Wind Loading on Tall Buildings

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ABSTRACT: Simple quasi-static treatment of wind loading, which is universally applied to design of typical low to medium-rise structures, can be unacceptably conservative for design of very tall buildings. On the other hand such simple treatment can easily lead to erroneous results and under-estimations. More importantly such a simplified treatment for deriving lateral loads does not address key design issues including dynamic response (effects of resonance, acceleration, damping, structural stiffness), interference from other structures, wind directionality, and cross wind response, which are all important factors in wind design of tall buildings. This paper provides an outline of advanced levels of wind design, in the context of the Australian Wind Code, and illustrates the exceptional benefits it offers over simplified approaches. Wind tunnel testing, which has the potential benefits of further refinement in deriving design wind loading and its effects on tall buildings, is also emphasized.

1 INTRODUCTION

Wind is a phenomenon of great complexity because of the many flow situations arising from the interaction of wind with structures. Wind is composed of a multitude of eddies of varying sizes and rotational characteristics carried along in a general stream of air moving relative to the earth's surface. These eddies give wind its gusty or turbulent character. The gustiness of strong winds in the lower levels of the atmosphere largely arises from interaction with surface features. The average wind speed over a time period of the order of ten minutes or more, tends to increase with height, while the gustiness tends to decrease with height.

The wind vector at a point may be regarded as the sum of the mean wind vector (static component) and a dynamic, or turbulence, component

$$V(z,t) = \overline{V}(z) + v(z,t) \qquad (1)$$

A consequence of turbulence is that dynamic loading on a structure depends on the size of the eddies. Large eddies, whose dimensions are comparable with the structure, give rise to well correlated pressures as they envelop the structure. On the other hand, small eddies result in pressures on various parts of a structure that become practically uncorrelated with distance of separation. Eddies generated around a typical structure are shown in Fig. 1.
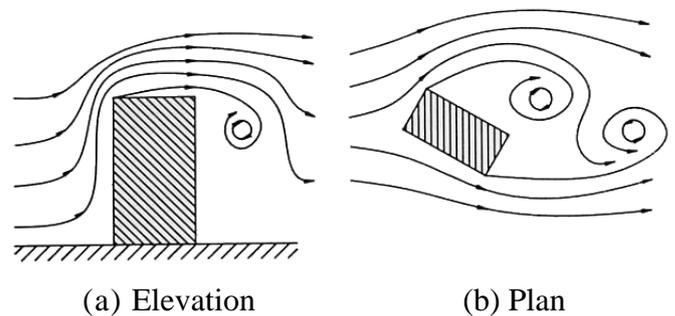

(a) Elevation	(b) Plan

Figure 1: Generation of eddies.

Some structures, particularly those that are tall or slender, respond dynamically to the effects of wind. The best known structural collapse due to wind was the Tacoma Narrows Bridge which occurred in 1940 at a wind speed of only about 19 m/s. It failed after it had developed a coupled torsional and flexural mode of oscillation.

There are several different phenomena giving rise to dynamic response of structures in wind. These include buffeting, vortex shedding, galloping and flutter. Slender structures are likely to be sensitive to dynamic response in line with the wind direction as a consequence of turbulence buffeting. Transverse or cross-wind response is more likely to arise from vortex shedding or galloping but may also result from excitation by turbulence buffeting. Flutter is a coupled motion, often being a combination of bending and torsion, and can result in instability. For build-

ing structures flutter and galloping are generally not an issue.

An important problem associated with wind-induced motion of buildings is concerned with human response to vibration and perception of motion. At this point it will suffice to note that humans are surprisingly sensitive to vibration to the extent that motions may feel uncomfortable even if they correspond to relatively low levels of stress and strain. Therefore, for most tall buildings serviceability considerations govern the design and not strength issues.

The next few sections give a brief introduction to the dynamic response of structures in wind. More details can be found in wind engineering texts (e.g. Holmes (2001); Sachs (1978)).

2 WIND SPEED

At great heights above the surface of the earth, where frictional effects are negligible, air movements are driven by pressure gradients in the atmosphere, which in turn are the thermodynamic consequences of variable solar heating of the earth. This upper level wind speed is known as *the gradient wind velocity.*

Different terrains can be categorized according to their associated roughness length. Table 1 shows the different categories specified in the Australian/New Zealand wind code, AS/NZS1170.2 (2002). Closer to the surface the wind speed is affected by frictional drag of the air stream over the terrain. There is a boundary layer within which the wind speed varies from almost zero, at the surface, to the gradient wind speed at a height known as the gradient height. The thickness of this boundary layer, which may vary from 500 to 3000 m, depends on the type of terrain, as depicted in Fig. 2. As can be seen the gradient height within a large city centre is much higher than it is over the sea where the surface roughness is less.

Table 1. Terrain category and roughness length (z_o)

Terrain category	Roughness length, z_o, (m)
1. Exposed open terrain with few or no obstructions and water surfaces at serviceability wind speeds.	0.002
2. Water surfaces, open terrain, grassland with few, well scattered obstructions having heights generally from 1.5 to 10 m.	0.02
3. Terrain with numerous closely spaced obstructions 3 to 5 m high such as areas of suburban housing.	0.2
4. Terrain with numerous large, high (10.0 m to 30.0 m high) and closely spaced obstructions such as large city centres and well-developed industrial complexes.	2

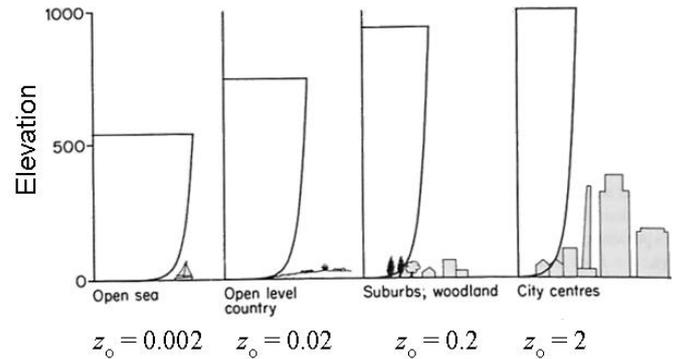$$z_o = 0.002 \quad z_o = 0.02 \quad z_o = 0.2 \quad z_o = 2$$

Figure 2: Mean wind profiles for different terrains.

