Building to Resist the Effect of Wind, Volume 2: Estimation of Extreme Wind Speeds and Guide to the Determination of Wind Forces

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Building To Resist The Effect Of Wind

VOLUME 2. Estimation of Extreme Wind Speeds and Guide to the Determination of Wind Forces

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Building To Resist The Effect Of Wind

In five volumes

VOLUME 2: Estimation of Extreme Wind Speeds and Guide to the Determination of Wind Forces

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ABSTRACT

The Agency for International Development sponsored with the National Bureau of Standards, a three and a half year research project to develop improved design criteria for low-rise buidings to better resist the effects of extreme winds.

Project results are presented in five volumes. Volume 1 gives a background of the research activities, accomplishments, results, and recommendations. In Volume 3, a guide for improved use of masonry fasteners and timber connectors are discussed. Volume 4 furnishes a methodology to estimate and forecast housing needs at a regional level. Socio-economic and architectural considerations for the Philippines, Jamaica, and Bangladesh are presented in Volume 5.

Volume 2 consists of two reports. The first reviews the theoretical and practical considerations that are pertinent to the estimation of probabilistically defined wind speeds. Results of the statistical analysis of extreme wind data in the Philippines are presented and interpreted. Recommendations based on these results are made with regard to the possible redefinition of wind zones, and tentative conclusions are drawn regarding the adequacy of design wind speeds currently used in the Philippines. Report two describes some of the more common flow mechanisms which create wind pressures on low-rise buildings and the effects of building geometry on these pressures. It is assumed that the basic wind speeds are known and a procedure is outlined for calculating design wind speeds which incorporates the expected life of the structure, the mean recurrence interval, and the wind speed averaging time. Pressure coefficients are tabulated for various height-to-width ratios and roof slopes. The steps required to calculate pressures and total drag and uplift forces are summarized and an illustrative example is presented.

Key words: Building codes; buildings; codes and standards; housing; hurricanes; pressure coefficients; probability distribution functions; risk; statistical analysis; storms; structural engineering; tropical storms; wind loads; wind speeds.

Cover: Instruments to measure wind speed and direction being installed on a 10 meter mast at the project test site in Quezon City, Philippines.

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Facing Page: A wind sensor is installed on the wall of a test house in Quezon City, Philippines. Pressures acting on walls and on the roof of the test building are converted by these sensors into electrical signals which are recorded on magnetic tape.

1. ESTIMATION OF EXTREME WIND SPEEDS—APPLICATION TO THE PHILIPPINES

by
E. Simiu

1.1 INTRODUCTION

In modern building codes and standards [1, 2] basic design wind speeds are specified in explicitly probabilistic terms. At any given station a random variable can be defined, which consists of the largest yearly wind speed. If the station is one for which wind records over a number of consecutive years are available, then the cumulative distribution function (CDF) of this random variable may, at least in theory, be estimated to characterize the probabilistic behavior of the largest yearly wind speeds. The basic design wind speed is then defined as the speed corresponding to a specified value F_0 of the CDF or, equivalently (in view of the relation $N=1/(1-F_0)$ in which $N=$ mean recurrence interval), as the speed corresponding to a specified mean recurrence interval. For example, the American National Standard A58.1 [1] specifies that a basic design wind speed corresponding to a 50-year mean recurrence interval (i.e., to a value F_0 of the CDF equal to 0.98, or to a probability of exceedance of the basic wind speed in any one year equal to 0.02) be used in designing all permanent structures, except those structures with an unusually high degree of hazard to life and property in case of failure, for which a 100-year mean recurrence interval ($F_0 = 0.99$) must be used, and structures having no human occupants or where there is negligible risk to human life, for which a 25-year mean recurrence ($F_0 = 0.96$) may be used. A wind speed corresponding to a N-year recurrence interval is commonly referred to as the N-year wind.

The mean recurrence intervals specified by building codes, rather than being based on a formal risk analysis—which is in practice not feasible in the present state of the art—are selected in such a manner as to yield basic wind speeds which, by professional consensus, are judged to be adequate from a structural safety viewpoint. Nevertheless, it is generally assumed that adequate probabilistic definitions of design wind speeds offer, at least in theory, the advantage of insuring a certain degree of consistency with regard to the effect of the wind loads upon structural safety. This is true in the sense that, all relevant factors being equal, if appropriate mean recurrence intervals are used in design, the probabilities of failure of buildings in different wind climates will, on the average, be the same.

In the practical application of the probabilistic approach to the definition of design wind speeds, certain important questions arise. One such question pertains to the type of probability distribution best suited for modeling the probabilistic behavior of the extreme winds. The provisions of the National Building Code of Canada [2] are based upon the assumption that this behavior is best modeled by a Type I (Gumbel) distribution. The American National Standard A58.1 [1], on the other hand, assumes that the appropriate models are Type II (Frechet) distributions with location parameters equal to zero and with tail length parameters dependent only upon type of storm. Finally, Thom [29] has proposed a model consisting of a mixed probability distribution, the parameters of which are functions of (a) the frequency of occurrence of tropical cyclones in the 5° longitude-latitude square under consideration and (b) the maximum average monthly wind speed recorded at the station investigated. The question of selecting the most appropriate distribution is one that deserves close attention: indeed, as indicated in References 23 and 22, the magnitude of the basic design wind speed may depend strongly upon the probabilistic model used.

Assuming that the type of probability distribution best suited for modeling the behavior of the extreme winds is known, a second important question arises, viz., that of the errors associated with the probabilistic approach to the definition of design wind speeds. Such errors depend primarily upon the quality of the data and upon the length of the record (i.e., the sample size) available for analysis.

These questions will be dealt with in this work, which will also present results of statistical analyses of wind speed data recorded in the Philippines. In the light of the material presented herein, possible approaches will be examined to the definition of extreme wind speeds for purposes of structural design in the Philippines.

1.2 WIND SPEED DATA

For the statistical analysis of extreme wind speeds to be meaningful, the data used in the analysis must be reliable and must constitute an homogeneous set. The data may be considered to be reliable if:

- The performance of the instrumentation used for obtaining the data (i.e., the sensor and the recording system) can be determined to have been adequate.
- The sensor was exposed in such a way that it was not influenced by local flow variations due to the proximity of an obstruction (e.g., building top, ridge or instrument support).

A set of wind speed data is referred to herein as homogeneous if all the data belonging to the set may be considered to have been obtained under identical or equivalent conditions. These conditions are determined by the following factors, which will be briefly discussed below:

- type of instrumentation used
- averaging time (i.e, whether highest gust, fastest mile, one-minute average, five-minute average, etc. was recorded).
- height above ground
- roughness of surrounding terrain (exposure)
- in the case of tropical cyclone winds, distance inland from the coastline.

1.2.1 Type of instrumentation

If, during the period of record, more than one type of instrument has been employed for obtaining the data, the various instrument characteristics (anemometer and recorder) must be carefully taken into account and the data must be adjusted accordingly.

1.2.2 Averaging Time

If various averaging times have been used during the period of record, the data must be adjusted to a common averaging time. This can be done using graphs such as those presented in Reference 19 and included in figure 1 in which Z_0 is a parameter defining the terrain roughness (see, for example, Ref. 10).

1.2.3 Height Above Ground

If, during the period of record, the elevation of the anemometer had been changed, the data must be adjusted to a common elevation as follows: Let the roughness length and the zero plane displacement be denoted by Z_0 and Z_d, respectively (Z_0, Z_d are parameters which define the roughness of terrain. see Ref. 10). The relation between the mean wind speeds $U(Z_1)$ and $U(Z_2)$ over horizontal terrain of uniform roughness at elevation Z_1 and Z_2 above ground, respectively, can be written as

$$\frac{U(Z_1)}{U(Z_2)} = \frac{\ln \frac{Z_1 - Z_d}{Z_0}}{\ln \frac{Z_2 - Z_d}{Z_0}} \qquad (1)$$

Suggested values of the roughness length Z_0 are given in table 1 (see refs. 10,21,7). For example, at Sale, Australia, for terrain described as open grassland with few trees, at Cardington, England, for open farmland broken by a few trees and hedge rows, and at Heathrow Airport in London, $Z_0 = 0.08$ m [10, 21]. At Cranfield, England where the ground upwind of the anemometer is open for a distance of half a mile across the corner of an airfield, and where neighboring land is broken by small hedged fields, $Z_0 = 0.095$m [9]. The values of Z_0 for built-up terrain should be regarded as tentative. It is noted that Equation 1 is applicable to mean winds and should not be used to represent the profiles of peak gusts.

Table 1. Suggested Values of Z_0 for Various Types of Exposure

Type of Exposure	Z_0 (meters)
Coastal	0.005-0.01
Open	0.03- 0.10
Outskirts of towns, suburbs	0.20- 0.30
Centers of towns	0.35- 0.45
Centers of large cities	0.60- 0.80

The zero plane displacement Z_d may in all cases be assumed to be zero, except that in cities (or in wooded terrain) $Z_d = 0.75$ h, where h = average height of buildings in the surrounding area (or of trees) [10, 16]. Thus, for example, if in open terrain with $Z_0 = 0.05$ m, $U(23) = 30$ m/s, then adjustment of this value to the height $Z_2 = 10$ m, using Equation 1, gives

$$U(10) = U(23) \frac{\ln \frac{10}{0.05}}{\ln \frac{23}{0.05}} = 30 \frac{5.30}{6.13} = 25.9 \text{ m/s.}$$

It is noted that, in most cases, the roughness parameters Z_0, Z_d must be estimated subjectively, rather than being determined from measurements. Good judgment and experience are required to keep the errors inherent in such estimates within reasonable bounds. In conducting statistical studies of the extreme winds, it is advisable that for any particular set of data, an analysis be made of the sensitivity of the results to possible errors in the estimation of Z_0 and Z_d.

