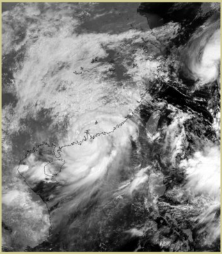
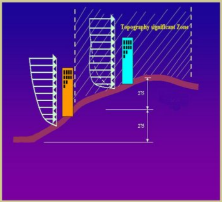
EXPLANATORY MATERIALS TO THE CODE OF PRACTICE ON WIND EFFECTS IN HONG KONG 2004

Code of Practice on Wind Effects in Hong Kong 2004





屋宇署
BUILDINGS DEPARTMENT

Explanatory Materials to the Code of Practice on Wind Effects in Hong Kong 2004

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Foreword

The Explanatory Materials give a summary of the background information and considerations reviewed by the code drafting committee during the preparing of the Code of Practice on Wind Effects in Hong Kong 2004, which will be referred to as 'the Code' in this document.

As the Code aims to retain the essence of a simple format of its predecessor for ease of application, the Explanatory Materials was set out to accomplish the Code by explaining in depth the major changes in the Code and to address on situations where the application of the Code may require special attention.

The Explanatory Materials is a technical publication and should not be taken as a part of the Code.

Acknowledgment

The compilation of the Explanatory Materials to the Code of Practice on Wind Effects Hong Kong 2004 owes a great deal to Dr. K. M. Lam and Ir. K. L. Lo for their contribution of manuscripts, and to the Chairman of the Ad-hoc Committee to review the Code of Practice on Wind Effects, Ir. K. M. Cheung for his advice and guidance in formulating the document.

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Thank is also due to the Hong Kong Observatory for providing the cloud imagery on the cover page which was originally captured with the Geostationary Meteorological Satellite (GMS-5) of Japan Meteorological Agency.

C O N T E N T S

Section 1 The Basic Wind Velocity Profile

Wind Characteristics in Hong Kong

1.1 Wind characteristics near the ground are mainly described by the hourly mean wind velocity profile, peak gust wind velocity profile, turbulence intensity profile, and directional distribution of wind speed. Two dominant factors shape the extreme wind loading in Hong Kong. The first is the exposure to severe typhoons. The second is the protection afforded by one of the most sheltered natural harbours in the world. These two characteristics tend to interact.

1.2 The wind characteristics for Hong Kong environment have been discussed by many researchers in past years including Mackey(13), Ko(14), Chen(18), Choi(16), Davenport et al(19), Melbourne(20), Jeary(25), and Holmes et al.(39) However, due to the difficulties involved in both the understanding of typhoon structure over large hills or mountains and the measurement of wind characteristics during typhoons, the wind characteristics near the ground in Hong Kong associated with building design are still not fully understood.

Reference Wind Speed

1.3 The Hong Kong Observatory was founded in 1883 and has been keeping almost complete records of wind speed from 1884 onwards. These records have enabled estimates of extreme wind speed at the ground surface to be made. The Hong Kong Observatory maintains a large number of measuring stations. Among these stations, the one at Waglan Island is considered to be the principal source of information during the past years. There are two main reasons for this. Firstly, the Waglan data are obtained from measurements on an isolated island exposed to the predominant winds. Secondly, data from other sources within the city areas have been subject, over the years, to a changing environment or to the influence of topographical features.

1.4 Before 1993, the height of the anemometer at Waglan Island was 75 metres. As a result of the erection of a new mast in 1993, the anemometer height at Waglan was adjusted to 82 metres. The steep rocky profile of the island presents a blockage to the wind, and the subsequent speed-up over the island means that the measurements at anemometer height are actually representative of the wind speeds at a greater height over the open water approaching the island. Melbourne suggested that the measurements at anemometer height are representatives of unobstructed measurements at 90 metres. In practice, this correction makes only a small difference to the absolute estimates of wind speeds and an effective reference height of 90 metres is adopted in the Code for derivation of the design wind speeds.

1.5 The data from measurements at Waglan Island were analysed using the Lieblein BLUE (Best Linear Unbiased Estimator) Techniques[22] to establish the probability of occurrence of certain mean and gust wind speeds. All available typhoon data measured at Waglan Island since 1953 form the basis for analysis.

1.6 Based on these analyses and other sources of published information, the Code has adopted reference hourly-mean and 3-second gust wind speeds of 46.9 and 65.2 m/s respectively at a height of 90 m above mean sea level.

1.7 When comparing the adopted hourly-mean wind speed of 46.9m/s and 3-second gust wind speed of 65.2m/s with the measurements at Waglan Island for some severe typhoons occurred in Hong Kong in the past years (see Table 1.1), it can be seen that the adopted values have demonstrate the expected level of confidence for design purposes.

Table 1.1 Measurement of Severe Typhoon Data at Waglan Island

Typhoon	Hourly-mean wind speed	Gust wind speed
Wanda (1962)	41.4 m/s	60.2 m/s
Rose (1971)	39.0 m/s	52.4 m/s
Ellen (1983)	44.2 m/s	62.7 m/s
York (1999)	42.5 m/s	65.0 m/s
Reference Speed	46.9 m/s	65.2 m/s

Hourly Mean Wind Velocity Profile

1.8 The profile of hourly-mean wind velocity against height may be characterised near the ground surface by a logarithmic relationship and the velocity reaches a value that is reasonably constant at the gradient height at which the ground friction influence becomes insignificant. In the Code, a power law profile is used as an arithmetical approximation to cover the whole range of heights and for use in calculating wind loads on buildings.

1.9 The hourly-mean wind velocity $\bar{v}_z$ at height z can be described by the following power law relationship:-

$$\frac{\bar{v}_z}{\bar{v}_g} = \left(\frac{z}{z_g}\right)^{\alpha} \qquad (1.1)$$

where $\bar{v}_g$ = the hourly-mean wind speed at the gradient height z_g

α = the power law exponent.