In practice, it has been found useful to start with a reference wind speed based on statistical analysis of wind speed records obtained at meteorological stations throughout the country. The definition of the reference wind speed varies from one country to another. For example in Australia/New Zealand, it is the 3-sec gust wind speed at a height of 10 m above the ground assuming terrain category 2. Contour maps of reference wind speeds that apply for nominated statistical Return Periods in various countries are usually available.

An engineering wind model for Australia has been developed in Melbourne from the Deaves and Harris model (1978). This model is based on extensive full-scale data and on the classic logarithmic law in which the mean velocity profile in strong winds applicable in non-cyclonic regions (neutral stability conditions) is given by Eq. 2.

$$\overline{V}_z \approx \frac{u^*}{0.4}\left[\log_e\left(\frac{z}{z_o}\right) + 5.75\left(\frac{z}{z_g}\right) - 1.88\left(\frac{z}{z_g}\right)^2 - 1.33\left(\frac{z}{z_g}\right)^3 + 0.25\left(\frac{z}{z_g}\right)^4\right]$$

(2)

The numerical values are based on a mean gradient wind speed of 50 m/s.

For values of z <30.0m the z/z_g values become insignificant and the Eq. 2 simplifies to :

$$\overline{V}_z \approx \frac{u^*}{0.4}\log_e\left(\frac{z}{z_o}\right)$$

(3)

where

$\overline{V}_z$ = the design hourly mean wind speed at height z, in meters per second

u^* = the friction velocity

$$u^* = \sqrt{\frac{surface\ friction\ shear\ stress}{\text{atmospheric density}}}$$

z_g = a "stretched" version of the gradient height in metres (the value ranges from 2000 m to 3300 m), that allows fitting of Eq 13 to $\overline{V}_{z_g}$ = 50 m/s, see Table 2.

$$z_g = \frac{u^*}{6*10^{-4}}$$

Table 2 Roughness length, friction velocity and gradient height

Terrain Category	z_o	u^*	Z_g
	(m)	(m/s)	(m)
1	0.002	1.204	2006
2	0.02	1.385	2308
3	0.2	1.626	2710
4	2	1.963	3272

As given in Table 2, there is an interaction between roughness length and terrain category, so it is necessary to define a terrain category to find the design hourly wind speeds and gust wind speeds. The link between hourly mean and gust wind speeds is as follows:

$$V = \overline{V}\left[1 + 3.7\left(\frac{\sigma_v}{\overline{V}_z}\right)\right]$$

(4)

where

$$\sigma_v = 2.63 \, \eta u^* \left[0.538 + 0.09\log_e\left(\frac{z}{z_o}\right)\right]^{\eta^{16}}$$

(5)

$$\eta = 1.0 - \left(\frac{z}{z_g}\right)$$

(6)

For design, the basic wind speed is classified into three different speeds as follows:

V_s = V_{20yr} = *serviceability limit state design speed* having an estimated probability of exceedence

of 5 % in any one year, that is adopted for the serviceability limit states.

V_p = V_{50yr} = *permissible, or working stress design wind speed* which can be obtained directly from V_u using the relation $V_p = V_u/(1.5)^{0.5}$

V_u = V_{1000yr} = *ultimate limit state design wind speed* having an estimated probability of exceedence of 5 % in a lifetime of 50 years, for the ultimate limit states.

Basic design wind speeds for different directions and different return periods can be derived using a rigorous analysis incorporating probability distributions for wind speed and direction. For example AS/NZS1170.2 provides a wind direction multiplier, which varies from 0.80 for wind from the East to 1.0 for wind from the West, and wind speeds up to a 2000 year return period.

3 DESIGN WIND LOADS

The characteristics of wind pressures on a structure are a function of the characteristics of the approaching wind, the geometry of the structure under consideration, and the geometry and proximity of the structures upwind. The pressures are not steady, but highly fluctuating, partly as a result of the gustiness of the wind, but also because of local vortex shedding at the edges of the structures themselves. The fluctuating pressures can result in fatigue damage to structures, and in dynamic excitation, if the structure happens to be dynamically wind sensitive. The pressures are also not uniformly distributed over the surface of the structure, but vary with position.

The complexities of wind loading, should be kept in mind when applying a design document. Because of the many uncertainties involved, the maximum wind loads experienced by a structure during its lifetime, may vary widely from those assumed in design. Thus, failure or non-failure of a structure in a wind storm can not necessarily be taken as an indication of the non-conservativeness, or conservativeness, of the Wind Loading Standard. The Standards do not apply to buildings or structures that are of unusual shape or location. Wind loading governs the design of some types of structures such as tall buildings and slender towers. It often becomes attractive to make use of experimental wind tunnel data in place of the coefficients given in the Wind Loading Code for these structures.

3.1 Types of Wind Design

Typically for wind sensitive structures three basic wind effects need to be considered.

- *Environmental wind studies -* investigate the wind effects on the surrounding environment caused by erection of the structure (e.g. tall building). This study is particularly important to assess the impact of wind on pedestrians, motor vehicles and architectural features such as fountains, etc, which utilise public domain within the vicinity of the proposed structure.

- *Wind loads for façade -* to assess design wind pressures throughout the surface area of the structure for designing the cladding system. Due to the significant cost of typical facade systems in proportion to the overall cost of very tall buildings, engineers cannot afford the luxury of conservatism in assessing design wind loads. With due consideration to the complexity of building shapes and dynamic characteristics of the wind and building structures, even the most advanced wind codes generally cannot accurately assess design loads. Wind tunnel testing to assess design loads for cladding, is now normal industry practice, with the aim of minimising initial capital costs, and more significantly avoiding expensive maintenance costs associated with malfunctions due to leakage and/or structural failure.

