processes for collecting, storing, transporting, and testing samples; and reporting test results.

The selection of parameters for testing should be consistent with the test method and applicable for the intended purpose or concerns identified. It is essential the results be reliable and

Water Testing, Fig. 1 CTC Analytics COMBI PAL auto sampling or sample preparation system used for chemical and physical analyses of water in a Stable Isotope Laboratory. Photograph courtesy of Illinois State Geological Survey

compared to analytical references for estimating the uncertainty of measurement. Following testing standards, such as from American Society for Testing and Materials (ASTM) International https://www.astm.org/Standards/water-testing-standards.html, US Environmental Protection Agency (US EPA) https://www.epa.gov/dwanalyticalmethods, and International Organization for Standardization (ISO) https://www.iso.org/ics/13.060.45/x/p/1/u/1/w/1/d/0, assures the measurement results are of the same quality as the time of validation (Fig. 1).

Cross-References

- ▶ Acidity (pH)
- ▶ Aquifer
- ▶ Geochemistry
- ▶ Groundwater
- ▶ Hydrogeology
- ▶ Hydrology
- ▶ Pollution
- ▶ Water

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Wells

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Definition

Wells are excavations into the Earth for developing groundwater, oil, brine, gas, and engineering geology operations (Driscoll 1986; Hunt 2005).

Water wells are dug, driven, bored, or drilled into the saturated zone of water-bearing formations (aquifers) to extract groundwater for drinking and irrigation (Barrocu 2014; Campbell and Lehr 1973).

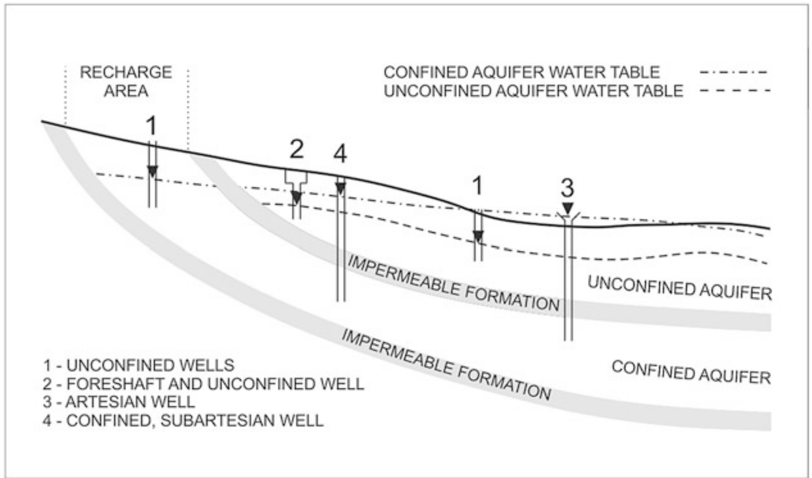
Shallow water wells are excavated into loose and soft rocks by hand, or mechanically, mainly in unconfined aquifers. They are rarely deeper than 50–60 m, normally with a circular section of 1–3.5 m in diameter. Their walls are supported by masonry or a precast concrete ring lining to prevent collapsing. The lining often extends above the ground to prevent people from falling into the well and to reduce contamination.

In dug wells, if the water table becomes lower due to abstraction the well can be deepened or new wells can be dug (Fig. 1). In the ancient past, water was brought to the surface by buckets lowered into the well with a rope by hand, pulley, or a counterbalanced sweep (shadoof), but today typically by pumping.

In Middle Eastern Qanat systems, aligned water wells are excavated down to an infiltration gallery consisting of a low gradient draining tunnel that collects groundwater by gravity flow (Angelakis et al. 2016). Driven wells are constructed by hammering down joint perforated pipe sections into unconsolidated materials.

Drilling methods depend on purposes, rock characteristics, well depth, the type of samples needed (rock chips or core samples), penetration rates, and costs. Augering methods are adopted in geotechnical engineering and mineral prospecting in unconsolidated materials. Rotary Air Percussion drilling is common for mineral exploration, water wells, and blast-hole

Wells, Fig. 1 Water wells in unconfined and confined aquifers



drilling. Cable Tool drilling is suitable for large diameter water wells in low yielding aquifers. Diamond Core drilling and Reverse Circulation drilling are convenient to core samples for geotechnical purposes and in geological and mineral prospecting. Direct Push drilling, including Cone Penetration Testing (CPT) and direct push sampling rigs, are limited to perforate unconsolidated and very soft rocks. Hydraulic Rotary drilling rigs are best for oil and gas exploration and production. Sonic drilling easily penetrates in most geological formations by fluidizing soil particles.