1.10 The gradient height is the height at which the ground friction influence becomes insignificant. Recent field research by the National Hurricane Centre of the United States has confirmed that the maximum hurricane velocities occur at a height of around 500m above the ocean(37, 39). These data were obtained by dropping many hundreds of GPS drop-sondes into hurricane eyewalls since 1997 and formed the largest data set yet gathered on tropical cyclone wind profiles. The field data supports the appropriateness of the Code adopting a gradient height of 500m over open sea condition and modelling a conventional boundary layer below this level.

1.11 The field data referred to in Clause 1.10 show that power law exponents of 0.10 to 0.11 for equation (1.1) are appropriate over deep open water at the design wind speed range expected in Hong Kong. Since Waglan Island, from where the basic reference data were collected for analysis, is an isolated island exposed to open sea, it is therefore decided to adopt an open sea condition and used a power law exponent of 0.11 for the construction of the velocity profile in the Code.

1.12 In addition to the analysis of Waglan Island data, computer simulations using Monte-Carlo statistical techniques have become a standard tool in the prediction of typhoon strengths and directionality. Conveniently, most of these simulations assume that gradient balance occurs at 500 m. These include the works conducted at the University of Western Ontario and by Dr Peter Vickery at Applied Research Associates. The latter have been the subject of a number of peer-reviewed publications in recent years (33)(34)(35)(36).

1.13 Using equation (1.1) with reference hourly mean wind speed of 46.9m/s at 90m, gradient height of 500m and α value of 0.11, the gradient hourly mean wind speed is calculated to be 56.6m/s. This value is slightly higher than the gradient hourly mean wind speeds predicted using computer simulation techniques, but is considered within the acceptable error range associated with these techniques.

Gust Wind Velocity Profile

1.14 The gust wind velocity profile is obtained by applying a gust factor to the hourly mean wind profile. The gust factor in turn is a function of the turbulence intensity. The relationship between the gust and the hourly mean wind speed can be expressed through the following relationship:-

$$v_z = \bar{v}_z G = \bar{v}_z (1 + g_v I_z) \qquad (1.2)$$

where G = the gust factor;

g_v = the peak factor which reflects the measured relationship between the peak and the hourly mean wind speeds measured using a standard anemometer. The value is normally taken between 3.4 and 3.7;

I_z = the turbulence intensity at height z;

v_z = the gust wind velocity at height z;

$\bar{v}_z$ = the hourly mean wind velocity at height z.

1.15 Based on a review of the analyses by many researchers, the turbulence intensity at the reference height of 90m is taken to be 0.1055.

1.16 The turbulence intensity defines the degree of gustiness of the wind and is related to root mean square (RMS) wind velocity. With the RMS wind velocity taken to have a constant value at different heights, the turbulence intensity also varies with height according to the power law, but with the power exponent equal to $-\alpha$. As a result, the turbulence intensity would vary with height according to the following expression:

$$\frac{Iz}{Ig} = \left(\frac{v_z}{v_g}\right)^{-1} = \left(\frac{z}{z_g}\right)^{-\alpha} \qquad (1.3)$$

1.17 By combining equations (1.1), (1.2) and (1.3), the gust velocity at any height can be calculated as :-

$$v_z = \bar{v}_g \left(\frac{z}{z_g}\right)^{\alpha} \left[1 + g_v I_g \left(\frac{z}{z_g}\right)^{-\alpha}\right] \qquad (1.4)$$

where $\bar{v}_g$ = gradient mean wind speed = 56.6m/s

z_g = gradient height = 500m

I_g = turbulent intensity at gradient height = 0.087

g_v = peak factor = 3.7

α = power exponent = 0.11

1.18 The gust wind speed at gradient height of 500m is thus calculated to be 74.9m/s.

The Design Velocity and Pressure Profiles

1.19 There is still considerable uncertainty about wind speeds and profiles in typhoons. The above simplified approach assumes that the wind profile follows a "normal" power law until the gradient value of hourly mean wind speed or gust speed is achieved. In determining the appropriate design wind speeds for a Code of Practice in typhoon wind climate areas like Hong Kong, it is necessary to obtain an adequate level of reliability. Taking into consideration the uncertainties inherent in the prediction of typhoon wind speeds and to ensure an appropriate level of safety in structural design, the code recommends that the wind speed for design to be increased by 5% above the derived 50 year return wind speed. The use of a higher design wind speed also accounts for the fact that high localized wind pressure coefficients in excess of code values are often detected during wind tunnel tests.

1.20 Using equation (1.1), the design hourly mean wind speed $\bar{v}_z$ at height z is thus expressed as:-

$$\bar{v}_z = 1.05\bar{v}_g \left(\frac{z}{z_g}\right)^{\alpha} \qquad (1.5)$$

where $\bar{v}_g$ = hourly mean wind speed at gradient height = 56.6m/s

z_g = gradient height = 500m

α = power exponent for mean wind = 0.11

The variation of design hourly mean wind speed with height is calculated by equation (1.5) and the results are tabulated in Table F3 of the Code.

1.21 The design 3 second gust wind speed v_z at height z is calculated by combining equations (1.2), (1.3) and (1.5):-

$$v_z = \bar{v}_z(1 + 3.7I_z)$$

$$= 1.05\bar{v}_g\left(\frac{z}{z_g}\right)^{\alpha}\left[1 + 3.7Ig\left(\frac{z}{zg}\right)^{-\alpha}\right]$$

where $\bar{v}_g$ = hourly mean wind speed at gradient height = 56.6m/s

I_g = turbulence intensity at gradient height = 0.087

z_g = gradient height = 500m

α = power exponent for mean wind = 0.11

1.22 The design hourly-mean wind speed and 3-second gust wind speed at reference height and gradient height respectively are summarised as follows :-

Design hourly-mean wind speed at reference height of 90m	= 49.2m/s
Design hourly-mean wind speed at gradient height of 500m	= 59.5m/s
Design 3-sec gust wind speed at reference height of 90m	= 68.5m/s
Design 3-sec gust wind speed at gradient height of 500m	= 78.7m/s

1.23 The design wind pressure q_z at height z is calculated as :-

$$q_z = \frac{1}{2}\rho v_z^{2}$$

where ρ = density of air = 1.2kg/m^3

The variation of design gust and hourly-mean wind pressure with height are given in Table 1 and Table 2 in the Code.