In the case of winds associated with large-scale extratropical storms, the mean wind $U(Z)$ at height Z in terrain of roughness Z_0, Z_d is related as follows to the mean wind $U_1(Z_1)$ at height Z_1 in terrain of roughness Z_{01}, Z_{d_1} [21]:

$$U(Z) = \beta\, U_1(Z_1) \frac{\ln \frac{Z - Z_d}{Z_0}}{\ln \frac{Z_1 - Z_{d_1}}{Z_{01}}} \qquad (2)$$

The quantity, β, may be obtained from figure 2, which was developed in Reference 21 on the basis of theoretical and experimental work reported by Csanady [4] and others [26]. (Note that $Z_{01} < Z_0$.)

Equation 2 may be applied if the roughness of the terrain is homogeneous over a horizontal distance from the anemometers of about 100 times the anemometer elevation [18, 24].

Let, for example, $U(Z_1) = 29$ m/s, $Z_1 = 10$ m, $Z_{01} = 0.05$ m, $Z_{d_1} = 0$. The corresponding speed $U(Z)$ at $Z = 40$ m, say, in open terrain of roughness $Z_0 = 0.25$ m, $Z_d = 0$ is

$$U(10) = 1.12 \times 29 \frac{\ln \frac{40}{0.25}}{\ln \frac{10}{0.05}} = 31.1 \text{ m/s.}$$

where 1.12 is the value of β for $Z_{01} = 0.05$ m, $Z_0 = 0.25$ m, obtained from figure 2.

It is pointed out that, just as in the case of Equation 1, errors are inherent in Equation 2 that are associated with the subjective estimation of the roughness parameters. Also, recent research suggests that in the case of tropical cyclone winds Equation 2 underestimates wind speeds over built-up terrain, calculated as functions of speeds over open terrain, by amounts of the order of 15% or more[17].

1.2.4 Distance Inland from the Coastline

The intensity of hurricane or typhoon winds is a decreasing function of the distance inland from the coastline. Hurricane wind speeds may be adjusted to a common distance from the coastline by applying suitable reduction factors. Such reduction factors have been proposed by Malkin, according to whom the ratios of peak gusts at 48, 96 and 144 km from the coastline to peak gusts at the coastline are 0.88, 0.82 and 0.78, respectively [8, 14].

1.3 PROBABILISTIC MODELS OF EXTREME WIND SPEEDS

The nature of the variate suggests that an appropriate model of extreme wind behavior is provided by probability distributions of the largest values, the general expression for which is [11]:

$$F(v) = \exp\left\{-[(v-\mu)/\sigma]^{-\gamma}\right\} \qquad \begin{array}{l} \mu < v < \infty \\ -\infty < \mu < \infty \\ 0 < \sigma < \infty \\ \gamma > 0 \end{array} \qquad (3)$$

where v = wind speed, μ = location parameter, σ = scale parameter, γ = tail length parameter. Equation 3 may be regarded as representing a family of distributions, each characterized by a value of the tail length parameter γ. As γ becomes larger, the tail of the probability curve becomes shorter, i.e., the probability of occurrence of large values of the variate becomes smaller. In particular, as $\gamma \rightarrow \infty$, Equation 3 may be shown to become

$$F(v) = \exp\left\{-\exp[-(v-\mu)/\sigma]\right\} \qquad \begin{array}{l} -\infty < v < \infty \\ -\infty < \mu < \infty \\ 0 < \sigma < \infty \end{array} \qquad (4)$$

The distributions given by Equations 3 and 4 are known as the type II and the type I distributions of the largest values, respectively.

Two basic procedures for estimating probabilities of occurrence of extreme winds are currently in use. The first procedure consists in estimating the parameters of a probability distribution of the largest values from the series of annual highest wind speeds at the station considered. This procedure has been applied by various authors as follows:

(a) In Reference 23, estimates are made of all three parameters, μ, σ and γ in Equation 3, no specific value being assigned *a priori* to any of these parameters.

(b) In References 27 and 28, the location parameter is assumed to have the value $\mu = 0$. Estimates are then made of the remaining parameters, σ and μ. The arbitrary assumption that $\mu = 0$ entails a sacrifice in goodness of fit; the justification for using this assumption is that it makes possible the application of Lieblein's well-known estimation procedure [13] to obtain values of σ and γ [27]. However, in view of the recent development of alternative estimation procedures applicable to type II distributions with $\mu \neq 0$ [23], the assumption that $\mu = 0$ becomes unnecessary.

(c) Court in the United States [3], Davenport in Canada [5] and Kintanar in the Philippines [12] have assumed the universality of the type I distribution, i.e., that the tail length parameter is always $\gamma = \infty$. Estimates are then made of the parameters μ and σ.

The second procedure assumes the universal validity of the mixed distribution

$$F(v) = p_E \exp\left[-\left(\frac{v}{\sigma}\right)^{-9}\right] + p_T \exp\left[-\left(\frac{v}{\sigma}\right)^{-4.5}\right] \qquad (5)$$

proposed by Thom in Reference 29. The first and the second term in the sum of Equation 5 represent the probabilities that the winds associated with extratropical storms and with tropical cyclones, respectively, will not exceed the value, v, in any one year. The scale parameter, σ, is an explicit function of the maximum of the average monthly wind speeds recorded at the station considered. The second parameter of the mixed distribution, p_T, is an explicit function of the frequency of occurence of tropical cyclones in the 5° longitude-latitude square under consideration, and $p_E = 1 - p_T$. Thus, in this second procedure, the series of annual highest winds is not used for estimating distribution parameters.

An assessment of the models described in this section will now be presented.

1.4 ASSESSMENT OF PROCEDURES BASED ON THE ANNUAL HIGHEST SPEEDS

To assess the validity of current probabilistic models, statistical analyses of annual highest speeds were carried out using a computer program described in Reference 23. The results of the analyses, which are reported in detail in Reference 23, lend credence to the belief that a sufficiently long record of annual largest speeds will provide an acceptable basis for probabilistic extimates of the N-year winds—even for large values of N, such as are of interest in structural safety calculations—if the following conditions are satisfied. First, the value of opt (γ) for that record is large, say $\gamma \geq 40$ (opt(γ) = value of γ [see eq. 3] for which the best distribution fit of the largest values is obtained). Second, meteorological information obtained at the station in question, as well as at nearby stations at which the wind climate is similar, indicates that winds considerably in excess of those reflected in the record cannot be expected to occur except at intervals many times larger than the record length. Wind climates which satisfy these two conditions will be referred to as well-behaved.

Assuming that the wind speed data are reliable, lower bounds for the sampling error in the estimation of the N-year winds in a well-behaved climate may be calculated on the basis of a mathematical result, the Cramer-Rao relation, which states that for the type I distribution (see ref. 11, p. 282)

$$\mathrm{var}(\hat{\mu}) \geq \frac{1.10867}{n}\sigma^2 \qquad (6)$$

$$\mathrm{Var}(\hat{\sigma}) \geq \frac{0.60793}{n}\sigma^2 \qquad (7)$$

where var ($\hat{\mu}$), var ($\hat{\sigma}$) are the variances of $\hat{\mu}$ and $\hat{\sigma}$, where $\hat{\mu}$ and $\hat{\sigma}$ are the estimated values of μ and σ, respectively, obtained by using any appropriate estimator consistent with basic statistical theory requirements; σ is the actual value of the scale parameter and n is the sample size. Using Equations 6 and 7, lower bounds for the standard deviation of the sampling error in the estimation of the N-year wind, $SD[v(N)]$, can be approximated as follows. Equation 4, in which the parameters μ, σ are replaced by their estimates μ, σ, is inverted to yield

$$v(N) = \hat{\mu} - G(1-\frac{1}{N})\hat{\sigma} \qquad (8)$$

where

$$G(1-\frac{1}{N}) = -\ln[-\ln(1-\frac{1}{N})] = \qquad (9)$$

Then

$$SD[v(N)] \geq [\mathrm{var}(\hat{\mu}) + [G(1-\frac{1}{N})]^2\,\mathrm{var}(\hat{\sigma})]^{1/2} \qquad (10)$$

Equation 10 is based on the assumption that the error involved in neglecting the correlation between μ and σ is small. The validity of this assumption was verified by using Monte Carlo simulation techniques.

Since the actual value of σ is not known, in practical calculations the estimated value σ is used in Equations 6 and 7. For example, the distribution parameters corresponding to the wind speed data at Davao (n = 24, see table 2), estimated by using the technique described in Reference 23, are $\hat{\mu}$ = 38.89 km/hr, $\hat{\sigma}$ = 9.40 km/hr. It follows from Equation 8 that v (50) = 75 km/hr and from Equations 6, 7, and 10 that $SD[v(50)] \geq$ 5.18 km/hr. Subsidiary calculations not reported here have shown that Equation 10 provides a good indication of the order of magnitude of the sampling errors.

1.4.1 Wind Climates Characterized by Small Values of opt (γ).

Occasionally, a record obtained in well-behaved wind climates may exhibit small values of opt (γ); this will occur if that record contains a wind speed that corresponds to a large mean recurrence interval. There are regions, however, in which, as a rule, the statistical analysis of extreme wind records taken at any one station yields small values of opt (γ). This is the case if, in the region considered, winds occur that are meteorologically distinct from, and considerably stronger than the usual annual extremes. Thus, in the regions where tropical cyclones occur, opt (γ) will in general be small, unless most annual extremes are associated with tropical cyclone winds. An example of a record for which γ (opt) is small is given in figure 3a, which represents the probability plot with γ = opt (γ) = 2 for the annual extreme fastest mile-speeds recorded in 1949-73 at the Corpus Christi, Texas, airport. For purposes of comparison, the same data have been fitted to a type I distribution (opt (γ) = ∞, or Eq. 4); the fit in this case is seen to be exceedingly poor, i.e., the plot deviates strongly from a straight line (fig. 3b). As shown in Reference 23, a measure of the goodness of fit is given by the extent to which the probability plot correlation coefficient is close to unity; this coefficient is printed out in figures 3a and 3b.