Drilled wells are cased with steel or plastic/PVC pipes prevent loose materials collapsing into the borehole. Groundwater from aquifer formations enters the casing through slotted sections.

In artesian or flowing wells, the water level rises above the ground level so that groundwater overflows. In contrast, in subartesian wells (Fig. 1), submersible pumps are needed, as their water level rises above the confining formation of the aquifer without reaching the ground surface (Barrocu 2014). Wellpoint patterns are installed for dewatering during construction (Driscoll 1986; Hunt 2005).

Piezometers in wells are often used for measuring water table levels (piezometric heads). In observation wells, one can observe any physical or chemical parameter variations. Injection wells are drilled for artificial aquifer recharge, to increase the natural infiltration rate. Infiltration or diffusion wells are recharge wells sunk only into the unsaturated zone. In civil and mining engineering, they are used to strengthen rocks by grouting under pressure, in waste disposal, and geothermal energy.

Cross-References

- ▶ Artesian
- ▶ Aquifer
- ▶ Boreholes
- ▶ Borehole Investigations
- ▶ Cone Penetrometer
- ▶ Desert Environments
- ▶ Dewatering
- ▶ Drilling
- ▶ Drilling Hazards
- ▶ Fluid Withdrawal
- ▶ Groundwater
- ▶ Hydrocompaction
- ▶ Hydrogeology
- ▶ Hydrology
- ▶ Piezometer
- ▶ Reservoirs
- ▶ Saturation
- ▶ Voids
- ▶ Water
- ▶ Water Testing

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Y

Young's Modulus

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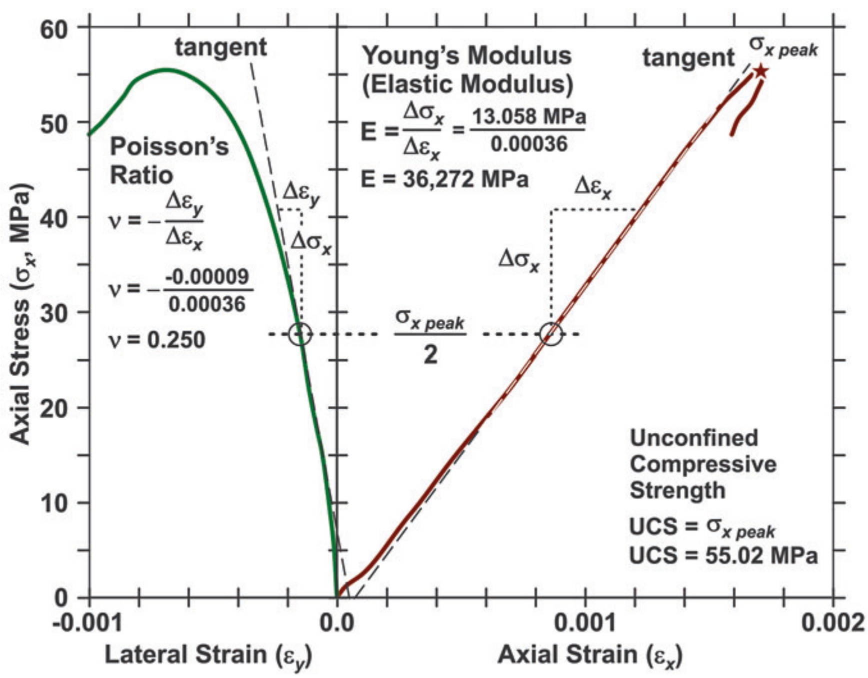
Definition

Young's modulus (E) is the ratio of principal stress in one direction (σ_x) to corresponding strain in the elastic range in the same direction (ε_x) (ASTM 2014); it is named after the individual who identified it as an important material property, Thomas Young (1773–1829). It is also known as the elastic modulus. A plot of stress and strain from a laboratory uniaxial compression test (Fig. 1) displays an elastic range where axial deformation is completely or largely recoverable and a plastic range where deformation is permanent. The laboratory test data in the example demonstrate the tangent Young's modulus at half the value of the unconfined compressive strength (Pariseau 2012).



Young's Modulus, Fig. 1 Unconfined compressive strength test results of a core sample of Cretaceous silty sandstone from a fluvial environment of deposition. Axial stress plotted against axial strain (positive x-axis) is used to define the unconfined compressive strength (UCS) at the peak value of the stress-strain curve. Axial stress plotted against lateral strain (negative x-axis) is used to define the Poisson's ratio as the slope of the stress-lateral strain curve corresponding to the midpoint of the stress-strain curve used to determine the UCS value

P. T. Bobrowsky, B. Marker (eds.), *Encyclopedia of Engineering Geology*,
https://doi.org/10.1007/978-3-319-73568-9

Cross-References

- ► Modulus of Elasticity
- ► Poisson's Ratio
- ► Strain
- ► Stress

References

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Z

Zone of Influence

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Definition

An area of engineering soils likely to be affected by loading due to engineering or building development.