Section 2 Terrain and Topographic Effect

Terrain Categorization

2.1 Terrain category is defined by the characteristics of the surface roughness of the ground for the wind flow. Normally ground roughness can be divided into 4 to 5 categories, ranging from smooth and open land or sea to built-up city with tall buildings. 3 categories of terrain are considered in the new UK Code BS6399: Part 2 i.e. the sea, the country and the town. Similarly, there are 3 terrain categories defined in the Canadian Code and 4 in the Australia/New Zealand Code and 4 in the current US ASCE-7 Code (although it is intended to be reduced to 2 in future editions).

2.2 However, research works indicate that a transition zone always exists at a change of terrain. The transition of the flow from one roughness to another normally takes a distance of several kilometres for the height ranges affecting most structures. Gust speeds change much more slowly than mean speeds within this transition. Hence the reduction in wind speed associated with the transition to rougher terrain should only be assumed if terrain of the stated roughness exists for this distance, or if suitable transition formulas are adopted.

2.3 As Hong Kong is a city so close to the sea, most of the built-up areas are within a transition zone that is influenced by complex topographic features and variable ground roughness. The relationship between the development height and the fetch distance is complicated. In view of the unique topography and geographical size of Hong Kong, a single terrain (i.e. open sea condition) is considered satisfactory.

Topographic Effect

2.4 Topography, or large vertical displacements of the ground surface, can have significant effect on the mean wind speed profile. In general, wind increases its speed when it moves up the windward slope of a hill or a ridge. The maximum increase in wind speed is usually experienced at or near the summit. When wind passes down a steep leeward slope, there may be separation of flow and the mean wind speed may experience a shelter effect. A valley or a pass has the effect of channelling the wind to flow parallel to its axis and may thus lead to very high wind speeds.

2.5 Most wind codes only take into account the most critical situations of wind speed increase that occur near the summits of hills, ridges or escarpments. Escarpments and ridges are mainly two dimensional land features. Hills differ from ridges in that the wind can diverge over the sides in addition to speeding up over the summits. The degree of speed-up effect for a hill is thus generally less than that for a ridge of the same slope.

Most wind codes adopt a speed-up ratio to account for the increase in wind speed. Where wind is accelerated on a topographic feature, the original wind speed v_z at a height z of the approaching wind increases to v'_z at the same value of height z above the surface of the slope. The speed-up ratio is defined as:

$$\beta = \beta(x,z) = \frac{v'_z - v_z}{v_z}$$

The ratio varies with height z as well as the along-wind location x relative to the summit of the topographic feature. The speed-up effect is mainly on the mean wind speed while the standard deviation of the wind speed remains essentially the same. As a result, the increase in the gust wind speed is not as significant as the increase in the mean wind speed. Ideally, there should be separate speed-up ratios for the mean and the gust wind speeds. (e.g., BS6399 – Part 2, Directional method).

Topography Factor

2.6 In the Code, the guidelines on topographic effect are adopted from the Standard method of BS6399 – Part 2. In BS6399-Part 2, the speed-up effect incorporates an altitude factor, which is one of the many factors to be multiplied to the basic wind speed to arrive at the site wind speed. This altitude factor however is not included in the Code due to the geographical nature of the Hong Kong .

2.7 Unlike the topography speed-up ratio of the BS6399 – Standard Method which is to be applied to the gust wind speed. In this Code, the topography factor S_a is applied to the design wind pressure that is the product of half the air density and the square of the gust wind speed. This is the reason of having a square in the equation for S_a, i.e. $S_a = \beta^2$.

2.8 To assess the effect of topography, the topographic feature is first approximated to an idealized ridge, escarpment or hill of uniform upwind and downwind slopes. In determining the slope, it may be more representative to use the top half of the hill, ridge or escarpment as reference because the speed-up occurs primarily on the top half and the wind speed is mostly affected near the summit. After drawing up the slope, the region of speed-up due to the topography effect can be found with the aid of Fig. C1 in the Code. The values of the various geometrical descriptions of the topographic feature are found with the aid of Fig. C2.

2.8 The speed-up ratio is a function of height, z and along-wind location, x. This is reflected in the topography location factor s, the value of which at different values of x and z is shown by the contours in Fig. C3 and Fig. C4 in the Code. The speed-up ratio is determined by the equation:

$$\beta = \sqrt{S_a} = 1 + 1.2\alpha_e s$$

Interpolation of values of s at intermediate values of x and z is allowable.

2.9 It should be noted that where the topography effect is severe complex the approach given in the Code may underestimate the speed-up [40] and hence not be applicable. In this case, specialist advice is required.

Section 3 Dynamic Response of Structures

Signpost to Dynamic Sensitivity

3.1 For the assessment of resonant dynamic response effect of a structure, a signpost is to be provided to first determine whether the resonant dynamic response is significant or not. In the case that it is, then several new elements of assessment are required for the purpose of estimation of dynamic response effect. These include the determination of turbulence intensity, damping ratio, natural frequency and some other descriptors of wind energy parameters. In the case that the structure is not with significant resonant dynamic response, then a quasi-static approach may be adopted.

3.2 Several codes of practice provide signposts to define whether a quasi-static approach is sufficient or a fully dynamic analysis is necessary for determining wind force acting on a structure. (Table 3.1 refers)

Table 3.1 Various Signposts for Dynamically Sensitive Structures

Authorised Code	Definitions of dynamic sensitive structure
Australia/New Zealand Standard AS/NZS 1170.2-1989	Height exceeds 5 times the least plan dimension, and the natural frequency in the first mode of vibration is less than 1.0 Hz.
ASCE Standard ASCE 7-02	Height exceeding 5 times the least horizontal dimension or a fundamental natural frequency less than 1.0 Hz.
National Building Code of Canada 1995	Height is greater than 4 times the minimum effective width or greater than 120m.