To small values of the tail length parameter there frequently correspond implausibly high values of the estimated speeds for large recurrence intervals. In the case of the 1912-48 record at Corpus Christi, for example, opt (γ) = 2 and the estimated 5-minute average is 327 mph (155 m/s) for a 1000-year wind, which is highly unlikely on meteorological grounds. For 20-year records, the situation may be even worse: thus, for the 1917-36 Corpus Christi record, which contains an exceptionally high wind speed due to the 1919 hurricane [3, 25], opt (γ) = 1 and the calculated 1000-year wind is 1952 mph (873 m/s) [23], a ridiculous result. Also, the situation is not likely to improve significantly if the record length increases. From a 74-year record, a plot quite similar to figure 3 would presumably be obtained, with twice as many points similarly dispersed, to which there would correspond a similar least squares line on probability paper.

It may be stated, consequently, that while in the case of well-behaved climates it appears reasonable to infer from a good fit of the probability curve to the data that the tail of the curve adequately describes the extreme winds, such an inference is not always justified if opt (γ) is small.

It may be argued that one could avoid obtaining unreasonable extreme values by postulating that the annual largest winds are described by a probability distribution of the type I, i.e., by assigning the value $\gamma = \infty$ to the tail length parameter. This has been done by Court [3] and Kintanar [12]. As can be seen in figure 4, the corresponding fit may be quite poor. However, the estimated extremes at the distribution tails will be reduced. The drawback of this approach is that unreasonably low estimated extremes may be obtained. For example, at Key West, Florida, if all three parameters of Equation 3 are estimated as in Reference 23, to the 1912-48 record there corresponds $v(100) = 99$ mph (44.2m/s) and $v(1000) = 188$ mph (84 m/s)—see Reference 23. If it is postulated that $\gamma = \infty$, then $v(50) = 70$ mph (31.7 m/s), $v(100) = 77$ mph (34.4 m/s) and $v(1000) = 97$ mph (38.8 m) [23], an unlikely result in view of the high frequency of occurrence of hurricanes (about 1 in 7 years) at Key West.

It may also be argued that since the estimated extremes resulting from small values of γ (say $\gamma < 4$) may be too large, and those corresponding to $\gamma = \infty$ may be too small, a probability distribution that might yield reasonable results is one in which γ has an intermediate value, say $4 < \gamma < 9$. Such an approach has been proposed by Thom and will now be examined.

1.5 ASSESSMENT OF PROCEDURE BASED ON THE HIGHEST AVERAGE MONTHLY SPEED

The procedure for estimating extreme winds in hurricane-prone regions on the basis of annual highest winds at a station was seen to have the following shortcomings. First, because hurricane winds are relatively rare events, the available data may not contain wind speeds associated with major hurricane occurrences and are therefore not representative of the wind climate at the station considered (see the case of Calapan in Section 1.7 of this report). Second, in regions subjected to winds that are meteorologically distinct from, and considerably stronger than the usual annual extremes, implausible estimates may be obtained.

The model proposed by Thom [Eq. 5] in Reference 29 represents an attempt to eliminate these shortcomings. It can be easily shown by applying the intermediate value theorem, that if this model is assumed, the estimated extreme winds may be obtained by inverting an expression of the form:

$$F(v) = \exp[(-v/\sigma)^{-\gamma(v)}]$$

in which $4.5 < \gamma(v) < 9$. If the mean rate of arrival of tropical cyclones in the region considered is high, then $\gamma(v)$ will be closer to 4.5. Otherwise, $\gamma(v)$ will be closer to 9; in regions where hurricanes cannot be expected to occur, $\gamma(v) = 9$. In order that estimates not be based upon possibly unrepresentative annual extreme data, Thom's model does not make use of annual extreme speeds. Rather, the parameter σ is estimated from the maximum of the average monthly wind speeds on record at the location considered, presumably a quantity for which the variability is small.

While the quasi-universal climatological distribution proposed by Thom is tentative, it will yield results which, for a first approximation, may in certain cases be regarded as acceptable. This model has recently been used by Evans [6] as a basis for obtaining design wind speeds for Jamaica. Estimates of extreme speeds obtained by Evans are substantially higher than the results obtained by Shellard [20] in his 1971 analysis of Commonwealth Caribbean wind data.

It was shown in the preceding section that the approach which utilizes the series of annual largest speeds may fail in regions in which hurricanes occur. For such regions, therefore, it may be that alternative approaches need to be developed. Among such approaches is one in which estimates of extreme winds are based upon the following information:

- average number of hurricanes affecting the coastal sector considered (per year)
- probability distribution of hurricane intensities
- radial dimensions of hurricanes
- dependence of wind speeds upon central pressure and distance from hurricane center.

This approach appears to provide useful estimates of extreme winds corresponding to large recurrence intervals—which are of interest in ultimate strength calculations—and is currently under study at the National Bureau of Standards.

1.6 STATISTICAL ANALYSIS OF EXTREME WIND DATA IN THE PHILIPPINES

Through the courtesy of the Philippine Atmospheric, Geophysical and Astronomical Services Administration (PAGASA), 16 sets of data were obtained consisting of maximum yearly wind speeds recorded during at least 14 consecutive years. The data for each of the 16 stations are listed in table 2. Table 3 includes subjective station descriptions provided by PAGASA personnel and the results of the analysis. In Table 3 are listed $V_N^{opt(\gamma)}$= N-year wind based on the distribution for which the best fit of the largest values is obtained and V_N^{∞} = N-year wind based on the type I distribution, N = mean recurrence interval in years.

TABLE 2. MAXIMUM ANNUAL WINDS (ONE MINUTE AVERAGES)

NO.	Station	Period of Record	Maximum Annual Winds for Each Year of Record (km/hour)
1	Davao	1950-73	39,52,40,39,40,37,35,35,32,40,40,40,80,48,48,48,56,46,52,50,46,52,46, 46
2	Cagayan de Oro	1950-73	47,24,19,13,19,19,12,12,12,19,16,14,21,6,24,17,19,37,37,46,37,48,41,41
3	Zamboanga	1950-73	48,64,40,39,48,61,43,40,48,45,48,72,48,48,50,56,68,67,70,56,61,74,61,78
4	Pasay City	1950-73	89,103,89,92,72,72,64,72,97,72,81,66,69,74,130,65,80,111,83,74,200,80,111,56
5	Manila	1949-70	72,105,97,89,101,97,100,105,81,72,97,89, 121,105,100,168,74,89,107,111,96,200
6	Manila Central	1902-40	46,56,65,80,73,55,77,70,41,68,50,69,64,68,42, 41,67,54,70,83,58,53,60,51,45, 63,50,70,52,55,58,37,100,60,45,52,103,53,56
7	Mirador	1914-40	79,79.70.79,121,102,94,72,105,107,122,58,89,107,93,63,77,92,63,53,93,73,75, 49,39,68,83
8	Baguio	1950-73	97,105,56,97,97,81,64,64,61,48,97, 48,53,98,111,107,80,144,107,102,137,85,89,133
9	Calapan	1959-73	145,185,97,72,40,68,40,96,109,41,33,111,102,111,83
10	Surigao	1952-73	105,43,113,40,40,40,43,48,64,64,89,39,74,52,78,104,167,111,96,107,96,74
11	Laoag	1949-73	34,72,118,71,108,100,64, 81,81,64,81,79,64,105,90,144,144,78,120,137,100,67,89,67,70
12	Iloilo	1949-73	97,106,64,64,97,64,64,64,72,74,64,78,78,89,74,.61,52,61, 89,56,74
13	Cebu	1950-73	97,56,121,48,81,64,48,48,48,43,48,43,64,64,56,48,56,70,93,65,56,100,74,59
14	Legaspi	1955-73	40,97,97,48,129,185,97,82,89,104,74,89,148,74,70,174,148,204,81
15	Tacloban	1949-73	137,58,106,113,56,68,60,47,48,69,87,42,105,93,74,111, 70,194,133,167,63,81,155,104,67
16	Infanta	1960-73	60,43,45,50,128,39,133,126,46,46,189,85,104,83

TABLE 3. STATION DESCRIPTIONS AND ESTIMATED EXTREME WIND SPEEDS

No.	Station	Wins Zone— See Ref. 15	Period of Record	No. of Years	Anemometer Elevation (meters)	Description of Terrain	opt (γ)	$V_N^{opt(\gamma)}$[e,f] (km/hr)			V_N^{∞}[f] (km/hr)		
								N=50	N=100	N=1000	N=50	N=100	N=1000
1	Davao	III	1950-73	24	10	Town	∞				75	82	105
2	Cagayan de Oro	III	1950-73								61	69	94
3	Zamboanga	III	1950-73	24	10	Open	∞				88	95	117
4	Pasay City	II	1950-73	24	10	Airport	2	220	292	820	164	180	234
5	Manila	II	1949-70	22	Top of 5-story building	Port Area	7	208	238	365	192	212	276
6	Manila Central	II	1902-40	39	6[a]	Open (Park)	35	103	112	144	101	110	138
7	Mirador Baguio	II	1914-40	27	10[b] (1500 m above sea level)	Mountain top	∞				139	151	192
8	Baguio	II	1950-73	24	10	Moutain top (1500 m above sea level)	∞				164	180	230
9	Calapan	II	1959-73	15	10	Top of hill; town on one side.	∞				209	234	316
10	Surigao	II	1952-73	22	7	Large City Top of 40m hill	40	170	191	262	167	187	250
11	Laoag	II	1949-73	25	12	Open	∞				174	192	252
12	Iloilo	II	1950-73	24	25 (5m above 20m bldg.)	Town	2	180	235	640	138	152	192
13	Cebu	II	1950-73	24	10	Airport	∞			127	140	185	
14	Legaspi	I	1955-73	19	12	Airport[c]	90	235	264	350	235	264	340
15	Tacloban	I	1949-73	25	12 (3m above 9m bldg.)	Top of 15m hill Military Camp[d]	14	213	242	352	204	228	306
16	Infanta	I	1960-73	14	10	Town; Residential area	6	242	290	488	214	242	333

[a]3 cup anemometer; mean speed averaged over one minute.
[b]Mean speed averaged over one minute.
[c]Trees at East side of anemometer.
[d]North and East: sea exposure.
[e] Omitted if opt (γ) = ∞.
[f] One minute averages.