- *Wind loads for structure –* to determine the design wind load for designing the lateral load resisting structural system of a structure to satisfy various design criteria.

3.2 Design Criteria

In terms of designing a structure for lateral wind loads the following basic design criteria need to be satisfied.

- **Stability** against overturning, uplift and/or sliding of the structure as a whole.

- **Strength** of the structural components of the building is required to be sufficient to withstand imposed loading without failure during the life of the structure.

- **Serviceability** for example for buildings, where interstorey and overall deflections are expected to remain within acceptable limits. Control of deflection and drift is imperative for tall buildings with the view to limiting damage and cracking of non structural members such as the facade, internal partitions and ceilings.

The ultimate limit state wind speed is adopted by most international codes to satisfy stability and strength limit state requirements. In many codes such a speed has a 5% probability of being exceeded in a fifty year period.

An additional criterion that requires careful consideration in wind sensitive structures such as tall buildings is the control of sway accelerations when subjected to wind loads under serviceability conditions. Acceptability criteria for vibrations in buildings are frequently expressed in terms of acceleration limits for a one or five year return period wind speed and are based on human tolerance to vibration discomfort in the upper levels of buildings. These limits are also dependent on building sway frequencies. Wind response is relatively sensitive to both mass and stiffness, and response accelerations can be reduced by increasing either or both of these parameters. However, this is in conflict with earthquake design optimisation where loads are minimised in buildings by reducing both the mass and stiffness. Increasing the damping results in a reduction in both the wind and earthquake responses.

The detailed procedure described in wind codes is sub-divided into Static Analysis and Dynamic Analysis methods. The static approach is based on a quasi-steady assumption, and assumes that the building is a fixed rigid body in the wind. The static method is not appropriate for tall structures of exceptional height, slenderness, or susceptibility to vibration in the wind. In practice, static analysis is normally appropriate for structures up to 50 metres in height. The subsequently described dynamic method is for exceptionally tall, slender, or vibration-prone buildings. The Codes not only provide some detailed design guidance with respect to dynamic response, but state specifically that a dynamic analysis *must* be undertaken to determine overall forces on any structure with both a height (or length) to breadth ratio greater than five, and a first mode frequency less than 1 Hertz.

Wind loading codes may give the impression, that wind forces are relatively constant with time. In reality wind forces vary significantly over short time intervals, with large amplitude fluctuations at high frequency intervals. The magnitude and frequency of the fluctuations is dependent on many factors associated with turbulence of the wind and local

gusting effects caused by the structure and surrounding environment.

To simplify this complex wind characteristic, most international codes have adopted a simplified approach by utilising a quasi-steady assumption. This approach simply uses a single value equivalent static wind pressure, to represent the maximum peak pressure the structure would experience.

3.3 *Static Analysis*

This method assumes the quasi-steady approximation. It approximates the peak pressures on building surfaces by the product of the gust dynamic wind pressure and the mean pressure coefficients. The mean pressure coefficients are measured in the wind-tunnel or by full-scale tests and are given by $p_{bar}/q_{z(bar)}$. The implied assumption is that the pressures on the building surface (external and internal) follow faithfully the variations in upwind velocity. Thus, it is assumed that a peak value of wind speed is accompanied by a peak value of pressure or load on the structure. The quasi-steady model has been found to be fairly reliable for wind loading on small structures.

In static analysis, gust wind speed V_z is used to calculate the forces, pressures and moments on the structure.

The main advantages and disadvantages of the quasi-steady/peak gust format, can be summarised as follows:

Advantages:

- Simplicity

- Continuity with previous practice

- Pressure coefficients should need little adjustment for different upwind terrain types

- Existing meteorological data on wind gusts is used directly.

Disadvantages:

- The approach is not suitable for very large structures, or for those with significant dynamic response.

- The response characteristics of the gust anemometers and the natural variability of the peak gusts tend to be incorporated into the wind load estimates.

- The quasi-steady assumption does not work well for cases where the mean pressure coefficient is near zero.

However, the advantages often outweigh the disadvantages - certainly for smaller, stiff structures for which the code is mainly intended.

The philosophy used in specifying the peak loads in AS/NZS 1170.2, has been to approximate the real values of the extremes. In many cases, this has required the adjustment of the quasi-steady pressures using factors such as Area Reduction Factors and Local Pressure Factors.

The dynamic wind pressure at height z is given by

$$\bar{q}_z = 0.6\bar{V}_z{}^2 * 10^{-3}$$

(7)

where

$\bar{V}_z$ = the design gust wind speed at height z, in meters per second.
$= V.M_{(z,cat)}.M_z.M_t.M_i$
where

V = is the basic wind speed

The multiplying factors (M) take into account the type of terrain (M_t), height above ground level (M_z), topography and importance of the structure (M_i). The above derivation essentially forms the basis of most international codes.

The mean base overturning moment M_{bar} is determined by summing the moments resulting from the net effect of the mean forces acting on the structure given by

$$\bar{F}_z = \sum C_{p,e}\bar{q}_z A_z$$

$$\bar{F}_d = \sum C_d \bar{q}_z A_z$$

(8)

$\bar{F}_z$ = the hourly mean net horizontal force acting on a structure at height z

$C_{p,e}$ = the pressure coefficients for both windward and leeward surfaces

A_z = the area of a structure or a part of a structure, at height z, in square metres