3.3 The first governing condition relates to an aspect ratio of the structure and the least horizontal dimension is intended to account for stepped or tapered building profiles. This condition would exclude short squat buildings from the requirement for a dynamic analysis.

3.4 The second condition relates to the fundamental natural frequency of the structure, or indirectly to the height of the structure. In general, if the fundamental natural frequency is less than 1 Hz, the building has to be designed for the effect of resonant dynamic response.

3.5 Jeary and Yip[11] carried out a study on three different signposts in 1994. The signposts were taken from the proposed ISO Code for wind loading (Davenport 1989), the Australian Wind Loading Code (AS 1170.2 - 1989) and the BRE digest series 346 which was issued for a basis of the new Eurocodes. The ISO and BRE versions both have formulae to evaluate the signposts, whilst the Australian Code has a simple requirement.

3.6 Seven buildings for which dynamic data were available were used to study the effect of the different signposts. These included the Jardine House (179m high), Bank of China (305m high), Hong Kong Bank (179m high), Peoples College, Nottingham (4.8m high) and three harmony blocks from Housing Department ranging from 82m to 118m high. These buildings were chosen to represent a set includes clearly dynamically sensitive (Jardine Houses, Bank of China and Hong Kong Bank Building) to clearly quasi-static (People's College) with the three harmony blocks close to the threshold.

3.7 The results from the three sets of signposts are in broad agreement with the three categories of buildings in the set assumed above. More details of the evaluation and outcome can be found in the research report by Jeary and Yip (1994)[11].

3.8 The 1989 Australian approach is chosen as the basis for formulating the signpost for Hong Kong. The Australian Code required the aspect ratio to be less than 5 and the natural frequency greater than 1.0 Hz if the structure is not to be classified as with significant resonant dynamic response. Use of the standard Ellis formula for evaluation of the fundamental natural frequency i.e. natural frequency = 46/height of structure in metres, would imply that any building with a height greater than 46 metres would be classified as being dynamically significant. Although many standard forms of construction in Hong Kong are particularly stiff and study by Jeary and Yip suggested a 100m restriction is reasonable for Hong Kong typical buildings, it is noted that slender buildings with a height of less than 100 m may also be dynamically significant. In the Code, a building is therefore considered to be one with significant resonant dynamical response if it has either one of the following properties unless it can be justified that the fundamental natural frequency of the building is greater than 1 Hz:-

(a) The height exceeds five times the least horizontal dimension.

(b) The height of the building is greater than 100m.

Along-wind Response

3.9 The wind-induced dynamic force on a tall structure may be resolved into two components: along-wind dynamic force parallel and cross-wind dynamic force normal to the direction of incident mean wind velocity. The response of the structure to the along-wind dynamic force is called the along-wind dynamic response, and correspondingly the response of the building caused by the cross-wind dynamic force is regarded as the cross-wind response. Torsional dynamic response of a tall structure may also occur especially when the along-wind and/or cross-wind dynamic forces and/or the centre of mass do not coincide with the elastic centre of the structure.

3.10 Most modern wind loading codes in the world provide the gust factor method for estimating the along-wind dynamic response. The gust factor approach was derived from the early work of Davenport in the 1960's. It is recognised as a satisfactory assessment method for the along-wind response where the design peak base overturning moment is determined by multiplying the mean base overturning moment by the gust factor. For ease of application, the Code recommends that the gust factor be defined as a dynamic magnification factor that represents the amount by which the mean wind forces shall be multiplied to account for the resonant dynamic behaviour.

3.11 The basic mechanism of along-wind response of a slender structure is turbulence buffeting. Wind flow is turbulent and the gustiness in the wind produces fluctuating forces on the structure. The fluctuating along-wind loading acting on a structure is primarily a function of turbulence intensity and turbulence scale. The turbulence intensity determines the local magnitude of fluctuating loading while the turbulence scale, in relation to the size of the structure, determines how well the fluctuations are correlated over the structure. The dynamic response may be calculated as a sum of quasi-static response for low frequency component and resonance response at the first natural frequency. The following gust factor equation given in Appendix F of the Code is the simplified one.

$$G = 1 + 2I_h \sqrt{g_v^2 B + \frac{g_f^2 SE}{\xi}} \qquad (3.1)$$

3.12 The turbulence intensity, I_h, at the roof top of the structure, can be assessed by using the power law expression as discussed in Section 1. The two functions underneath the square root sign in the equation represent the quasi-static response (or background response) and resonant response of the structure respectively.

3.13 The peak factor, g, is a measure of the degree of randomness of the fluctuating component. The peak factors for the background response and resonant response are identified separately as g_v and g_f in the equation. The peak factor for background response g_v is taken as 3.7 while the peak factor for resonant response g_f is a function of the first natural frequency, n_a , $g_f = \sqrt{2\log_e(3600n_a)}$. The natural frequency of the structure can be estimated from the height of the structure, h by using the empirical expression of Ellis (1980): n_a = 46/h. In critical cases , the natural frequency should be obtained from a modal dynamic analysis.

3.14 The background factor B is used to measure the background component of the fluctuating response caused by the lower frequency wind speed variation and is primarily a function of the dimension of the structure.

$$\mathrm{B} = \frac{1}{1+\dfrac{\sqrt{36h^2+64b^2}}{L_h}}$$

Where h = the height of the structure in metres

b = the width of the structure in metres

L_h = the effective turbulence length scale in metres and expressed as :

$$L_h = 1000\left(\frac{h}{10}\right)^{0.25}$$

3.15 The parameter S is a size factor to consider the correlation of pressures over a structure and it is related to the first natural frequency n_a, the design hourly mean wind speed $\overline{V}_h$ at structure height h and the dimensions of the structure. The design hourly mean wind speed for different heights can be determined by using equation (1.5) in Section 1 and the values are given in Table F3 of the Code.

$$\mathrm{S} = \frac{1}{\left(1+3.5n_a\dfrac{h}{\overline{V}_h}\right)\left(1+\dfrac{4n_ab}{\overline{V}_h}\right)}$$

3.16 The parameter, E is a spectrum of turbulence in the approaching wind stream and it is given by

$$E = \frac{0.47N}{\left(2 + N^2\right)^{5/6}}$$

where N is an effective reduced frequency and is equal to

$$N = n_a \frac{L_h}{\overline{V}_h}$$

3.17 The damping ratio, ζ, reflects the damping capacity of the structure and is defined as a fraction of the critical damping. In general, the damping ratio includes both structural damping and aerodynamic damping. The Code recommends 1.5% of critical damping for steel structures and 2% of critical damping for concrete structures, which are generally accepted as reasonable figures for design purposes at design load levels. For very squat or very slender structures, the value of structural damping may be lower or higher respectively. As structural damping is amplitude dependent, it is common to use lower values if assessing dynamic response (e.g. accelerations) at lower return periods.