1.7 INTERPRETATION OF RESULTS

The results will be grouped into three classes, according to the wind zone (as defined in Ref. 15) in which the stations considered are located (table 3).

1.7.1 Zone III

It is noted that for all three Zone III stations listed in Table 3, opt (γ) = ∞. It is convenient to adjust the speeds at Davao and Cagayan de Oro to open terrain exposure. On the basis of the terrain descriptions of Table 3, if it is assumed $Z_0 = 0.30$ m, $Z_d = 0$, $Z_{01} = 0.08$ m, $Z_{d_1} = 0$, it follows from Equation 2 that

$$\frac{U(10)}{U\,(10)} = 0.8$$

where $U(10)$, $U_1(10)$ are mean speeds above ground in town and open exposure, respectively. Thus, in Davao and Cagayan de Oro the calculated 50-year mean speeds at 10 m above ground in open terrain are 58 mph and 47 mph (94 km/hr and 76 km/hr) respectively, versus 55 mph, (88 km/hr) in Zamboanga. If the corresponding highest gusts are obtained by multiplying the one-minute means by a factor of, say, 1.20 (see fig. 1), the estimated highest 50-yr gusts at Davao, Cagayan de Oro and Zamboanga are at most $94 \times 1.20 = 113$ km/hr, (70mph), i.e., considerably lower than the value specified for design purposes by the National Structural Code of the Philippines [15] for Zone III, viz., 95 mph (153 km/hr). This suggests that the requirements of Reference 15 regarding wind loading in the Zone III portion of Mindanao are conservative and might be somewhat reduced. (It can be easily shown, on the basis of Eq. 2 and figure 1, that this statement holds even if it is assumed that the errors in the estimation of the parameter values $Z_0 = 0.30$ m and $Z_{01} = 0.08$ m are of the order of as much as 50%.) To validate such a conclusion it would however be necessary to determine, from long-term records of tropical cyclone occurences, that the 1950-73 data at the three stations analyzed are indeed representative for southern Mindanao.

1.7.2 Zone II.

Several difficulties arise in interpreting the results for the Zone II stations in table 3. It is noted, first, that the results obtained at stations in and near Manila (stations 4, 5, 6 in table 3) are widely divergent. The discrepancies between the results for Pasay City and Manila may be due to the different elevations of the respective anemometers. It may also be conjectured that the discrepancies between these results and those obtained from the 1902-1940 Manila Central record are due to differences in the averaging times and in the exposure, elevation and calibration of the instruments, as well as to possibly inaccurate estimates of the maximum speed in Manila and Pasay City in 1970 (200 km/hr, see table 2).

The estimated wind speeds at Baguio based upon the 1950-73 record are higher than those obtained from the 1914-40 data. No explanation is offered for these differences; an investigation into their causes seems warranted.

The record at Calapan illustrates the limitations of the approach to the definition of design wind speeds based on the statistical analysis of the highest annual winds. From the data covering the period 1961-72, the estimated 50-yr wind based on a Type I distribution is 88 mph (141 km/hr) [12], versus 131 mph (209 km/hr), as obtained if the data covering the period 1959-1973 are used (see table 3). Since wind loads are proportional to the square of the wind speeds, the ratio between the respective estimated winds loads is $(209/141)^2 = 2.2$.

Although the record at Pasay City is best fitted by a type II distribution with opt (γ) = 2, it is unlikely, as noted previously, that such a distribution correctly describes the behavior of the extreme winds. This is obvious, particularly in the case of the 1000-yr wind, which, on physical grounds, could not possibly attain 509 mph (820 km/hr) (see table 3).

The National Structural Code of the Philippines specifies, for Zone II and elevations under 9.15 m, a design wind of 109 mph (175 km/hr). In the light of the data shown in table 2, the value appears to be reasonable. It will be noted that tables 2 and 3, and figure 13 of Reference 15 indicate that the extreme speeds and the frequency of occurrence of tropical cyclones, are considerably higher at Laoag than at Cebu. This suggests that Zone II could be divided, accordingly, into two subzones, with wind load requirements higher in the northern than in the southern subzone.

1.7.3 Zone I.

As indicated previously, if γ(opt) is small, i.e., if the differences among maximum wind speeds recorded in various years are large, the probability distributions that best fit the data may not describe correctly the extreme wind speeds for large recurrence intervals. The minimum and the maximum winds for the period of record are, at Legaspi, 25 mph (40 km/hr) and 127 mph (204 km/hr), respectively, and, at Tacloban, 26 mph (42 km/hr) and 120 mph (194 km/hr), respectively. In the writer's opinion, the reliability of the N-year wind estimates obtained at these stations for N=50, 100 and 1000 is therefore doubtful. The same comment applies to the estimates for Infanta, where the record length is quite insufficient (14 years). The writer therefore believes that the results of table 3 should not be used to assess the adequacy of the design wind speed requirement for Zone I specified in Reference 15. Rather, it is reasonable to base such an assessment on a comparison between wind speeds in Zone I and in areas affected by hurricanes in the United States. In the light of U.S. experience, it is the opinion of the writer that from such a comparison it follows that the 124 mph (200 km/hr) wind speed requirement for Zone I and elevations under 30 ft. (9.15 m) is adequate for structural design purposes.

1.8 CONCLUSIONS

From the analysis of available extreme speed data in the Philippines, the following conclusions may be drawn:

1. The design wind speeds specified by the National Structural Code of the Philippines for the Zone III part of Mindanao appear to be conservative and might be somewhat reduced. For this conclusion to be validated, it would be necessary to determine, from long-term records of tropical cyclone occurrences, that the data analyzed herein are representative for southern Mindanao.
2. Methodological difficulties and uncertainties with regard to the reliability of the data preclude, at this time, the estimation for Zones II and I of *N*-year extreme winds that could be used, with a sufficient degree of confidence, as design values within the framework of an explicitly probabilistic code.
3. According to the data included herein, Zone II can be divided into two subzones, with wind load requirements higher in the northern than in the southern subzone.
4. The data included herein suggest that the wind speed requirement specified by the National Structural Code of the Philippines for Zone I is adequate for purposes of structural design, except as noted below.
5. Higher wind speed values than those specified by the National Structural Code of the Philippines should be used—except perhaps in the Zone III part of Mindanao—in open, and in coastal exposure.
6. Improved design criteria for Zones II and I, including possible redefinitions of these zones, could in the future be achieved by applying the methodology briefly described at the end of the section "Assessment of Procedure Based on the Highest Average Monthly Speed." This would require, in addition to data on the frequency of occurrence of tropical cyclones at various locations in the Philippines, that the following data be available:
   a. Reliable wind speeds, carefully defined with respect to terrain roughness, averaging time and distance from shore line.
   b. Approximate radial dimensions of tropical cyclones.
   c. Approximate dependence of tropical cyclone speeds upon minimum central pressure and distance from storm center.

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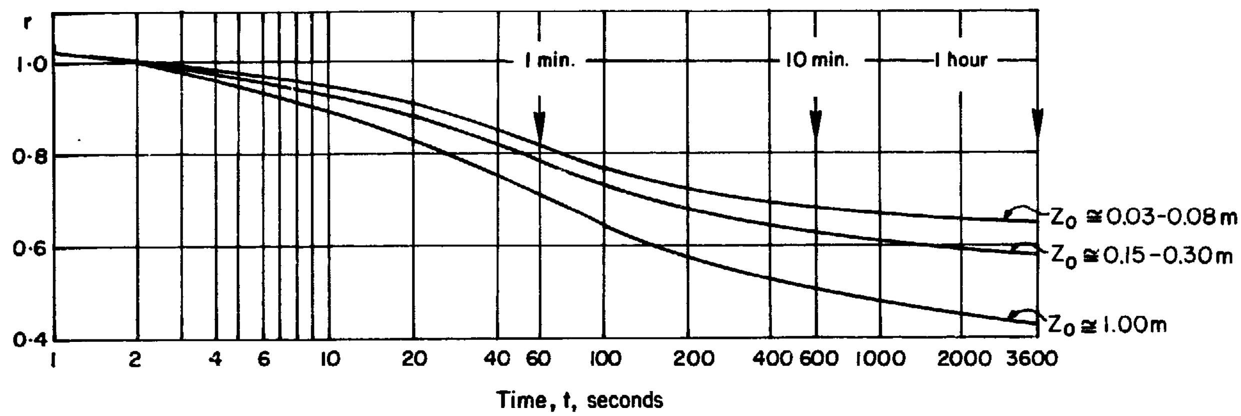


FIGURE 1. RATIO, r, OF MAXIMUM PROBABLE WIND SPEEDS AVERAGED OVER *t* SECONDS TO THOSE AVERAGED OVER 2 SEC.