$\overline{F}_d$ = the hourly mean drag force acting on discrete elements

C_d = the drag force coefficient for an element of the structure

4 ALONG AND CROSS-WIND LOADING

Not only is the wind approaching a building a complex phenomenon, but the flow pattern generated around a building is equally complicated by the distortion of the mean flow, flow separation, the formation of vortices, and development of the wake. Large wind pressure fluctuations due to these effects can occur on the surface of a building. As a result, large aerodynamic loads are imposed on the structural system and intense localised fluctuating forces act on the facade of such structures. Under the collective influence of these fluctuating forces, a building tends to vibrate in rectilinear and torsional modes, as illustrated in Fig. 3. The amplitude of such oscillations is dependant on the nature of the aerodynamic forces and the dynamic characteristics of the building.
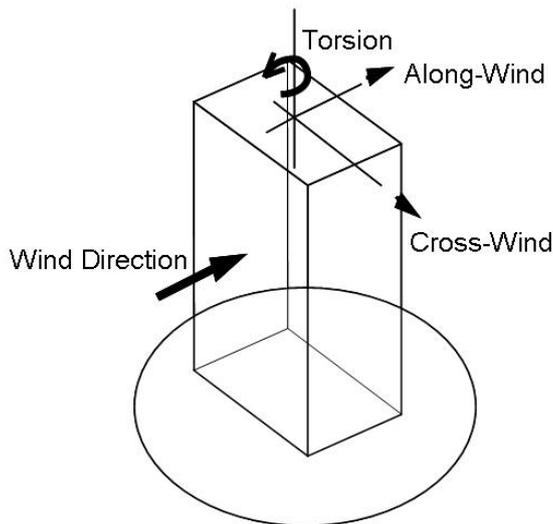

Figure 3: Wind Response Directions

4.1 *Along-Wind Loading*

The along-wind loading or response of a building due to buffeting by wind can be assumed to consist of a mean component due to the action of the mean wind speed (eg, the mean-hourly wind speed) and a fluctuating component due to wind speed variations from the mean. The fluctuating wind is a random mixture of gusts or eddies of various sizes with the larger eddies occurring less often (i.e. with a lower average frequency) than for the smaller eddies. The natural frequency of vibration of most structures is sufficiently higher than the component of the fluctuating load effect imposed by the larger eddies. i.e. the average frequency with which large gusts occur is usually much less than any of the structure's natural frequencies of vibration and so they do not force the structure to respond dynamically. The loading due to those larger gusts (which are sometimes referred to as "background turbulence") can therefore be treated in a similar way as that due to the mean wind. The smaller eddies, however, because they occur more often, may induce the structure to vibrate at or near one (or more) of the structure's natural frequencies of vibration. This in turn induces a magnified dynamic load effect in the structure which can be significant.

The separation of wind loading into mean and fluctuating components is the basis of the so-called "gust-factor" approach, which is treated in many design codes. The mean load component is evaluated from the mean wind speed using pressure and load coefficients. The fluctuating loads are determined separately by a method which makes an allowance for the intensity of turbulence at the site, size reduction effects, and dynamic amplification (Davenport, 1967).

The dynamic response of buildings in the along-wind direction can be predicted with reasonable accuracy by the gust factor approach, provided the wind flow is not significantly affected by the presence of neighbouring tall buildings or surrounding terrain.

4.2 *Cross-Wind Loading*

There are many examples of slender structures that are susceptible to dynamic motion perpendicular to the direction of the wind. Tall chimneys, street lighting standards, towers and cables frequently exhibit this form of oscillation which can be very significant especially if the structural damping is small. Crosswind excitation of modern tall buildings and structures can be divided into three mechanisms (AS/NZ1170.2, 2002) and their higher time derivatives, which are described as follows:

(a) Votex Shedding. The most common source of crosswind excitation is that associated with 'vortex shedding'. Tall buildings are bluff (as opposed to streamlined) bodies that cause the flow to separate from the surface of the structure, rather than follow

the body contour (Fig. 4). For a particular structure, the shed vortices have a dominant periodicity that is defined by the Strouhal number. Hence, the structure is subjected to a periodic cross pressure loading, which results in an alternating crosswind force. If the natural frequency of the structure coincides with the shedding frequency of the vortices, large amplitude displacement response may occur and this is often referred to as *the critical velocity effect*. The asymmetric pressure distribution, created by the vortices around the cross section, results in an alternating transverse force as these vortices are shed. If the structure is flexible, oscillation will occur transverse to the wind and the conditions for resonance would exist if the vortex shedding frequency coincides with the natural frequency of the structure. This situation can give rise to very large oscillations and possibly failure.
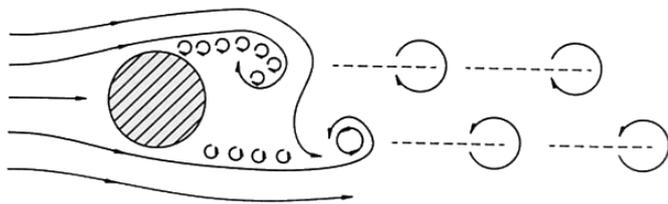

Figure 4: Vortex formation in the wake of a bluff object.

(b) The incident turbulence mechanism. The 'incident turbulence' mechanism refers to the situation where the turbulence properties of the natural wind give rise to changing wind speeds and directions that directly induce varying lift and drag forces and pitching moments on a structure over a wide band of frequencies. The ability of incident turbulence to produce significant contributions to crosswind response depends very much on the ability to generate a crosswind (lift) force on the structure as a function of longitudinal wind speed and angle of attack. In general, this means sections with a high lift curve slope or pitching moment curve slope, such as a streamline bridge deck section or flat deck roof, are possible candidates for this effect.

(c) Higher derivatives of crosswind displacement. There are three commonly recognized displacement dependent excitations, i.e., 'galloping', 'flutter' and 'lock-in', all of which are also dependent on the effects of turbulence in as much as turbulence affects the wake development and, hence, the aerodynamic derivatives. Many formulae are available to calculate these effects (Holmes, 2001). Recently computational fluid dynamics techniques (Tamura, 1999) have also been used to evaluate these effects.

5 WIND TUNNEL TESTS

There are many situations where analytical methods cannot be used to estimate certain types of wind loads and associated structural response. For example, when the aerodynamic shape of the building is rather uncommon or the building is very flexible so that its motion affects the aerodynamic forces acting on it. In such situations, more accurate estimates of wind effects on buildings can be obtained through aeroelastic model testing in a boundary-layer wind tunnel.

