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Wind Design of Tall Buildings: The State of the Art

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ABSTRACT: The construction of tall and slender buildings has seen recent growth in many cities around the world. Tall buildings are susceptible to dynamic excitation under wind effects which typically govern the structural design for strength, stability, and serviceability. This paper presents the state of the art in the analysis and design of tall buildings against wind effects. Structural design criteria are discussed in detail, with serviceability criteria relating to occupant comfort noted as being of particular importance. The latest in wind analysis tools and techniques is also presented. Wind tunnel testing remains the gold standard for determining wind loads on tall buildings, while the emerging use of computational fluid dynamics (CFD) is noted as being particularly useful for concept design stages. The paper aims to provide a valuable reference for engineers, architects, and designers involved in wind analysis and design of tall buildings.

Keywords: wind loading, tall buildings, human perception of motion, computational fluid dynamics, wind drift

1 INTRODUCTION

Wind loads on tall and slender buildings are critical and often govern their overall structural design. Wind loads can generate large pressures on the building surface and large lateral forces on the structural frame, resulting in high demand on structural and facade elements. Moreover, wind forces can induce resonant responses which can lead to excessive floor acceleration and occupant discomfort particularly at top storeys (Burton et al., 2015; Kwok et al., 2009; Lamb and Kwok, 2017; Lamb et al., 2013). These wind effects, and others presented in this paper, need to be carefully considered when assessing the structural performance of tall buildings against design criteria for stability, serviceability, and strength.

This paper describes general principles and state-of-the-art tools for analysing and designing tall buildings against the effects of wind. The general approach for analysing wind effects on structures is presented first. Critical wind design criteria and wind mitigation measures for tall buildings are discussed thereafter. Finally, the latest in wind analysis tools is presented and their advantages and limitations are discussed. While this paper aims to provide an overview of current wind design practices, in-depth background on wind engineering theory can be found in texts such as Holmes (2015), Stathopoulos and Baniotopoulos (2007), and Tamura and Kareem (2013).

2 ANALYSIS OF WIND EFFECTS ON TALL BUILDINGS

The basic approach for analysing wind effects on structures, commonly referred to as the wind loading chain, was first proposed by Alan Davenport in his 1961 doctoral thesis (Davenport, 1961). The wind loading chain is a logical, step-by-step approach that proposes that wind loads on a structure may be estimated from the combined effect of various contributing factors that include the local wind climate and site exposure, and the aerodynamic and structural properties of the building. These factors are illustrated schematically in Figure 1.

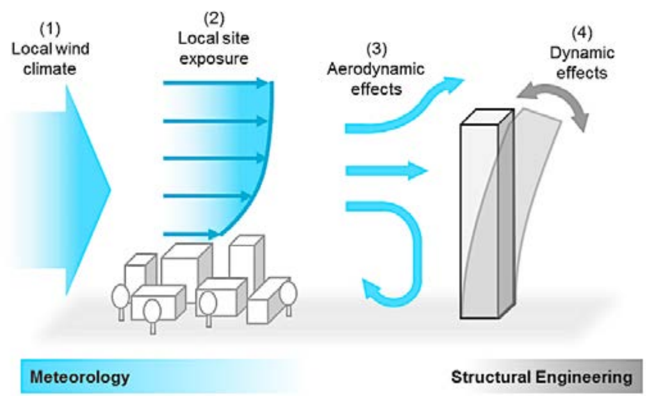


Figure 1: Schematic depiction of the wind loading chain (Abu-Zidan, 2019)

The first contributing factor is the local wind climate where meteorological models, derived from historical wind measurements, are used to estimate design wind speeds for a given return period. As the approaching wind travels towards the building, it interacts with local site conditions. Hence, the effects of topology, terrain roughness, and site exposure are considered in the second link of the wind loading chain. As the wind profile reaches the building, it generates aerodynamic loads that act on the structure. The magnitude and dynamic properties of these loads are heavily influenced by the geometrical and aerodynamic properties of the building. These effects are evaluated in the third link of the wind loading chain. Finally, the fourth link accounts for the dynamic excitation of susceptible structures, such as tall and flexible buildings. Dynamic effects are influenced by structural properties, such as mass, stiffness, and damping, as well as the frequency and magnitude of the exciting load.