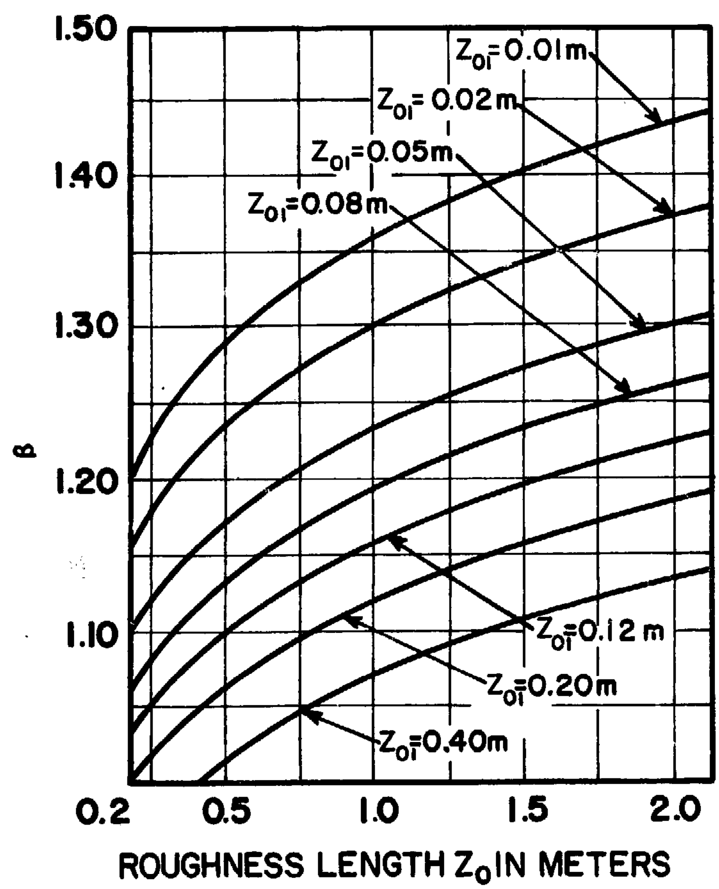


FIGURE 2. QUANTITY β

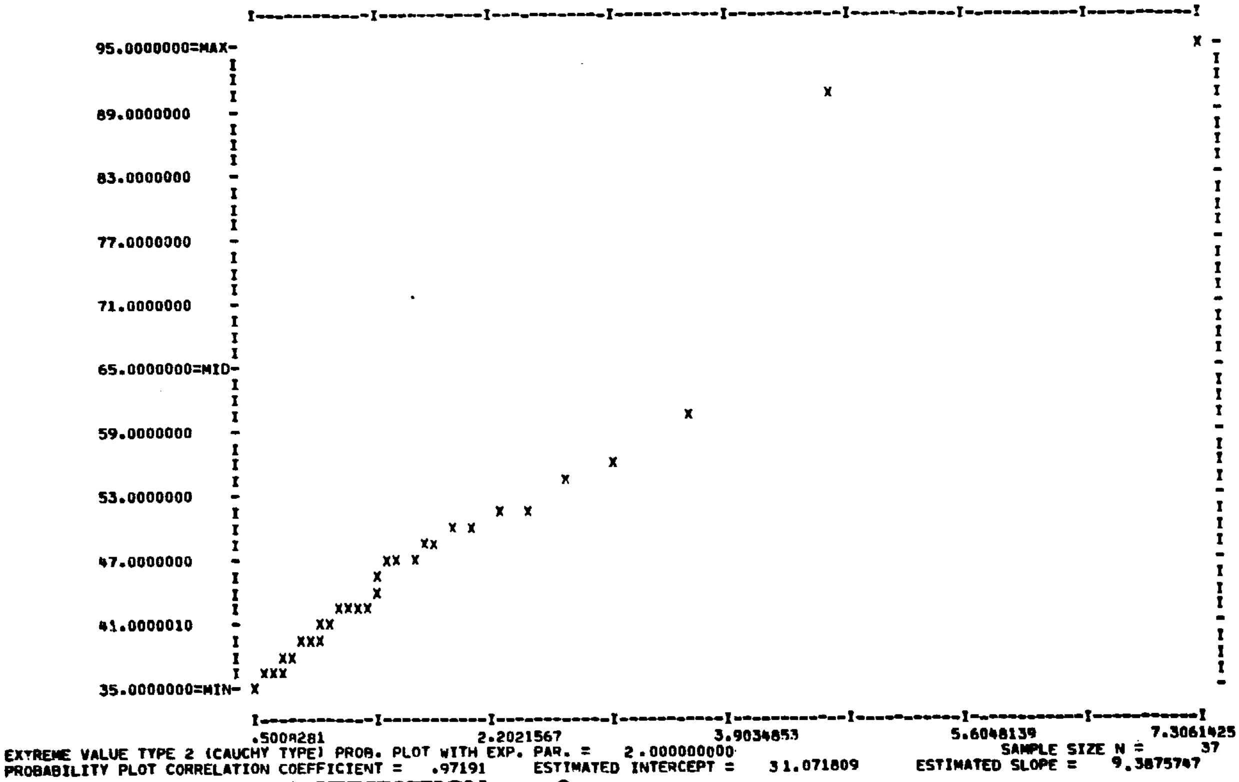


FIGURE 3a. TYPE II DISTRIBUTION, $\gamma = 2$.

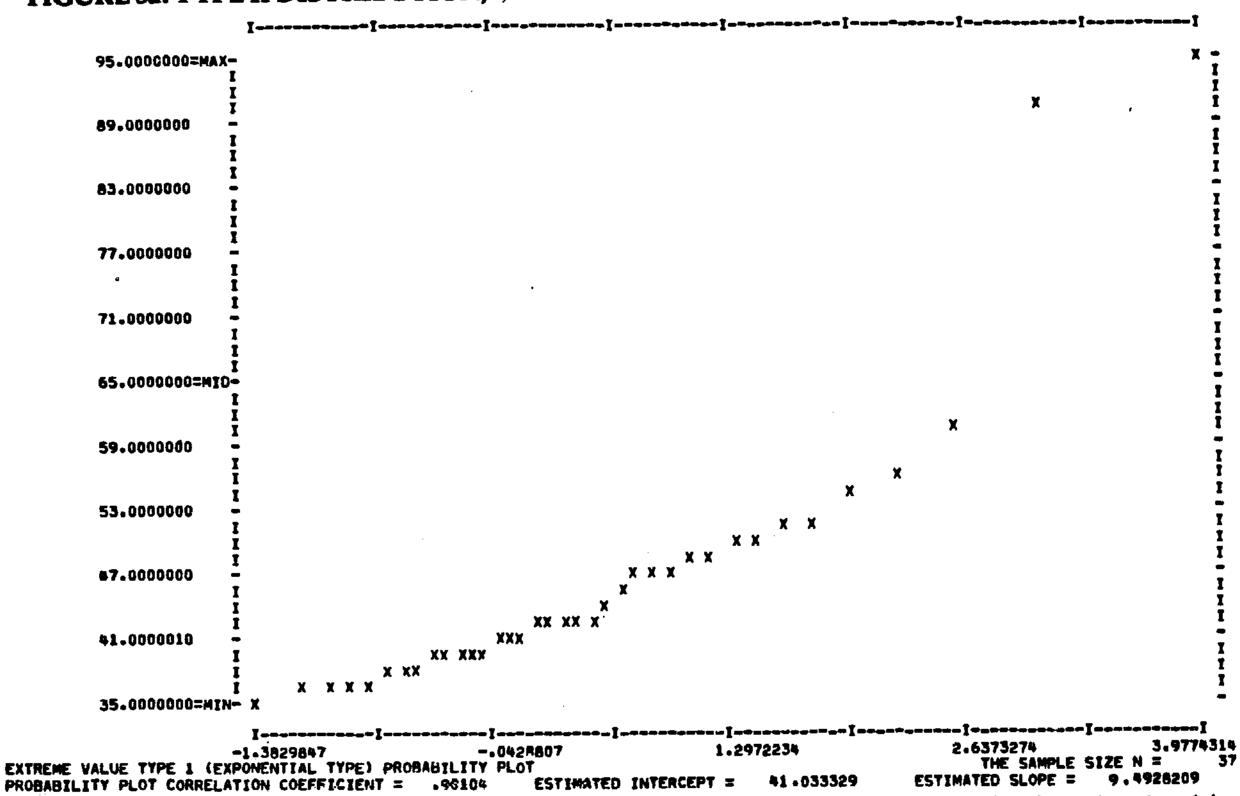


FIGURE 3b. TYPE I DISTRIBUTION.

Facing Page: This wind tunnel at the University of the Phillipines is used to study wind effects on scale-model buildings. Shown is a model of the CARE, Inc., test house. The rows of blocks on the floor of the tunnel generate turbulence or gustiness similar to that observed in full scale.

2. A GUIDE TO THE DETERMINATION OF WIND FORCES

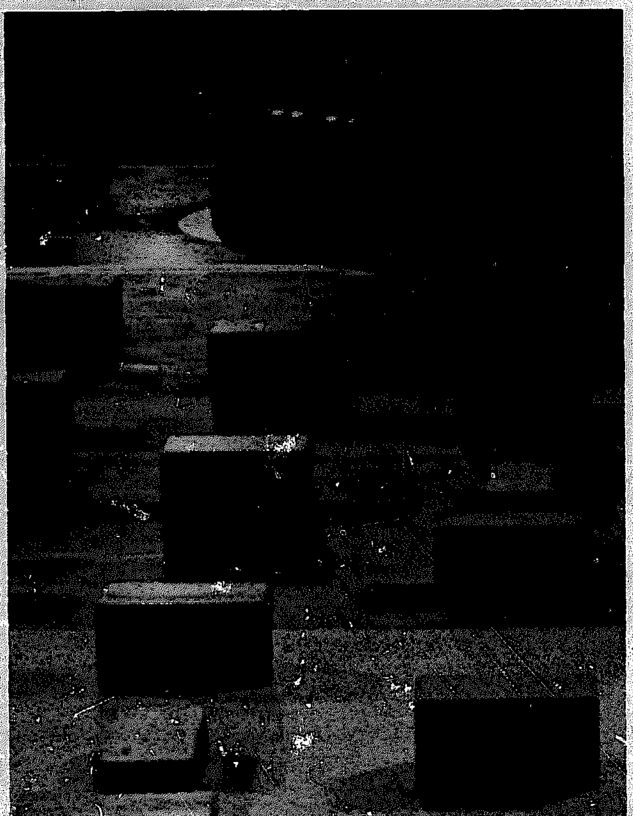
by
R. D. Marshall

2.1 INTRODUCTION

This paper deals with the nature of wind flow around buildings, the pressures generated by wind and the determination of forces acting on building elements as well as on the overall structure. It is assumed that buildings designed in accordance with the procedures outlined in the following sections and tables do not exceed 33 ft (10 m) in height or 164 ft (50 m) in plan dimension and have a height to width ratio (h/w) not exceeding four.

2.2 AERODYNAMICS OF BUILDINGS

The flow of wind around buildings is an extremely complex process and cannot be completely described by simple rules or mathematical formulae. Wide variations in building size and shape, type of wind exposure, local topography and the random nature of wind all tend to complicate the problem. Only by direct observation of full-scale situations or by resorting to properly conducted wind-tunnel tests can the characteristics of these flows be established. In spite of these complications, guidance can be provided by considering some typical flow situations.