3.18 With the assumed fundamental natural frequency of 46/h and critical damping values of 1.5% and 2% as suggested in Clause 3.17, the dynamic response multiplication factor, G, can be determined from the height, h, and breath, b, of the structure. Variation of G values with height (h), and breath (b) of a structure for critical damping of 1.5% and 2% are given in Tables F1 and F2 in the Code for designers' easy reference and use. When more refined estimates of the natural frequency and critical damping value appropriate to the structure are available, designers should use the basic equation (3.1) to derive the value of G for the structure.

Cross-wind and Torsional Responses

3.19 Cross-wind vibration of structures is caused by the combined effects from buffeting, vortex shedding and galloping. Due to the complex interaction of these forces, there is no precise analytical method available to calculate cross-wind response of tall structures. Saunders and Melbourne (1975) and Kwok (1982) carried out extensive aero-elastic tests of tall buildings of various sizes in wind tunnels and proposed a spectral method to estimate cross-wind response of tall buildings. This method is based on the generalized first mode cross-wind force spectrum measured from wind tunnel tests as well as the random vibration theory. This method has been adopted in the Australian Wind Loading Code (1989). In the Australia/New Zealand Standard (2002), values of the cross-wind force spectrum coefficient for both square and rectangular section buildings are provided in the form of curves which are functions of turbulence intensity and the Strouhal Number, and they are obtained from wind tunnel tests of isolated buildings under typical wind conditions. Similar data for typical Hong Kong buildings shapes in the wind regime described in the Code are not available at this time. However, it should be noted that the Australia/New Zealand Standard indicates that, for slender exposed buildings, the cross-wind loads can greatly exceed the along-wind loads. For such buildings, it is recommended that specialist advice should be sought.

3.20 The torsional dynamic response of a tall building may be especially significant when the along-wind and/or cross-wind dynamic forces or the centre of mass do not coincide with the elastic centre of the building. This may occur, for example, as a result of building shape, structural eccentricity and/or uneven load patterns resulting from the surroundings. Furthermore, when the mass centre of the building does not coincide with the elastic centre, coupled translational-torsional vibration may occur. The code-based procedure for evaluating the torsional response and the coupled translational-torsional response of tall buildings is still under development. Boundary layer wind tunnel tests or specialist advice may have to be sought to tackle these problems for buildings of unusual shapes and buildings with complex surroundings.

3.21 The Code does not provide any guidance for assessment of the cross-wind and torsional responses of tall structures, but designers are reminded by Clause 7.3 in the Code that in the case of a structure for which the cross wind response and/or torsional response may be significant, the dynamic effects should be investigated in accordance with the recommendations given in published literature and/or through the use of dynamic wind tunnel model studies. The total response of a structure may normally be taken as a combination of the responses in the three fundamental modes of vibration.

Section 4 Force Coefficients and Pressure Coefficients

Force Coefficients

4.1 The Code adopts the force coefficient method in the determination of total force on a building due to wind effects. The static wind force acting on a building is expressed as the product of the design wind pressure and the force coefficient, which is in turn a function of the building shape and the height aspect ratio. A similar approach was adopted in the Code of Practice on Wind Effects Hong Kong -1983. The total wind force on a building, F, is thus expressed as :-

$$F = C_f \sum q_z A_z \qquad (4.1)$$

Where C_f is the force coefficient for the building, which is a product of the height aspect factor, C_h and the shape factor C_s given in Appendix D of the Code;

q_z is the design wind pressure at height z;

A_z is the effective projected area of the building

4.2 In the absence of accurate data on irregular shapes, and for the convenience of application, the Code adopts only a few fundamental shapes i.e. square, rectangular and circular, and recommends using coefficients for the respective enclosing rectangles for all other shapes. Nevertheless, allowance is made in Clause D1.1(b) of the Code for designers to use appropriate values specified in other international codes.

4.3 Contiguous buildings may be regarded as one single building in aerodynamic terms although they may be categorically structurally independent from each other. It is therefore recommended in Clause D1.2(b) of the Code that contiguous buildings may be considered as a whole building block when considering the wind loading effect. The shape factor and the height aspect factor for this type of contiguous building structure shall embrace the overall enclosed building. For dynamically sensitive buildings, care should be taken to ensure that the differential wind-induced loads and motions between contiguous buildings are adequately catered for.

4.4 The reduction factors for buildings with large frontal area and force coefficients for open framework buildings remain the same as those given in the Code of Practice on Wind Effects Hong Kong - 1983.

Pressure Coefficients

4.5 The total force on a building element is the sum of the forces acting on the external and internal faces of the element. Internal and external pressure coefficients should be chosen to give the most critical positive and negative (suction) forces on the element. The critical combinations of these coefficients for normal rectangular buildings have been calculated in the Code and they are given as the resultant pressure coefficients in Table E1. The value of internal pressure coefficient assumed in deriving the resultant pressure coefficients is +0.2 or -0.3. The positive pressure coefficients act towards a building surface (or downwards in the case of canopies) and negative pressure coefficients act away from a building surface (or upwards in the case of canopies). Therefore, it is necessary to combine the negative external pressure coefficient with the positive internal pressure coefficient to cater for the worst net negative pressure coefficient, or reverse the above for calculating the worst net positive pressure coefficient.