Like a physical chain, the strength of the overall wind prediction is dictated by the weakest link in the chain. It is therefore important that each of the contributing factors is assessed accurately. These are discussed in further detail later in the following subsections.

To account for the inherent randomness of wind, Davenport (1961) pioneered the use of the statistical concept of stationary random processes in wind engineering. This entails that the statistical properties of a wind signal are stationary and determinable even though the wind signal itself is inherently random and unpredictable. The assumption of a stationary process was seen to hold true for synoptic wind signals with a duration between 10 minutes to 1 hour (Isyumov, 2012; Solari, 2017), which corresponds to the spectral gap observed in energy spectra of synoptic wind signals (Figure 2). The stationary process approach of Davenport has proved highly effective in wind engineering applications over the years and is still used today for designing structures under synoptic wind conditions.

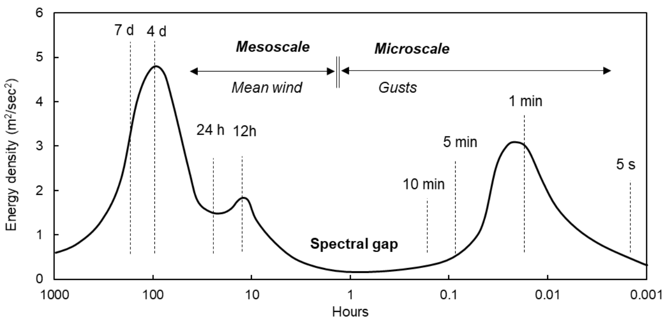


Figure 2: Horizontal wind speed spectrum within the atmospheric boundary layer (Van der Hoven, 1957)

2.1 *Predicting local wind climate*

Local wind climate is influenced by many factors that range from microscale features to mesoscale metrological phenomena. Large-scale weather systems are caused by pressure gradients due to cooling at the earth's poles, and by the earth's rotation known as the Coriolis effect. These large-scale systems result in synoptic wind events. Occasionally, mesoscale systems such as tropical cyclones, downbursts, and tornadoes, result in non-synoptic wind events. These events tend to be powerful and destructive, but they are short-lived, geographically isolated, and less frequent than synoptic winds. Therefore, this paper will only focus on wind loading under synoptic wind events.

Basic wind speeds for different return periods can be derived using extreme value analysis of historical records. Most commonly, the Gumbel (Type I) distribution is used as it provides a good fit for records of extreme winds. The Gumbel distribution is a specific case of the Generalised Extreme Value (GEV) distribution.

Several methods are available for fitting historical data to Type I GEV distributions. Most widely used are Gumbel's method (1954) and Gringorten's method (1963). These are based on a single maximum gust speed reading for each year and do not account for multiple windstorm readings that may occur in the same year. To overcome this limitation, Holmes and Moriarty (1999) proposed the "peak-over-threshold" approach which considers all independent storm events over a given period where the wind speed is above a minimum threshold. This approach has been adopted for determining design wind speed in the Australian wind design standard AS/NZS 1170.2:2021 (Standards Australia, 2021).

Before extreme value analysis can be carried out, wind records must be separated by storm type and converted to standard height and terrain conditions. Correction to a common gust duration may also be required particularly for data obtained using older equipment (Holmes and Ginger, 2012). The definition of basic wind speed varies from one country to another. For instance, in Australia and New Zealand, it is the 0.2-sec gust wind speed at a reference height of 10 m above the ground in open country terrain.

The Australian wind standard AS/NZS 1170.2:2021 (Standards Australia, 2021) provides the following expression for basic wind speed V_R in Melbourne, Australia as a function of return period R in years. This expression was derived using the peaks-over-threshold method (Holmes, 2002). The standard also provides a wind direction multiplier, which, for Melbourne, varies from 0.80 for wind from the East to 1.0 for wind from the West.

$$V_R = 67 - 41R^{-0.1} \tag{2.1}$$

The return period (R) in Eq. (2.1) is the reciprocal of the annual probability of exceedance. That is, the probability that a 1000-year wind (R = 1000 years) will be exceeded in any given year is 1/1000 or 0.1%. This is not to be confused with the probability of exceedance over the expected lifespan of the structure, calculated as follows:

$$P(V \geq V_R) = 1 - \left(1 - \frac{1}{R}\right)^L \tag{2.2}$$

where L is the lifespan of the structure (in years). Using Eq. (2.2), the probability of a structure with a 50-year lifespan experiencing a 1000-year wind during its lifespan is 4.9%.