2.2.1 Typical Wind Flow Around Buildings

A typical flow situation is illustrated in figure 4 where the wind is blowing face-on to a building with a gable roof. The flow slows down or decelerates as it approaches the building, creating a positive pressure on the windward face. Blockage created by the building causes this flow to spill around the corners and over the roof. The flow separates (becomes detached from the surface of the building) at these points and the pressure drops below atmospheric pressure, creating a negative pressure or suction on the endwalls and on certain portions of the roof.

A large low-pressure region of retarded flow is created downwind of the building. This region, called the wake, creates a suction on the leeward wall and leeward side of the roof. The pressures are neither uniform nor steady due to the turbulent character of the oncoming wind and varying size and shape of the

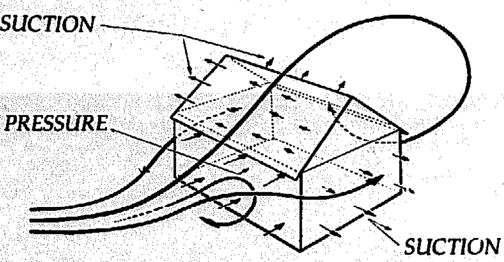


FIGURE 4. TYPICAL FLOW PATTERN AND SURFACE PRESSURES.

wake. However, it has been established that the patterns of wind flow around bluff bodies such as the building in figure 4 do not change appreciably with a change in wind speed.

This allows dimensionless pressure coefficients (to be discussed later) determined for one wind speed to be applied to all wind speeds. In general, the wind pressure is a maximum near the center of the windward wall and drops off rapidly near the corners. Pressures on the side or endwalls are also non-uniform, the most intense suctions occurring just downstream of the windward corners.

2.2.2 Effect of Roof Slope

The pressures acting on a roof are highly dependent upon the slope of the roof, generally being positive over the windward portion for slopes greater than 30 degrees. For slopes less than 30 degrees, the windward slope can be subjected to severe suctions which reach a maximum at a slope of approximately 10 degrees. Under extreme wind conditions, these suctions can be of sufficient intensity to overcome the dead weight of the building, thus requiring a positive tiedown or anchorage system extending from the roof to the foundation to prevent loss of the roof system or uplift of the entire building.

Intense suctions are likely to occur along the edges of roofs and along ridge lines due to separation or detachment of the flow at these points. For certain combinations of roof slope and wind direction, a conical vortex can be developed along the windward edges of the roof as shown in figure 5. This is a "rolling up" of the flow into a helical pattern with very high speeds and, consequently, very intense suctions. If not adequately provided for in the design, these vortices along the edges of the roof can cause local failures of the roofing, often leading to complete loss of the roof. Areas where intense suctions can be expected are shown in figure 6.

2.2.3 Roof Overhangs

In calculating the total uplift load on a roof, the pressure acting on the underside of roof overhangs must also be included. These pressures are usually positive and the resultant force acts in the same direction as the uplift force due to suction on the top surface of the roof. Pressures acting on the inside of the buiding (to be discussed later) can also contribute to the total uplift force and must likewise be accounted for.

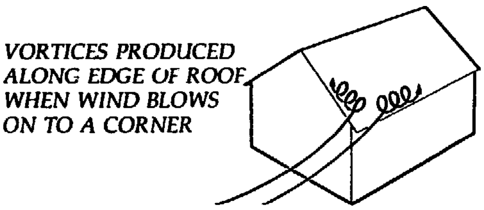


FIGURE 5. VORTICES ALONG EDGE OF ROOF.

FIGURE 6. AREAS OF INTENSE SUCTIONS.

2.3 DESIGN WIND SPEED

Several factors must be considered in selecting a wind speed on which to base the design loads for a building or other structure. These include the climatology of the geographic area, the general terrain roughness, local topographical features, height of the building, expected life of the building and acceptable level of risk of exceeding the design load. The assessment of climatological wind data and the procedure for obtaining basic wind speeds are discussed in section 1.0. The selection of the basic wind speed and the determination of modifying factors to obtain the design wind speed are discussed in the following sections.

2.3.1 Mean Recurrence Interval

The selection of a mean recurrence interval, with which there is associated a certain basic wind speed, depends upon the intended purpose of a building and the consequences of failure. The mean recurrence intervals in table 4 are recommended for the various classes of structures.

2.3.2 Risk Factor

There is always a certain risk that wind speeds in excess of the basic wind speed will occur during the expected life of a building. For example the probability that the basic wind speed associated with a 50-year mean recurrence interval will be exceeded at least once in 50 years is 0.63. The relationship between risk of occurrence during the expected building life and the mean recurrence interval is given in table 5. It should be noted that the risk of exceeding the basic wind speed is, in general, not equal to the risk of failure.

2.3.3 Averaging Time and Peak Wind Speed

It is well known that the longer the time interval over which the wind speed is averaged, the lower the indicated peak wind speed will be. The calculated design loads will thus depend upon the averaging time used to determine the design wind speeds. In this document, it has been assumed that all speeds used in pressure and load calculations are based upon an averaging time of 2 seconds. Wind speeds for averaging times other than 2 seconds can be converted into 2-second average speeds using the procedure described in section 1.0.

2.4 DESIGN PRESSURES

2.4.1 Dynamic Pressure

When a fluid such as air is brought to rest by impacting on a body, the kinetic energy of the moving air is converted to a dynamic pressure q, in accordance with the formula

$$q = 1/2\, \rho U^2 \qquad (1)$$

where $q = N/m^2$, ρ is the mass density of the air in kg/m^3 and U is the free-stream or undisturbed wind speed in m/s. The mass density of air varies with temperature and barometric pressure, having a value of 1.225 kg/m^3 at standard atmospheric conditions. In the case of tropical storms, the mass density may be 5 to 10 percent lower. However, this is offset somewhat by the effect of heavy rainfall, and the value quoted above should be used for all wind pressure calculations, i.e.,

$$q = 0.613\, U^2 \qquad (2)$$

2.4.2 Mean and Fluctuating Components of Pressure

As in the case of wind speed, pressures acting on a building are not steady, but fluctuate in a random manner about some mean value. A typical recording of wind speed and pressure at a point on the roof of a house is shown in figure 7.

A close inspection of figure 7 reveals the following characteristics:

(a) The average or mean pressure is negative (suction)

(b) Pressure fluctuations tend to occur in bursts

(c) Maximum departures from the mean are in the negative (suction) direction

(d) The peak values far exceed the mean value

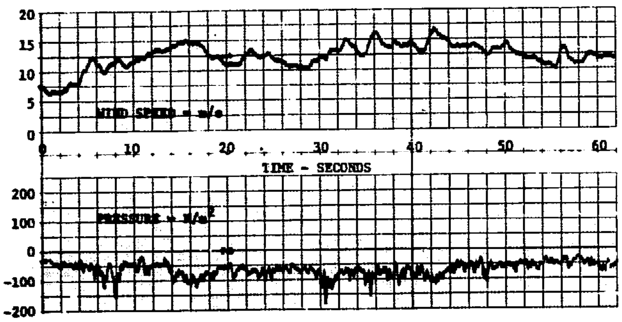


FIGURE 7. TYPICAL RECORD OF WIND SPEED AND SURFACE PRESSURE.

To quantify these pressures, it is essential that a sufficiently long time interval be used to obtain a stable mean, $\bar{p}$. The fluctuations are described by their standard deviation or root-mean-square, p_{rms}, taken about the mean. Finally, the peak pressure fluctuations are described by a peak factor, g, which indicates the number of standard deviations that the peak pressure deviates from the mean. Thus, the peak pressure can be expressed as

$$p_{max} = \bar{p} + g\,p_{rms} \text{ or } p_{min} = \bar{p} - g\,p_{rms} \qquad (3)$$

It should be noted that the peak factor, g, is a random variable and has a probability distribution function that depends on the geometry of the building and turbulent structure of the wind. The values of g are selected so that the associated probabilities of being exceeded are in line with the expected building life.

2.4.3 Pressure Coefficients

It is convenient to express pressures acting on the surfaces of a building in terms of the dynamic pressure as follows

$$p = C_p q \qquad (4)$$

where C_p is a pressure coefficient whose value depends upon the geometry of the building and local flow conditions. Pressure coefficients are specified for particular surfaces or elements of a building and, when multiplied by the surface area and dynamic pressure, give the wind loads acting in a direction normal to those surfaces or elements. The total resultant forces and moments acting on a building can then be determined by considering the appropriate components of these loads acting on each of the surfaces or elements.

As discussed in the previous section, the instantaneous peak pressure can be expressed in terms of the mean pressure and a fluctuating component. Since pressure fluctuations are limited in spatial extent, it is necessary to consider the size of the building surface or element when selecting the pressure coefficient.

Pressures on Extended Areas: For the purpose of determining wind loads acting on sizeable surface areas such as the walls and roof of a building, the pressure coefficients listed in tables 6 and 7 should be used. These coefficients have been determined experimentally from measurements taken on full-scale buildings and from wind-tunnel tests and they represent an upper limit of conditions likely to occur on the indicated building surfaces.

Pressures on Localized Areas: It is to be expected that the smaller the area considered, the larger the effective peak pressure will be. In addition, there are certain surface areas where intense suctions occur as pointed out in sections 2.2.1 and 2.2.2. To provide for these cases, pressure coefficients for localized areas are included in tables 6 and 7. These coefficients are for the purpose of assessing wind loads on local cladding and roofing elements and should not be used to calculate overall loads on buildings. They should be used in conjunction with the internal pressure coefficients (where appropriate) as described in the following.

Internal Pressures: As indicated in section 2.2.3 the net load or force acting on the roof or walls of a building depends not only on the external surface pressures, but on the internal pressure as well. The magnitude of the internal pressure depends upon the building geometry, size and location of openings, and wind speed and direction. As with external pressures, it is convenient to express internal pressures in terms of the dynamic pressure and a pressure coefficient C_{pi}. These coefficients can be positive or negative as indicated in table 8. The net pressure acting on a building element is the algebraic sum of the external and internal pressures

$$p = q(C_p - C_{pi}) \qquad (5)$$

Thus a positive internal pressure will increase the loading on those areas of roofs and walls subjected to external suction.