It is the designer's task to define the bounds of a domain that should be considered in any computational analysis of ground loading. For that purpose, a zone of influence is defined.

Highly sophisticated approaches, especially numerical procedures, have been created to solve this task. All require input of material parameters and prescription of boundary and initial conditions. It is, therefore, necessary to estimate the influence zone. This, however, requires the solution of problems.

It has been experimentally confirmed that soil substantially changes its material properties when subjected to external loading. Apart from that, when subjected to a certain loading history, soil has the ability to "memorize" the highest past level of loading. This property is known as preconsolidation and is mathematically represented by the over-consolidation ratio. Hydraulic and mechanical properties of the soil are described in detail in Terzaghi et al. (1996).

In a virgin state, the soil deformability is relatively high. On the contrary, following unloading/reloading the state shows almost negligible deformation until the highest stress state the soil has so far experienced is reached. When using standard recommendations in a design (e.g., in the analysis of settlement of foundation subsoil), the depth and width of the influence zone (Fig. 1) must be specified.

The estimation of soil structure interaction generally has many stages. The solution is time dependent so it is necessary to first determine the initial stage, that is, the stage with zero deformations. A certain virtual state of virgin soil can be established by considering gravity loads only in this computational step. In software terminology, this is termed as the geostatic procedure. Further details are provided in, for example, the Abacus user guide (http://bobcat.nus.edu.sg:2080/English/SIMACAEANLRefMap/simaanl-c-geostatstress.htm).

Once the initial effective stress state, usually termed the effective geostatic stress state, is established, it is possible to estimate the depth of influence zone. Assuming a linear behavior of the subsoil opens the way to the application of elastic layer theory. The formula for a quick estimate of the influence zone depth is presented in Ramesh et al. (2017) together with information concerning the estimated width of the influence zone. It is accompanied by an example. The effect of preconsolidation and its validation using triaxial testing is given in detail in Kuklik (2011), which also briefly outlines the theory of elastic layer and verification using finite element modeling.

Other complementary factors that affect the influence zone over the life of the structure, for example, water table variations, water flow, weathering, etc., are mentioned in Bowles (1996).

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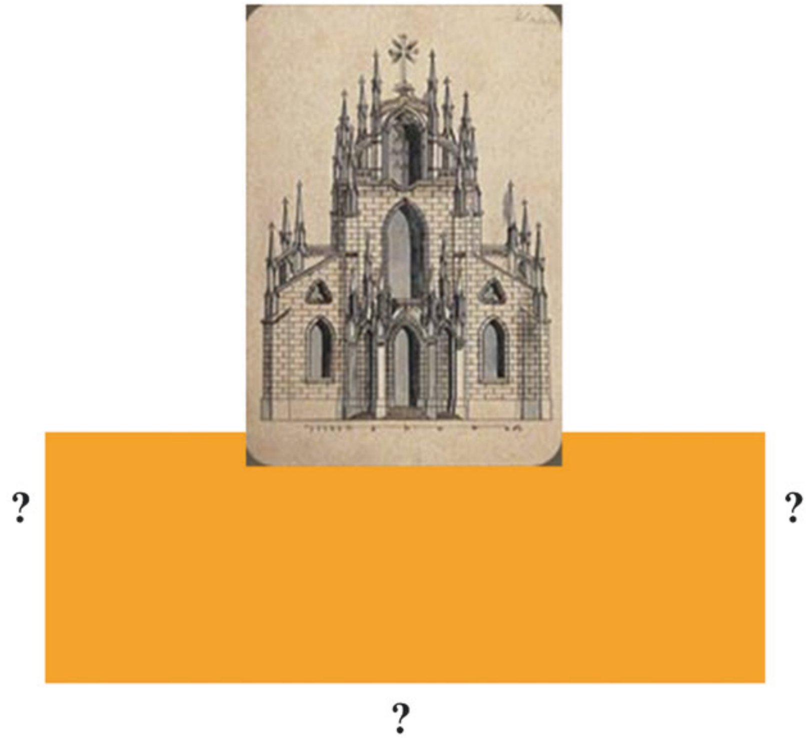
Zone of Influence, Fig. 1 Depth and width of influence zone



Cross-References

- ▶ Foundations
- ▶ Soil Properties
- ▶ Stress

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