4.6 Table E1 of the Code gives generalised pressure coefficients for elements such as roofs, cladding and wall panels. Pressure coefficients for design of canopies are also introduced based on the assessment given in the Australia/New Zealand Standard. The wind loading on the building element, F_p, is equal to the product of the pressure coefficient at the location, the projected area of the building element and the basic design wind pressure.

$$F_p = C_p q_z A_m \qquad (4.2)$$

Where C_p = the pressure coefficient for individual elements;

q_z = the design wind pressure corresponding to the height z of the element;

A_m = the surface area of the element.

Wind Pressure near Ground Surface

4.7 Wind tunnel data from building studies indicate that high pressures and suctions are experienced near the ground surface for tall buildings. These are resulted from the way the wind is channelled down the façade of a tall building and subsequently accelerated to flow around building corners. This effect has an impact on the design of canopies, claddings and wall panel elements. In the Code, the design wind pressure, q_z for design of roofs, canopies, wall and cladding panels is adjusted to allow for the effect of larger pressures and suctions occurred at lower level of a building. Clause 6.2 of the Code specifies a minimum value for q_z over the lower part of the building. The design wind pressure, q_z, shall be taken as constant over a height which is equal to the breadth of the building or the actual height of the building whichever is the lesser.

Section 5 Wind Tunnel Test

General

5.1 Wind tunnel testing is a physical modelling of the situation of wind flow around a building in a reduced scale 1:λ. The characteristics of natural wind flow at a building site is simulated in a boundary layer wind tunnel and wind effects on the building are measured with appropriate equipment and techniques. Examples of wind effects that can be investigated include: wind pressures on the building façade, wind loads on the building, wind-induced dynamic responses and the associated dynamic wind loads, and environmental wind conditions at the pedestrian level around the base of the building. The wind effect data obtained from the wind tunnel are in the model scale (subscript $_m$) and it is usually required to combine the data with a climate model to arrive at full-scale (prototype) wind effect data (subscript $_p$).

5.2 In many countries, there are guidelines and quality assurance for wind tunnel testing, especially for the determination of wind loads. A summary can be found in Lam and Tam (1996)[(28)]. At a number of places in this Explanatory Material, reference is made to the Australian Wind Engineering Society's Quality Assurance Manual (AWES 2001)[(29)], hereafter referred to as AWES-QAM-1-2001. Another good source of information about wind tunnel testing is the ASCE Manual of Practice on Wind Tunnel Studies of Buildings and Structures[(30)]. The information contained in the following sections should therefore be considered as a basic introduction to the factors that affect wind tunnel testing of typical Hong Kong building structures.

Static Structures

5.3 The natural wind in the Atmospheric Boundary Layer (ABL) is most conveniently modelled by the turbulent airflow over a rough surface in a wind tunnel. Although the two forms of flow are not the same, it has been shown that the lower part of the wind tunnel turbulent boundary layer presents an exact analogy of the lower part of the ABL. Wind simulation in a typical boundary layer wind tunnel is achieved by the use of a long fetch of roughness elements on the wind tunnel floor. A shorter fetch of roughness elements may also be used when other means of initial boundary layer generation, such as tall spires and/or trip boards are employed.

5.4 One important characteristic of wind is how the mean wind speed varies with height at a particular ground terrain type. The roughness of the ground retards the wind in the ABL. The wind speed is zero at the ground level. It increases with height above ground until gradient height, above which the wind speed is assumed to remain roughly constant with height. In tropical cyclones, this is a simplification as wind speed can actually decrease at great heights (>500-600 m), while other extreme wind events such as thunderstorms, downbursts and tornadoes have quite different structures.

5.5 The variation of hourly-mean wind speed with height, or the hourly mean wind speed profile, is specified in the Code. It is described by using the power law as:

$$\frac{V}{V_g} = \left(\frac{z}{z_g}\right)^{\alpha}$$

A similar profile is required in the wind tunnel modelling. The data of hourly mean wind speed at different heights in the wind tunnel, when plotted in this non-dimensional form, should fall closely onto the target profile with a power exponent α. AWES-QAM-1-2001 recommends that the simulation is acceptable if the wind tunnel speed data fall within 10% of the target profile.

5.6 Another important feature of wind is its turbulence or gustiness. This plays an important role in generating peak pressures on a structure and inducing vibrations on a flexible structure. On the first level, wind turbulence can be measured by the turbulence intensity. It is the ratio of the standard deviation value of wind speed fluctuations to the mean wind speed. The turbulence intensity normally decreases with increasing height. The turbulence intensity profile is specified in the Code and a power law is used to describe the profile:

$$I_u = \frac{\sigma_u}{V}$$

$$\frac{I_u}{I_{u,ref}} = \left(\frac{z}{z_{ref}}\right)^{\beta}$$

5.7 Like the hourly mean wind speed profile, the turbulence intensity profile in the wind tunnel to that in the ABL is checked by plotting the measured values in the wind tunnel alongside the target profiles with the height normalised by the reference height. The same accuracy within 10% is recommended by AWES-QAM-1-2001 for this profile.

The allowable variations in mean wind speed and turbulence intensity can result in significant deviation from the intended peak gust pressures (of plus or minus about 30%). There is thus a requirement in the Code to calibrate the wind profiles to reproduce the gust pressures as given in the Code.

5.8 The size of dominant gust eddies in the turbulent wind are important in producing peak loads on an area of a building. This is measured by the integral scales of turbulence and the turbulence spectra. The usual practice in wind tunnel modelling is to measure the along-wind spectrum $S_{uu}(n)$ at an appropriate height and then match it to the universal wind turbulence spectrum. The universal spectrum is on a non-dimensional frequency axis of $nL_{u,x}/V$ and the value of the longitudinal scale of along-wind turbulence, $L_{u,x}$, is obtained from the best match. Ideally, this value should match with the full-scale ABL value, that is $(L_{u,x})_p/(L_{u,x})_m = \lambda$. In practice, it is seldom possible to obtain a perfect match and a mismatch of not more than 2 and 3 is considered acceptable respectively for modelling of overall wind loads and for cladding pressures (AWES-QAM-1-2001).