2.4.4 Correction Factor for Height of Building

The pressure coefficients described above are based on building heights of 33 ft (10 m) and peak wind speeds at 33 ft (10 m) above ground, averaged over 2 seconds. Overall loads calculated for buildings appreciably less than 33 ft (10 m) in height (measured to eaves or parapet) will thus be overestimated if these coefficients are used without modification. On the other hand, tributary areas such as doors, windows, cladding and roofing elements will respond to pressure fluctuations with duration times considerably less than 2 seconds. To account for this, the pressures must be multiplied by the correction factors, R, in table 9. Thus the expression for the net pressure acting on a building surface becomes

$$p = q(C_p R - C_{pi} R_i) \qquad (6)$$

and the force acting normal to a surface of area A is

$$F = q(C_p R - C_{pi} R_i)\,A \qquad (7)$$

where R and R_i are correction factors for external and internal pressures, respectively.

2.5 PROCEDURE FOR CALCULATING WIND FORCES

The procedure for calculating wind forces on a building is summarized in the following steps.

1. Select the appropriate mean recurrence interval from table 4
2. Check the associated factor of risk in table 5 and select a longer mean recurrence interval if appropriate.
3. Determine the basic wind speed for this mean recurrence interval and the appropriate terrain roughness and type of exposure as outlined in section 1.
4. Convert the resulting basic wind speed to a 2-second mean speed using the procedure described in section 1.
5. Calculate the dynamic pressure q using the expression

$$q = 0.613\ U^2$$

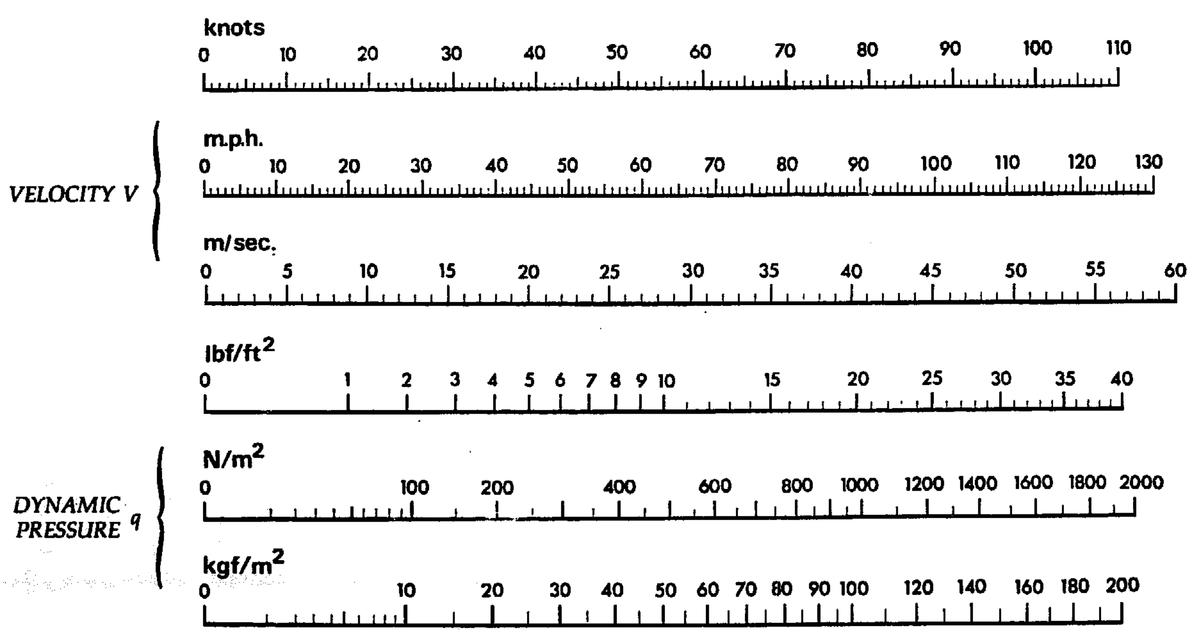
6. Select the appropriate pressure coefficients from tables 6, 7 and 8.
7. Select the appropriate correction factors from table 9.
8. Calculate the pressures from the expressions

$$p = q C_p R$$

or

$$p = q(C_p R - C_{pi} R_i)$$

9. Multiply these pressures by the respective surface areas to obtain the wind forces.
10. Sum appropriate components of these forces to obtain net uplift and drag loads.



CONVERSION CHART FOR WIND SPEED AND DYNAMIC PRESSURE HEAD

ACKNOWLEDGMENTS

Acknowledgment is made to the Building Research Establishment (UK) for the illustrations used in this document. The writer also wishes to acknowledge useful comments and suggestions provided by members of the Philippine Advisory Committee and by Dr. Emil Simiu of the Center for Building Technology.

TABLE 4 MEAN RECURRENCE INTERVAL

Class of structure	Mean recurrence interval years
All structures other than those set out below.	50
Structures which have special post-disaster functions, e.g. hospitals, communications buildings, etc.	100
Structures presenting a low degree of hazard to life and other property in the case of failure.	20

TABLE 5. RELATIONSHIPS BETWEEN RISK OF OCCURRENCE, MEAN RECURRENCE INTERVAL AND EXPECTED LIFE OF BUILDING

Desired Lifetime N Years	Risk of exceeding in *N* years the wind speed corresponding to the indicated mean recurrence interval					
	0.632	0.50	0.40	0.30	0.20	0.10
	Mean Recurrence Interval in Years					
10	10	15	20	29	45	95
20	20	29	39	56	90	190
50	50	72	98	140	224	475
100	100	144	196	280	448	949

Note: From this table it will be seen that there is a 10% risk that the wind speed corresponding to a mean recurrence interval of 475 years will be exceeded in a lifetime of 50 years.

TABLE 6. PRESSURE COEFFICIENTS FOR WALLS OF RECTANGULAR BUILDINGS

Building Height/Width Ratio	Building Length/width Ratio	Wind Angle α (Degrees)	C_p for Face				Local C_p
			A	B	C	D	
$h/w < 0.5$	$1 \leq l/w < 1.5$	0	0.8	-0.5	-0.6	-0.6	-1.2
		90	-0.6	-0.6	0.8	-0.5	
	$1.5 \leq l/w < 4$	0	0.8	-0.5	-0.8	-0.8	-1.3
		90	-0.4	-0.4	0.8	-0.4	
$0.5 \leq h/w < 1.5$	$1 \leq l/w < 1.5$	0	0.8	-0.5	-0.7	-0.7	-1.4
		90	-0.7	-0.7	0.8	-0.5	
	$1.5 \leq l/w < 4$	0	0.8	-0.6	-0.9	-0.9	-1.4
		90	-0.4	-0.4	0.8	-0.3	
$1.5 \leq h/w < 4$	$1 \leq l/w < 1.5$	0	0.8	-0.5	-0.8	-0.8	-1.5
		90	-0.8	-0.8	0.8	-0.5	
	$1.5 \leq l/w < 4$	0	0.8	-0.6	-0.9	-0.9	-1.5
		90	-0.4	-0.4	0.8	-0.3	

Notes: (1) h is the height to eaves or parapet, *l* is the greater plan dimension of the building and w is the lesser plan dimension.
(2) Local C_p values (last column) should be used in conjunction with correction factors for overall areas in Table 9.

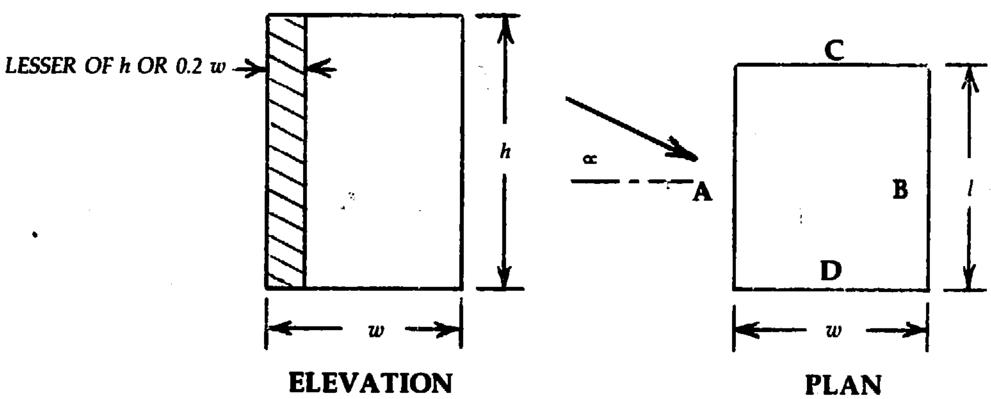
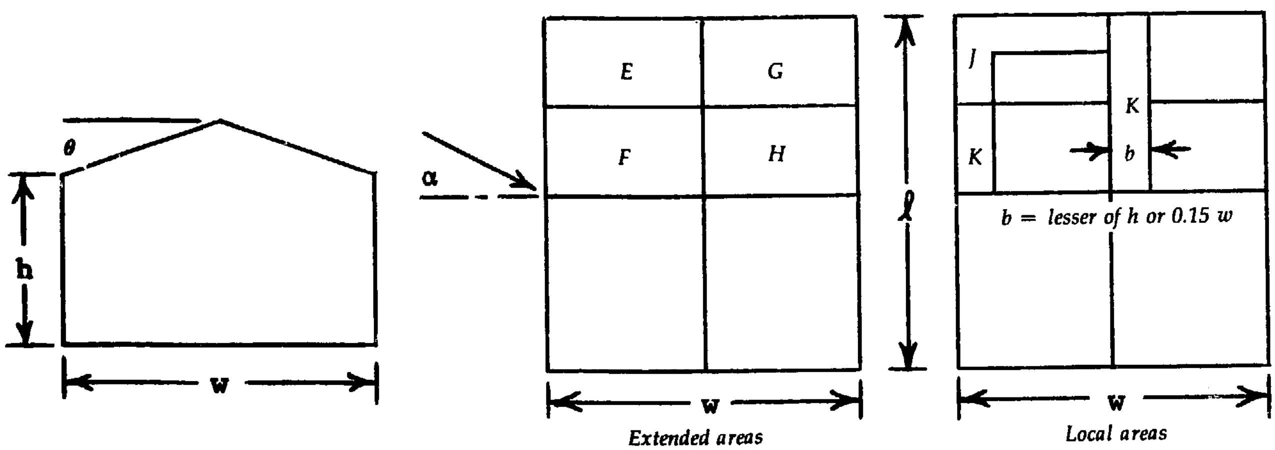