The appropriate return period R depends on design criteria. For ultimate limit state design in Australia, the National Construction Code (ABCB, 2019) and AS/NZS 1170.0:2002 (Standards Australia, 2002) specify return periods based on the importance level of the building. Most buildings correspond to importance levels 2 or 3 with a return period of 500-1000 years for ultimate state design. For serviceability limit state design, AS/NZS 1170.0:2002 specifies a lower return period of 25 years.

2.2 *Local site exposure and influence of terrain*

The second link in the wind loading chain involves estimating the profile of the atmospheric boundary layer at the location of the building. The atmospheric boundary layer (ABL) refers to the wind profile that develops in the lower part of the atmosphere under synoptic winds. It extends from the ground to several hundred meters where frictional effects of the ground become negligible. This upper-level wind is known as the gradient wind speed.

The shape of the ABL profile is influenced by local terrain and topological conditions at the ground surface. Smooth terrains, such as open fields, generate profiles with higher wind speeds and lower turbulence near the ground. Conversely, terrains of high roughness, such as cities and suburbs, generate profiles with slower wind speeds and higher turbulence near the ground. Table 1 lists the various terrain categories specified in the latest revision of AS/NZS 1170.2:2021 and their associated roughness lengths z_0. The relationship between the terrain category number (TC) and roughness length z_0 is given in the standard as follow:

$$z_0 = 2 \times 10^{(TC-4)} \tag{2.3}$$

Table 1: Terrain categories in AS/NZS 1170.2:2021 with corresponding roughness lengths (Standards Australia, 2021)

Terrain category	**Description**	**z_0 (m)**
1	Very exposed open terrain with very few or no obstructions and all water surfaces *(e.g. flat treeless poorly grassed plains, open ocean, rivers, canals, bays, and lakes).*	0.002
2	Open terrain with well-scattered obstructions of 1.5 – 5 m in height and no more than two obstructions per hectare *(e.g. farmland and cleared subdivisions with isolated trees and uncut grass).*	0.02
2.5	Terrain with some trees or isolated obstructions. Terrain in developing outer areas with scattered housing, or large acreage developments with 2-10 buildings per hectare.	0.063
3	Terrain with numerous closely spaced obstructions of 3 to 10 m in height. Minimum obstruction density equivalent to at least 10 houses per hectare *(e.g. suburban housing, light industrial estates, dense forests)*	0.2
4	Terrain with numerous large, high (10-30 m tall), and closely spaced constructions *(e.g. large city centres and well-developed industrial complexes).*	2

The thickness of the ABL profile also varies with terrain roughness. As depicted in Figure 3, the gradient height within a large city centre is much higher than it is over the sea where the surface roughness is lower.

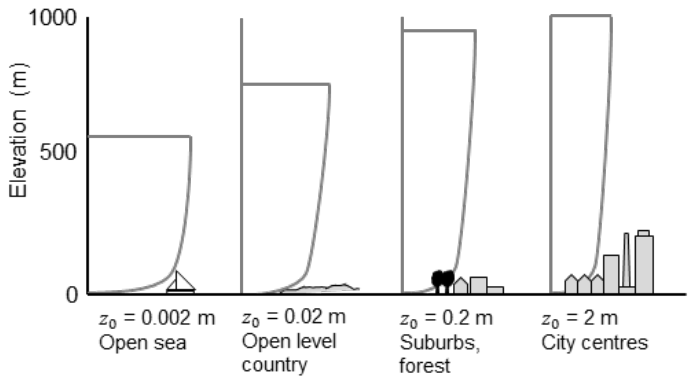


Figure 3: Mean wind profiles of different terrains (Mendis et al., 2007)