5.9 Wind speed, wind pressures and wind loads fluctuate with time due to wind gustiness. It is commonly accepted that fluctuations with a period shorter than one second produce negligible wind effects on a building structure. The various measuring equipment and techniques used in the wind tunnel are thus required to be able to measure quantities at fluctuations faster than this one-second criterion. In the wind tunnel, time and frequency are in the model scale as determined by the length scale and the velocity scale:

$$\lambda_t = \frac{1}{\lambda_n} = \frac{t_p}{t_m} = \frac{L_p}{L_m}\frac{V_m}{V_p} = \frac{\lambda}{\lambda_V}$$

where subscript *t* indicates time, subscript *n* indicates frequency, and subscript *v* indicates velocity.

5.10 For example, if a velocity scale 1:5 is used in a wind tunnel test and the length scale is 1:250, the model time is 1/50 the full-scale time. Fluctuations faster than 1/50 second need to be measured. In order to obtain adequate resolution to detect peaks at this frequency, the response of the instrumentation need to be good up to four times 50 Hz, that is *200* Hz.

5.11 During a wind tunnel test, a pitot-static tube is normally placed in the test section to monitor the wind speed and to provide the reference static pressure in the wind tunnel. The pitot-static tube should be located outside the zone in which the wind field is modified by the presence of the wind tunnel model. The pitot-static tube is suitable only for measurements of mean wind speed in relatively low turbulence conditions. For measurement of gust wind speeds and turbulence intensities, there are a number of other suitable instruments such as hot-wire and hot-film anemometers and miniature pressure probes.

5.12 The pressure at a point on the surface of the test building structure is usually measured by means of a pressure tap. Pressure at the tap location is commuted through a length of flexible tubing to a pressure transducer which converts the pressure signal into an electric signal. Wind pressures can vary greatly over the building surface and pressure taps need to be placed at a sufficient density in order to accurately measure the pressure distribution. AWES-QAM-1-2001 recommends that the average pressure tap density should be higher than one tap per 120 square metres of surface area on the test building.

5.13 The frequency response of pressure measurement is normally limited by that of the pressure tubing. The frequency response and the associated distortion to the pressure signal are greater for longer and larger diameter tubing. There are standard methods to correct for this tubing response problem including the use of restrictors, leaked tube systems or mathematical transform corrections.

5.14 Wind forces and moments on a building model are sometimes measured directly with force balances. Where dynamic loads are to be measured (either background and/or resonant components), the model-balance assembly must be stiff enough to have a combined natural frequency lying above and away from the frequency range of the wind loads to be measured.

5.15 Model values of wind pressures and wind load obtained in the wind tunnel can be converted to appropriate full-scale values through the use of loading coefficients. For example, wind forces are converted through an equation such as:

$$F_p = C_F \tfrac{1}{2}\rho V^2 A = F_m \lambda^2 \left(\frac{V_p}{V_m}\right)^2$$

The choice of the prototype wind speed depends on the full-scale event to be simulated. To estimate the design wind loads of a normal building, the design wind speed should be used as the prototype wind speed. The determination of wind forces and wind moments by this conversion is only valid for static buildings that do not exhibit resonant dynamic response to wind actions.

5.16 Wind effects depend on the incident wind directions. It is thus required to make wind tunnel measurements at a number of wind directions in order to obtain the most critical wind loading cases. In general, a minimum of 24 wind directions at 15^o intervals is necessary, with smaller intervals around the wind directions causing critical loads. Many wind tunnel laboratories now routinely test at 36 wind directions for pressures and loads.

Dynamic Structures

5.17 Many structures, such as tall buildings, towers, cable-suspended bridges and cable-suspended roofs are sensitive to wind-induced vibration that depends to a large extent on the characteristics of the structures. The most relevant dynamic properties of a structure are mode shapes of vibration, natural frequencies (which depend on the mass distributions and stiffness), modal masses, and damping. They define the mechanical admittance function, which describes how fluctuating deflections are produced from fluctuating forces.

5.18 In modelling the responses of a dynamic structure, the dynamic characteristics of the structure have to be modelled in addition to the wind characteristics and the aerodynamic shape characteristics. In structural dynamics, wind-induced fluctuating responses are usually analysed with the spectrum method in which the response spectrum is obtained from the product of the aerodynamic force spectrum and the mechanical admittance function of the structure. Therefore, similarity of responses requires both similarity of the aerodynamic force spectrum and similarity of the mechanical admittance function.

5.19 A satisfactory simulation of the wind characteristics for static structures discussed above will result in the similarity of the force spectrum. The equality of aerodynamic shape factors in the prototype and the model is provided by having the same external shapes for the structure and the model. The similarity of the mechanical admittance function can be achieved by introducing the dynamic characteristics of the building either physically in the wind tunnel model or analytically during the analysis of wind tunnel data.

5.20 Aero-elastic tests physically simulate the dynamic characteristics of the prototype building. Full similarity can only be met by a full aero-elastic model that reproduces in the appropriate scale the mass distribution, stiffness distribution and structural damping at every part of the prototype building. For a tall building, a rigid model on an elastic base can be an alternative to a full aero-elastic model. Most energy of the wind-induced vibrations of a tall building (with the exception of those where torsional response is significant) is contributed by the orthogonal fundamental translational modes whose modal deflection follows very closely a linear shape. It is thus possible to estimate the dynamic behaviour with a “stick model” where the dynamic properties of the building are lumped together at the base. A stick model is a rigid model of the building mounted elastically on a pivot at the base so that it can vibrate in a linear mode shape. If significant torsional response is likely, then it is necessary to use a three degree-of-freedom aero-elastic test rig.