Wind tunnel testing is now common practice for design of most tall buildings. In many cases, owners of proposed moderately tall buildings are also encouraged to allow for wind tunnel testing, as the costs associated with such testing can be offset by the substantial savings in the building costs, due to the reduced design wind loading.

The Australian wind code allows wind tunnel testing as a suitable alternative to the code recommendations to determine design wind loads for any structure. In order to regulate the highly specialised area of wind tunnel testing, a national committee has been established to develop a code of practice for wind tunnel testing. Australia is privileged to have some of the leading wind tunnel testing facilities in the world, such as the large 4m high by 12m wide test working section wind tunnel at Monash University.

5.1 *Aeroelastic modeling*

Aeroelastic model techniques take the guesswork out of the gust factor computation by directly measuring the dynamic loads in the wind tunnel. The main objective of the aeroelastic studies is to obtain more accurate prediction of the wind loads. This can only be achieved when the wind and the structure are both properly modelled, such that the model structure responds to the loading system in the same way as the full scale structure.

Wind tunnel tests currently being conducted on buildings and structures can be divided into two major types. The first is concerned with the determination of wind loading effects to enable design of a structure to be wind resistant. The second is concerned with the flow fields induced around the structure. For example how a structure affects pedestrian comfort and safety at ground level or for determining air intake concentration levels of exhaust pollutants.

Although wind tunnel testing attempts to simulate a rather complex situation, the actual models themselves are quite simple, and based on the premise

that the fundamental mode of displacement for a tall building can be approximated by a straight line. In general terms, it is not necessary to achieve a correct mass density distribution along the building height as long as the mass moment of inertial about the pivot point is the same as the prototype density distribution. The pivot point is typically chosen to obtain a mode shape which provides the best agreement with the calculated fundamental mode shapes of the prototype.

5.2 *Interference*

Buildings of similar size located in close proximity to the proposed building can cause large increases in cross-wind responses. The designer should not only consider the existing conditions but make allowance for future changes in the surrounding area during the design life of the structure. Obviously this needs responsible subjective engineering judgment making use of the best available knowledge at the time of design. Fortunately in wind tunnel studies, surroundings comprising of existing and/or future buildings can easily be incorporated with relatively minor costs.

As a guideline, interference due to buildings of similar size to the subject building, located within a distance equal to 10 times the building width, need be considered.

5.3 *Wind tunnel tests*

Wind tunnel testing is a powerful tool that allows engineers to determine the nature and intensity of wind forces acting on complex structures. Wind tunnel testing is particularly useful when the complexity of the structure and the surrounding terrain, resulting in complex wind flows, does not allow the determination of wind forces using simplified code provisions.

Wind tunnel testing involves blowing air on the building model under consideration and its surroundings at various angles relative to the building orientation representing the wind directions. This is typically achieved by placing the complete model on a rotating platform within the wind tunnel. Once testing is completed for a selected direction, the platform is simply rotated by a chosen increment to represent a new wind direction. A typical wind tunnel model testing facility located at Monash University is illustrated in Figure 5.

Wind tunnels are either an open-circuit or closed circuit type with a working section and a working length. A schematic of a typical open-circuit wind tunnel is shown in Figure 6. For average size tun-

nels testing tall buildings, the 1:400 scale model of the natural wind is usually generated using *the augmented growth method*. This method generates large-scale turbulence using devices such as trip boards and spires upstream of the fetch length. Carpet or roughness blocks are used along the fetch length to generate the required velocity profile. For

(A) 2m by 2m working section

(B) Part of Melbourne City Model

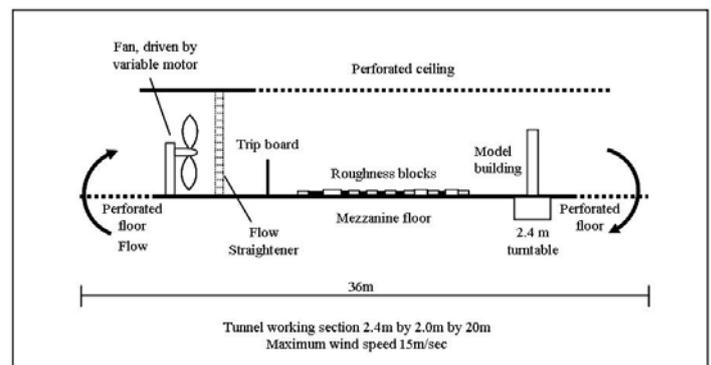Figure 5: Wind tunnel testing at Monash University

Figure 6: Schematic of a typical open-circuit wind tunnel

larger tunnels, generation of 1:200 or even 1:100 scale models may be possible.

In order to use wind tunnel results to aid in the prediction of wind forces acting on full-scale structure, the behavior of the natural wind must be satisfactorily modelled by the wind tunnel. The following variables are of particular importance:

$\overline{U}(z)$	=	mean longitudinal wind velocity at height z
σ_U	=	standard deviation of velocity fluctuations
n	=	frequency related to velocity fluctuations
$S_U(n)$	=	power spectral density of the velocity fluctuations
L	=	measurement of length
λ_L	=	length scale associated with the modelled building and natural wind
λ_T	=	time scale

To model the natural wind successfully, and maintain dynamic similarity between model and full-scale results, the following non-dimensional parameters are kept as near to constant as possible between the natural wind and the wind tunnel. They are: the velocity profile $\overline{U}(z)/\overline{U}(z_O)$, that is the variation of velocity with height normalised with respect to the values at height z_O, the height of the building under investigation; the turbulence intensity $\sigma_U/\overline{U}$; and the normalised power spectral density, $nS_U(n)/\sigma^2_U$, which defines the energy present in the turbulence at various frequencies. Reynold's number is not an important parameter in this case as a sharp edged model is used.