Two mathematical models are commonly used to describe mean speed profiles of the ABL as a function of height: the logarithmic (log) law, and the power law. The log law is derived from basic theoretical equations of fluid flow over a no-slip flat surface. The log law takes the following mathematical form:

$$\bar{V}(z) = \frac{u_*}{\kappa} \ln\left(\frac{z}{z_0}\right) \quad (2.4)$$

where $\bar{V}$ is the hourly mean wind speed at height z; u_* is the friction velocity; κ is the von Karman constant = 0.4; and z_0 is the roughness length of the terrain. The main drawback of the log law is that it is difficult to manipulate mathematically. Alternatively, the power law may be used with the following expression:

$$\bar{V}(z) = \bar{V}_{ref} \left(\frac{z}{z_{ref}}\right)^{\alpha} \quad (2.5)$$

where $\bar{V}_{ref}$ is the reference mean wind speed at a reference height z_{ref}, typically taken at 10 meters from the ground, and α is an exponent of terrain roughness.

Although the power law has no theoretical basis, it provides a reasonable approximation of the log law profile. The power law is more easily integrated over height z which makes it convenient for wind engineering applications, such as for calculating the base bending moment of a building under wind loading (Holmes, 2015). Wind tunnel tests of buildings typically report wind speed in power law format.

Both the log law and the power law account for the influence of terrain roughness on the mean wind speed. This is achieved with the z_0 variable in the log law and the α variable in the power law. Suggested values of z_0 for various terrains is listed in Table 1. Corresponding values for α can be calculated using the following equation:

$$\alpha = \left[\ln\left(\frac{z_{ref}}{z_0}\right)\right]^{-1} \quad (2.6)$$

It is important to note that the log and power laws are expressions for mean wind speed profiles and not gust speed profiles. The following expression can be used for converting between hourly mean wind speed $\bar{V}$ and gust wind speed V at a given height (z):

$$V(z) = \bar{V}(z)[1 + g_v\, I(z)] \quad (2.7)$$

where $I(z)$ is the turbulence intensity = $\sigma_V/\bar{V}$, and g_V is the peak factor for upwind velocity fluctuation. The AS/NZS 1170.2:2021 code specifies a peak factor value g_V = 3.4 for the 0.2 s gust wind speed definition used in the code. The code also provides tabulated values of $I(z)$ for various heights and terrain categories.

2.3 *Aerodynamic loads and dynamic response*

Wind flow interacting with the building geometry generates complex and unsteady flow patterns that are characterised by flow separation, development of the wake region, and shedding of vortices. These effects produce fluctuating wind pressures that act on the surface of the building. As a result, large aerodynamic loads are imposed on the structure while intense localised fluctuating forces act on the facade.

Under the collective influence of these fluctuating forces, a building tends to vibrate in translational and rotational modes, as illustrated in Figure 4. The amplitude of the oscillations is dependent on the nature of the aerodynamic forces and the dynamic properties of the building. These are assessed in the third and fourth stages of the wind loading chain.

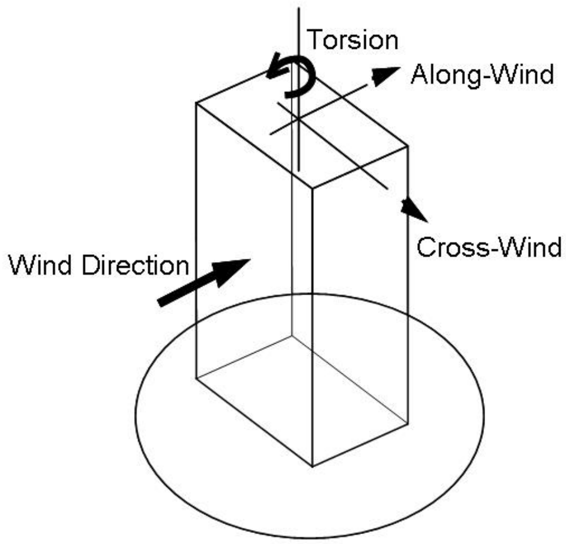


Figure 4: Wind Response Directions (Mendis et al., 2007)

For tall buildings with a natural frequency below 1 Hz, dynamic excitation of the structure is likely to occur at probable wind speeds (Holmes, 2015). The dynamic response of a tall building under wind loads often imposes high demands on its strength design as well as its serviceability performance. An important problem associated with the wind-induced motion of buildings is concerned with human response to vibration and perception of motion. It is worth noting that humans are quite sensitive to vibration to the extent that motions may feel uncomfortable even if they associate with relatively low levels of stress and strain. Therefore, for most tall buildings, serviceability considerations such as top deflections and accelerations govern the design.