5.21 The high-frequency force-balance (or base balance) technique is the most commonly used method of determining wind-induced loads and responses of tall buildings. The technique is based on the measurement of base moments, hence generalized forces, on a rigid model of the building and the computation of wind-induced response using the random vibration theory. Mean and fluctuating wind moments on the whole building are obtained from direct measurements on a lightweight rigid model of the building in the wind tunnel with a sensitive force balance. The base moments approximate the generalized wind forces on the building of the fundamental vibration mode. This provides the aerodynamic force spectrum. The dynamic properties of the building are obtained from a dynamic analysis of the building structure. Together with an assumed value of damping, the mechanical admittance function of the building is obtained. Following the random vibration theory, the measured generalised wind forces and the mechanical admittance function are combined to obtain the base moment of the prototype building. These base moments are typically then distributed to provide floor-by-floor loads for structural design.

5.22 The 'high-frequency pressure integration' is the third method sometimes used to obtain structural loads. Like the high-frequency force balance technique, this employs a rigid model and requires the mathematical incorporation of the mechanical admittance function. This technique is most commonly employed when testing long-span roofs but has recently been used on simple-shaped tall buildings. In Hong Kong there are practical limitations to this technique on many buildings where the surface area is high in relation to the internal volume (e.g. typical housing blocks) as it is not possible to fit sufficient pressure tubes into the model.

5.23 With all the above techniques, appropriate load cases combining responses in different modes and from different excitation mechanisms should be provided.

Topography and Proximity Modelling

5.22 The simulation methods described above for static structures produce the correct general wind characteristics over a uniform terrain. If the site is near to or located on a local topographic feature, the wind characteristics, in particular the mean wind speed profile, may be significantly modified by that feature. The same applies if the test building is surrounded by sizeable neighbouring buildings.

5.23 In these situations, it is necessary to include a detailed representation of the surrounding topographic features and/or neighbouring buildings in a region of some distance around the site. This "proximity model" includes a reproduction of the neighbouring buildings in the correct scale and may also include small local topographic features. If too small an area is represented, the simulation may not be able to include all the possible effects on the wind characteristics from the structures in the proximity. If too large an area is included, the linear scale of the model has to be reduced, given the dimensions of the wind tunnel. AWES-QAM-1-2001 recommends that "in general, all major buildings and topographical features within a radius of 500 metres of the building site should be modelled to the correct scale, to an accuracy of 10% or better".

5.24 In situations where wind characteristics at the site are affected by large-scale topographic features, it may be necessary to model and study the effects of these features separately. The topography model is built at a much smaller scale to cover a large area so that the large-scale topographic features can be included. The wind profiles at the location of the site are measured in the topography model. Afterwards, these wind profiles are reproduced in the test section of the wind tunnel at the normal geometric length scale and a model of the building, with the proximity model of neighbouring buildings if required, is tested under these wind conditions. It is normal to remove buildings that will be included in the proximity model from the topography model.

Model Scale Limitations

5.25 Wind flow patterns depend on the Reynolds number (Re) but it is impractical to achieve the same Reynolds number in the wind tunnel simulation as the full-scale prototype wind flow. Fortunately, the effect of the Reynolds number on the flow is significant only at low Reynolds numbers. When the flow is full turbulent, at sufficiently high Reynolds numbers, the flow patterns become almost independent of the Reynolds number. The Reynolds number effect becomes insignificant and a mismatch of Reynolds number between the model and the prototype is acceptable. For building shapes with sharp corners, AWES-QAM-1-2001 recommends the minimum value of $Re = 5 \times 10^4$ based on the smaller building width and the mean wind speed at roof height, while the ASCE Manual of Practice on Wind Tunnel Testing recommends a minimum value of $Re = 1 \times 10^4$. For building shapes of smooth profiles or with rounded corners, separation of wind flow on the building surfaces depends on Reynolds' number and turbulence to a much higher degree than on sharp edged surfaces where the separation is fixed by the sharp edge. In such cases, the Re mismatch in the wind tunnel must be addressed and dealt with. One simple and efficient way is to add roughness to the surfaces of the building model to trigger turbulent flow separation. Care should be taken to ensure that an appropriate degree of roughness is used as this can also cause inappropriate separation behaviour. In some cases, special larger scale studies will be necessary to calibrate this.

5.26 To satisfy the minimum Re requirement, the geometric length scale and velocity ratio used in the wind tunnel simulation cannot be too small. A general criterion for the velocity ratio is that it should be greater than 1:10. For the length scale of the building model, a scale larger than 1:500 is desirable. On the accuracy of the building model, AWES-QAM-1-2001 recommends that the overall dimensions of the test building model should be accurate to within 2% and architectural details should be included when their smallest dimension is 1 metre or greater.

5.27 On the other hand, the wind tunnel models should not be too large otherwise the flow will be distorted by blockage. Wind tunnel blockage is measured by the blockage ratio which is the ratio of frontal area of the building models (and proximity model) to the cross-sectional area of the wind tunnel test section. The blockage ratio should be kept to below 10%. At higher blockage, the test results may be adjusted by appropriate blockage corrections, however these are not simple to calculate in turbulent boundary layer conditions. The requirement for a maximum blockage ratio can be relaxed if blockage tolerant test section is employed.

Design Wind Pressure

5.28 In Clause 5.5 above, it is mentioned that a 10% uncertainty is allowed on the mean wind speed profile and on the turbulence intensity profile. In the conversion of measured model values of wind pressures and wind loads to full-scale values, the ratio of prototype velocity to model velocity is important. It is thus desirable to specify a reference height at which the velocity value, or the dynamic pressure value, is taken for the model-to-prototype conversion. In the Code, the velocity values are obtained from extreme wind analysis in Hong Kong and most of the wind data were obtained at a height of 90 m. Hence, the reference height for the purpose of this section is specified to be 90 m, or 2/3 of the building height, whichever is greater. The inclusion of the latter height is to account for very tall buildings where the wind pressures at upper levels play a major part in the overall wind loads. The use of the gust wind speed, or gust wind pressure, in the calibration is in line with the Code which tabulates the gust wind pressures at different heights. This should be regarded as a general guideline, although there are special cases where the use of other matching heights may be more appropriate, especially where topography is significant.

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