To relate wind tunnel pressure measurements to full-scale values, length and time scales must be determined. Let us assume that a length ratio of 1:400 and a velocity scale between the wind speed in the tunnel and full-scale winds of 1:3 is chosen. This results in a time scale of approximately 1:133. In other words, a one second record obtained in the wind tunnel, corresponds to a 133 second wind record in the real world. As the time scale is the inverse of the frequency scale, the rigid model is deemed to possess a natural frequency 133 times that of the full-scale building being modelled. By equating the model and full-scale Strouhal numbers, it can be seen that the time scale depends only on the length scale and the ratio of mean wind speed at the top of the model building to mean wind speed at the top of the full-scale building.

The design wind speed is based on meteorological data for the given city or area which is analysed to produce the required probability distribution of gust wind speeds. By appropriate integration processes and application of necessary scaling factors, directional wind speeds for the wind tunnel testing can be determined.

5.4 *Wind Drift Design*

Limits for wind deflection or the relative deflection between adjacent floors in buildings are specified in many wind loading and design codes (eg, New Zealand Code, NZS 4203; Canadian Code, NBCC). In some cases these limits are given as recommendations rather that as mandatory requirements.

In summary, the main reasons for adopting wind drift deflection limits are:

(a) To limit damage to the cladding on the building facade and to partitions and interior finishes;
(b) To reduce the effects of motion perceptibility;
(c) To limit the P-Delta or secondary loading effects.

Drift limits can be specified in terms of an average for the building (usually specified as the ratio of top deflection/building height), or considered as storey drift. There are two major contributions to storey drift. The first is the shear or "racking drift" which is the component of the relative movement of the adjacent floors measured in a direction parallel to the floors. The second is the component of displacement or "chord" drift caused by the relative rotations between floors. The sum of these two components gives the total storey drift or the difference in horizontal displacement between adjacent floors. With regard to damage in the partitions and facade cladding, it is usually only the shear drift components that induce significant loads in these non-structural elements.

Drift Damage limits for cladding and partitions should be specified in terms of serviceability wind speeds, and the limit should be related to the type of non structural materials used and the methods of fixing. For example, an unlined industrial building with metal cladding can tolerate significantly larger drifts than an apartment building fitted out with divided walls lined with plasterboard or masonry infill walls.

Because there is a lack of information available on the performance of partitions and cladding sys-

tems under racking loads (and a wide range of different systems are used in practice), it is difficult to establish a rational basis for specifying drift limits. Currently used limits appear to be based on judgment developed from satisfactory past performance of buildings.

The following limits for the prevention of damage to non- structural elements from Cooney and King (1988) provide some guidance:

(a) In-plane loading of walls of masonry and plaster

    d<h/500   <10 mm

(b) Moveable partitions
    d<h/500   <25 mm

(c) In-plane loads on facades and curtain walls
    d<h/150

(d) Fixed glazing
    d< 2b   < 10 mm

where,

d = shear or "racking" drift
h = height of wall or cladding unit
b = clearance in window frame

Most cladding systems can be designed and detailed to accept relatively large drifts. Thus an acceptable approach for cladding systems is to carry out a specific design, taking into account the drifts and loads imposed on the cladding under the serviceability wind speeds.

Although the problem of motion perception and human comfort is related to drift limits, it appears that it is best to specify criteria for motion perception acceptability in terms of lateral accelerations. .

P-Δ effects should be considered in the design analysis required to check strength and stability under the ultimate limit state wind speeds. Methods for calculating these secondary load effects are well established, and there seems to be no need to control them by arbitrarily set drift limits.

If it is accepted that cladding performance and P-Δ effects should be considered by specific design, then the only reason for specifying wind load drift limits is to prevent damage to partitions and interior finishes. Unless specific test-based data is available for setting rack drift limits for interior finishing, it is recommended that a limit if h/500 be used for the maximum inter-story racking drift under serviceability limit wind speeds. This value is consistent with a recommendation given in NBCC and survey results which indicated that designers of steel framed buildings in USA use a drift limit ranging from between

h/600 to h/200. (Note that in some codes and specifications, it is not clear whether the limit refers to average drifts or maximum storey drifts, or whether total drifts or the racking component should be used.)

5.5 *Wind Loads on Cladding*

Wind loading criteria specifically relating to exterior wall elements have received little attention in the form of documentation within building codes around the world, until the past few years. Wind tunnel model studies of building components, both structural and exterior facade elements, began nearly 30 years ago, but until the last decade, they were generally only performed for special building structures.

Such test programs utilize static pressure models for investigating the wind pressure conditions effecting the exterior wall components, and more complex, aeroelastic models to study the dynamic response of a few very special structures. More recently, "force balance" procedures are being pursued to obtain more accurate response predictions for the primary structural form, while still utilizing the same model being employed for the exterior facade pressure testing.

Although the wind tunnel investigatory community world wide, which has consistently performed such testing is relatively small, a wealth of documented data has been generated. Unfortunately, little of this data has been assimilated into design guidelines for wind loading within the various codes, with only the A.N.S.I. code (1982) specifically differentiating the requirements in wind loading between the primary structure and exterior facade components/systems.

Clearly, wind tunnel investigations have shown that the effects and factors producing wind loading design criteria for exterior wall components can be significantly different than those cases defining the design load criteria for the primary structure even though they both are derived from the same wind environment. This critical difference is directly related to the behavioural response characteristics of each system.

The usually highly redundant primary structure feels little of the specific effects of localised peak pressures such as may occur at building corners, setbacks, parapets and other changes in building configuration.

The exterior wall components which usually exhibit low degrees of structural redundancy, if any, can be significantly impacted by such local peak load conditions. This is the primary factor in acknowledging that the extensive wind loading crite-

ria, as presented in almost every code for the primary structure, were developed based on a philosophy which recognises the inherent redundancy of the structure. In many cases such structural building criteria may lead to unconservative loading conditions if applied directly as the wind loading criteria for the exterior facade elements. The wind conditions and directionality defining the wind loading criteria for the primary structure versus those for the exterior cladding systems can also be significantly different.