TABLE 7. PRESSURE COEFFICIENTS FOR ROOFS OF RECTANGULAR BUILDINGS

Building Height/Width Ratio	Wind Angle α (Degrees)	Area Designation	Roof Slope θ Degrees 0	10	20	25
h/w<0.5	0	EF	-1.0	-1.0	-0.4	-0.3
		GH	-0.6	-0.6	-0.8	-0.6
		J	-1.6	-1.9	-1.9	-1.6
		K	-1.4	-1.4	-2.0	-1.6
	90	EG	-1.2	-1.1	-1.0	-0.8
		FH	-0.6	-0.6	-0.6	-0.5
		J	-2.0	-1.8	-1.8	-1.6
		K	-1.4	-1.4	-2.0	-1.6
0.5≤h/w≤4	0	EF	-1.0	-1.0	-0.6	-0.4
		GH	-0.6	-0.6	-0.8	-0.8
		J	-1.8	-1.8	-1.8	-1.8
		K	-1.4	-1.4	-2.0	-1.6
	90	EG	-1.2	-1.1	-1.0	-0.8
		FH	-0.6	-0.6	-0.5	-0.4
		J	-1.8	-1.8	-1.6	-1.6
		K	-1.4	-1.6	-1.6	-1.6

Notes: (1) The pressure coefficient on the underside of roof overhangs should be taken as that on the adjoining wall surface.

(2) Local C_p values (*J* and *K*) should be used in conjunction with correction factors for overall areas in table 9.



ELEVATION

PLAN

TABLE 8. INTERNAL PRESSURE COEFFICIENTS FOR RECTANGULAR BUILDINGS

Condition	Internal pressure coefficient C_{pi}
1. Two opposite walls equally permeable, other walls impermeable: (a) Wind normal to permeable wall (b) Wind normal to impermeable wall	 +0.3 -0.3
2. Four walls equally permeable	-0.3 or +0.2 whichever is the more severe for combined loadings
3. Dominant opening on one wall, other walls of equal permeability: (a) Dominant opening on windward wall, having a ratio of permeability of windward wall to total permeability of other walls and roofs subject to external suction, equal to— 2 3 6 or more (b) Dominant opening on leeward wall (c) Dominant opening on side wall (d) Dominant opening in a roof segment	 +0.5 +0.6 +0.8 value of C_p for leeward external wall surface value of C_p for side external wall surface value of C_p for external surface of roof segment

Notes: (1) Internal pressures developed within an enclosed structure may be positive or negative depending on the position and size of the openings.

(2) In the context of table 8 the permeability of a surface is measured by the total area of openings in the surface under consideration.

(3) The value of C_{pi} can be limited or controlled to advantage by deliberate distribution of permeability in the wall or roof, or by the deliberate provision of a venting device which can serve as a dominant opening at a position having a suitable external pressure coefficient. An example of such is a ridge ventilator on a low-pitch roof, and this, under all directions of wind, can reduce the uplift force on the roof.

TABLE 9. CORRECTION FACTOR (R) FOR HEIGHT OF BUILDING

Terrain	Structural System	Area	h < 5	$5 \leq h \leq 10$
Smooth Zo $\leq$ 0.12 m	Walls	Overall Elements	0.85 1.00	1.00 1.20
	Roofs	Overall Elements	0.85 1.05	1.00 1.25
	Internal Pressure		0.85	1.00
Rough Zo > 0.12 m	Walls	Overall Elements	0.75 0.90	1.00 1.20
	Roofs	Overall Elements	0.75 0.95	1.00 1.25
	Internal Pressure		0.75	1.00

Notes: (1) The term "Overall" refers to the entire area of a given wall or roof slope.

(2) The term "Elements" refers to roof and cladding elements, doors, windows, etc.

(3) The terrain roughness parameter Z_0 must be estimated subjectively. The following values are suggested for various types of exposure.

TYPE OF EXPOSURE	Z_0 (meters)
Coastal	0.005-0.01
Open country	0.02-0.12
Outskirts of towns, suburbs	0.13-0.30
Centers of towns	0.40

APPENDIX A

ILLUSTRATIVE EXAMPLE

A housing development is to be located in flat, open country on the outskirts of Zamboanga, Philippines, and will ultimately consist of several hundred single-family dwellings of quite similar geometry. The period of construction is anticipated to be from 10 to 15 years. The basic plan dimensions are 6.2 × 7.5 m and the height to the eaves is 2.7 m. The gable roof has an overhang of 0.7 m on all sides and a slope of 10 degrees. Openings for doors and windows are evenly distributed on the exterior walls.

Because the development is to be built over a period of several years, it would not be appropriate to assume a built-up area in selecting the basic wind speed and flat, open country will be assumed here.

From table 4, a mean recurrence interval of 50 years is selected and it is considered that the associated risk of exceeding the basic wind speed (0.632) in table 5 is acceptable.

From section 1, the 1-minute average wind speed (N=50) for Zamboanga is 88 km/hr (Type I distribution). Since this is based on data obtained in open country at 10 m above ground, this speed can be converted directly to the design speed. Also, from section 1 the ratio of the 1-minute speed to the 2-second peak speed is 0.82. Thus the design speed is

$$U = 88/0.82 = 107.3 \text{ km/hr} = 29.8 \text{ m/s}$$

The dynamic presure is calculated from equation 2 of section 2.4.1

$$q = 0.613U^2 = (0.613)(29.8)^2 = 544 \text{ N/m}^2$$

Wind pressures are next calculated using equations 4-6 and the coefficients presented in table 6-9. Note that

$$h/w = 2.7/6.2 = 0.44$$

and

$$l/w = 7.5/6.2 = 1.21$$

WALLS

Inspection of tables 6 and 8 reveals that the worst cases are walls A and C with the wind blowing normal to the ridge. For wall A, C_p = 0.8 and for wall C, C_p = -0.6. The local C_p is -1.2. The internal pressure coefficients can range from 0.2 to -0.3. Table 9 indicates that the reduction factor is 0.85 for walls and internal pressures and 1.00 for cladding elements, doors, windows, etc.

For wall A,

$$p = (544)\,[0.8-(-0.3)](0.85) = 509 \text{ N/m}^2$$

For wall C,

$$p = (544)\,[-0.6-(0.2)](0.85) = -370 \text{ N/m}^2$$

For cladding elements, the worst cases are

$$p = (544)\,[0.8 - (-0.3)(0.85)] = 574 \text{ N/m}^2$$

and

$$p = (544)\,[-0.6 - (0.2)(0.85)] = -419 \text{ N/m}^2$$

For local pressures acting on strips of width 0.2 w = 1.2 m at each corner,

$$p = (544)(-1.2 - 0.2)(0.85) = -647 \text{ N/m}^2$$

ROOF

Inspection of table 7 reveals that the greatest uplift pressures on extended areas occur when the wind is blowing along the ridge.

For sections E and G,

$$p = (544)\,[-1.1 - (0.2)]\,(0.85) = -601 \text{ N/m}^2$$

For sections F and H,

$$p = (544)\,[-0.6 - (0.2)]\,(0.85) = -370 \text{ N/m}^2$$

Pressures acting on roofing elements in sections E and G are obtained as follows:

$$p = (544)\,[(-1.1)(1.05) - (0.2)(0.85)] = -727 \text{ N/m}^2$$

and for sections F and H,

$$p = (544)\,[(-0.6)(1.05) - (0.2)(0.85)] = -438 \text{ N/m}^2$$

Localized pressures act on strips of width 0.15 w = 0.93 m as shown in table 7. The worst case occurs for area J with the wind blowing normal to the ridge. Note that the uplift pressure under the eaves must also be included.

$$p = (544)\,[-1.9 - (0.8)]\,(0.85)$$
$$= -1.2k\ N/m^2$$

For area *K* in section F, this negative pressure or suction is slightly less

$$p = (544)\,[-1.4 - (0.8)]\,(0.85)$$
$$= -1.0k\ N/m^2$$

Along the ridge (area K), the localized pressure is

$$p = (544)\,[-1.4 - (0.2)]\,(0.85)$$
$$= -740\ N/m^2$$

TOTAL UPLIFT FORCE

The total uplift force on the building is calculated for the wind blowing normal to the ridge as follows:

$$\text{Area of one roof slope} = [7.5 + (2)(0.7)]\,[6.2/\,(2\text{Cos }10°) + 0.7]$$
$$= (8.9)(3.85)$$
$$= 34.2\ m^2$$

Note that areas E, F, G and H include areas J and K when calculating overall loads.

$$\text{Uplift} = (544)(1.0 + 0.6)(34.2)(\text{Cos }10°)(0.85) + (544)(6.2)(7.5)(0.2)(0.85)$$
$$= 29.2\ kN$$

TOTAL DRAG FORCE

The total drag force (neglecting the roof) is calculated as the sum of the loads on the windward and leeward walls.

$$\text{Drag} = (544)(2.7)(7.5)\,[0.8 - (-0.5)]\,(0.85)$$
$$= 12.2\ kN$$

COMMENT

The loads calculated above are the loads that can reasonably be expected to occur under the conditions stated in the example. They should be considered as the minimum suitable loads for use with stresses and load factors appropriate for the type of structural material used.

For geographical areas exhibiting large variations in annual extreme wind speeds, the basic wind speed should be selected with caution. The application of probabilistic models of extreme wind speeds and some of their limitations are discussed in section 1.0.

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