The normally considered translational and torsional deformations of the structural frame can be magnified, especially in tall, more slender buildings, by the "vortex shedding" behaviour of the wind/structure interaction leading to significant "cross-wind" deformation of the structure. Although in the design, wind loads are treated as static load events, the actual wind and its application to the building surfaces are always dynamic in nature, and this actual response behaviour needs to always be considered.

Building structures, which through wind tunnel studies are found to exhibit significant dynamic acceleration characteristics, should be even more carefully evaluated with respect to the exterior facade system response, in relation to the primary structure. Increases to the inherent or induced damping systems of the primary structure have been required in some buildings not only to modify the structure's dynamic behaviour with respect to human response, but to also achieve an acceptable performance range for the exterior facade systems.

With the advent of environmentally tighter buildings since the mid-1970s, increased differential pressures between the interior and exterior environments have also provided additional secondary conditions which need to be considered in conjunction with the external wind pressures. It should be noted that although structural wind design loadings for the primary structural systems generally decrease at the lower elevations, that due to ground turbulence effects, "downwash" effects, and significant building configuration changes to the facade at the lower parts of the building, the facade system design pressures may not decrease nearly as significantly.

Generally, the design concerns for individual cladding system components relate to wind pressure conditions perpendicular to the surface plane. The interface compatibility issues between the attached cladding systems and the primary structure generally relate to the deformations in the exterior plane of the structure.

The most common effect needing consideration is the "shear racking" or horizontal distortion of the structure's beam-column frames at the exterior of the building due to lateral deformation of the structure.

The attached exterior cladding systems attempt to respond to the deformed shape of the supporting structure inducing in-place deformations within cladding systems which, if restrained without relief mechanisms, generate significant force mechanisms leading to component distress or failure. Structural deformations due to lateral loads produce horizontal and vertical translations, and rotational movements which need to be absorbed within the facade system and within its anchorage elements to the primary structure.

The response to such deformation systems by the facade system components is most easily achieved by utilising smaller sizes for less ductile components, and allowing larger sizes when using more ductile elements. The greatest degree of susceptibility to distress is found among those elements such as large panels of glass which exhibit low levels of in-plane ductility. Although such horizontal wracking of the exterior facade systems is normally limited in magnitude for typical floor-to-floor dimensions by the limits of acceptable performance for structural behaviour or human physiological response, special tall floors or "soft structure" zones can produce unacceptable response ranges for some of the facade systems components.

It can be observed that the response to the "shear racking" effect, with respect to the differential deformation compatibility between exterior facade elements and supporting structural systems, would generally be more severe for the lower deflection ratios of h/400 - h/500. It should again be noted that the optimization of the primary structure's design utilizing the stronger, more ductile steel materials versus the less strong, less ductile concrete materials, produces a greater contrast when considering the interface deformation compatibility conditions of the attached systems.

It should also be noted that as the geometrical placement of the exterior skin becomes more directly aligned in the same plane as the exterior structural elements, the deformed configuration of the skin must more closely duplicate that of the deformed structure.

For more outwardly located facade systems, the interface anchorage system can provide a performance buffer and relieve some of the deformation differential. Generally, the response effects of the wind loading "shear wracking" deformation will need to be combined with the effects of gravity loading along the exterior face.

The differential axial deformations of the structure's exterior columns, due to the "cantilever behaviour" of the structure under wind loading, results in a

further series of differential, deformation design considerations, with the similar combined effects of shrinkage and creep.

6 COMFORT CRITERIA: HUMAN RESPONSE TO BUILDING MOTION

There is no generally accepted international standards for comfort criteria in tall building design. A considerable amount of research has however been carried out into the important physiological and psychological parameters that affect human perception to motion and vibration in the low frequency range of 0-1 Hz encountered in tall buildings. These parameters include the occupant's expectancy and experience, their activity, body posture and orientation, visual and acoustic cues, and the amplitude, frequency, and accelerations for both the translational and rotational motions to which the occupant is subjected. Table 3 gives some guidelines on general human perception levels.

Table 3. Human perception levels

LEVEL	ACCELERATION (m / sec²)	EFFECT
1	< 0.05	Humans cannot perceive motion
2	0.05 - 0.1	a) Sensitive people can perceive motion; b) hanging objects may move slightly
3	0.1 - 0.25	a) Majority of people will perceive motion; b) level of motion may affect desk work; c) long - term exposure may produce motion sickness
4	0.25 - 0.4	a) Desk work becomes difficult or almost impossible; b) ambulation still possible
5	0.4 - 0.5	a) People strongly perceive motion; b) difficult to walk naturally; c) standing people may lose balance.
6	0.5 - 0.6	Most people cannot tolerate motion and are unable to walk naturally
7	0.6 - 0.7	People cannot walk or tolerate motion.
8	> 0.85	Objects begin to fall and people may be injured

Acceleration limits are a function of the frequency of the vibration being felt. Upper limits have been recommended for corresponding frequencies of vibration with the relationship suggested by Irwin (1978). Peak acceleration limits as suggested by

Melbourne (1989) and Cheung have been plotted along with the Irwin's E2 curve in Fig. 7. To obtain the peak value of acceleration, the root mean square (rms) value can be multiplied by a peak factor. The peak factor is generally between 3 and 4.
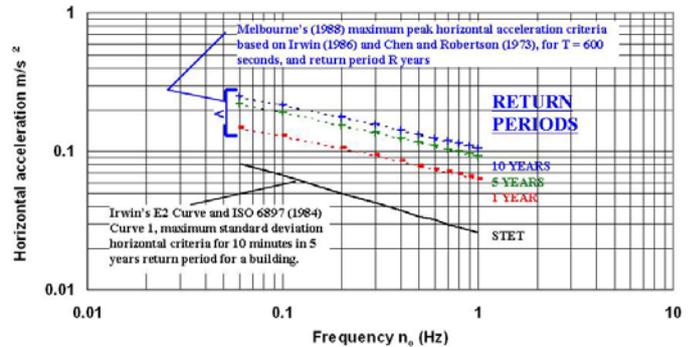

Figure 7: Horizontal acceleration criteria for occupancy comfort in buildings

7 DAMPERS

The damping in a mechanical or structural system is a measure of the rate at which the energy of motion of the system is dissipated. All real systems have some form of damping. An example is friction in a bearing. Another example is the viscous damping created by the oil within an automotive shock absorber. In many systems, damping is not helpful and it has to be overcome by the system input. In the case of wind sensitive structures such as tall buildings, however, it is beneficial, as damping reduces motion, making the building feel more stable to its occupants.

Controlling vibrations by increasing the effective damping can be a cost effective solution. Occasionally, it is the only practical and economical means of reducing resonant vibrations. Types of damping systems that can be implemented include, passive, active and semi-active dampers. Some examples of passive dampers are:

- Tuned Mass Damper (TMD) (an example is given in Fig. 8)
- Distributed Viscous Dampers
- Tuned Liquid Column Dampers (TLCD), also known as *Liquid Column Vibration Absorbers* (LVCA)
- Tuned Sloshing Water Dampers (TSWD)
- Impact Type Dampers
- Visco-Elastic Dampers
- Friction Dampers

Examples of active and hybrid dampers include:

- Active Tuned Mass Damper (ATMD)
- Active Mass Driver (AMD)

Examples of semi-active dampers include :
- Variable Stiffness Dampers
- Hydraulic dampers
- Controllable Fluid Dampers
- Magneto-Rheological (MR) Dampers
- Electro-Rheological (ER) Dampers
- Variable Friction Dampers

While general design philosophy tends to favour passive damping systems due to their lower capital and maintenance costs, active or semi-active dampers may be the ideal solution for certain vibration problems. More details about passive and active systems to control vibrations are given by Soong and Costantinou (1994).
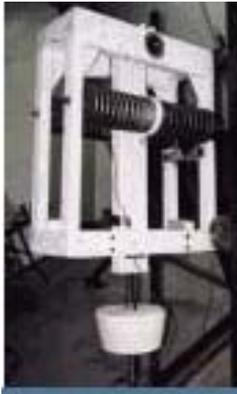
Figure 8: One of the TMDs designed for the skybridge legs of the Petronas Towers by RWDI Inc. (12 TMDs were installed three in each of the four legs)

8 COMPUTATIONAL FLUID DYNAMICS TECHNIQUES

In a number of fields, numerical simulation by means of CFD (Computational Fluid Dynamics) is becoming a promising and powerful tool for predicting the behaviour of structures in practical engineering cases. This includes applications involving fluid-structure interaction.

CFD techniques may be used for determination of wind effects where Standards are sometimes not directly or as easily applicable, for instance when designing tall buildings and non conventional structures.

Some examples of CFD studies conducted at the University of Melbourne are given below. A typical 1: 400 scale model of a 40 m x 40 m x 300 m building is shown in Figure 9 and Figure 10. The maxi-

mum wind speed at the top of the building is 40 m/s (in the prototype). The turbulence intensity follows the Australian code terrain category 2 wind. This analysis was conducted using program CFX10. The turbulence model is SST (shear stress transport).
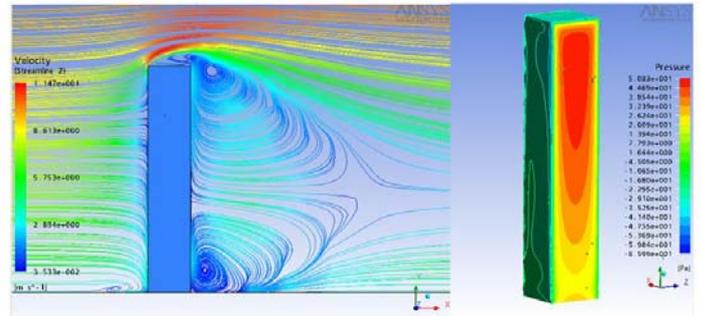

Figure 9: Stream line of a flow over a building model – Vertical view & Pressure distribution
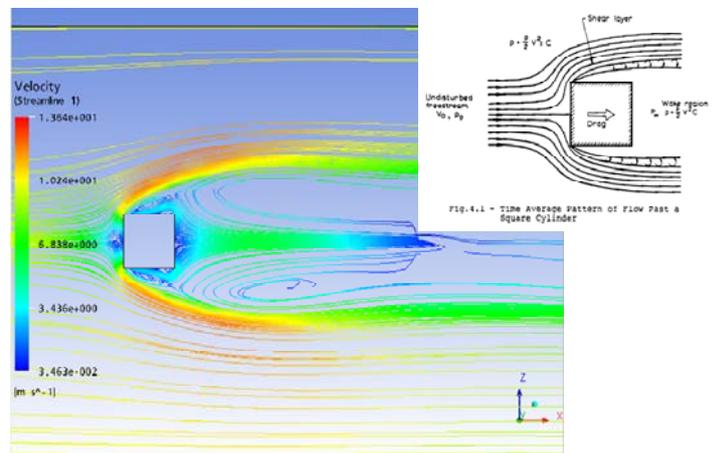

Figure 10: Stream line of a flow over a building model – Plan view at GL

9 CONCLUDING REMARKS

This paper has considered a number of key factors associated with the design of tall buildings to the effects of wind loading. The general design requirements for structural strength and serviceability assume particular importance in the case of tall building design as significant dynamic response can result from both buffeting and cross-wind wind loading excitation mechanisms. Serviceability with respect to occupier perception of lateral vibration response can become the governing design issue necessitating the introduction of purpose-designed damping systems in order to reduce these vibrations to acceptable levels. Dynamic response levels also play an important role in the detailed design of façade systems. State of the art boundary layer wind tunnel testing, for determining global and local force coefficients and the effects of wind directionality, topographical features and nearby structures on

structural response, is recognized as being particularly useful to tall building design. The emerging use of CFD codes, particularly at the concept design stage, is also noted as assuming increasing importance in the design of tall buildings.

10 ACKNOWLEDGEMENTS

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