2.3.1 *Along-wind response*

The along-wind response of a building constitutes a mean component due to the action of the mean wind speed and a fluctuating component due to the interaction of the structure with the turbulence in the approach flow. Turbulence in the approaching wind is a random mixture of gusts of various sizes ranging from larger eddies that occur less often (i.e. with a lower average frequency) to smaller eddies that are more frequent.

The natural frequency of most structures is sufficiently higher than the component of the fluctuating load imposed by larger eddies. Hence, loading due to large gusts (also referred to as background turbulence) does not induce a dynamic response in the structure and can therefore be treated in a similar way to loading due to mean wind. However, smaller eddies with a higher frequency may cause the structure to vibrate with a frequency close to one or more of its natural frequencies. This in turn induces a magnified dynamic load effect which can be significant on the structure.

The along-wind structural response spectrum has the general form illustrated in Figure 5. The region marked as ‘B’ in Figure 5 is the background component of the response spectrum. It represents the response of the building to the slow-moving, large-scale turbulence in the incoming flow. The region marked as ‘R’ is the resonant component of the spectrum and it occurs due to the response of the structure to smaller-scaled eddies of higher frequencies that fall in the natural frequency range of the structure.

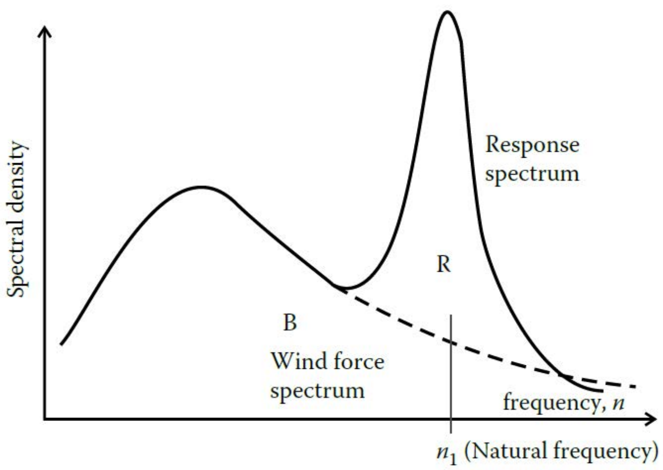


Figure 5: Spectrum of along-wind structural response (Holmes, 2015)

The separation of wind loading into mean and fluctuating components is the basis of the so-called "gust-factor" approach, which is used 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 that makes an allowance for the turbulence intensity 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.

2.3.2 *Cross-wind response*

The cross-wind response in modern tall buildings is typically larger than the along-wind response and often governs the structural design (Kwok, 1982; Saunders and Melbourne, 1977). The dominant mechanism for crosswind excitation in tall buildings occurs due to *vortex shedding*. Vortex shedding is a flow phenomenon that occurs in bluff bodies where vortices are shed alternating from one side of the object to the other, as shown in Figure 6.

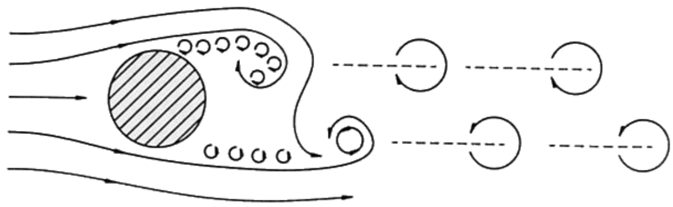


Figure 6: Vortex formation in the wake of a bluff object (Mendis et al., 2007)

Shedding of vortices produces asymmetric pressure distribution on the building surface, which results in periodic crosswind loading. If the structure is flexible, oscillation will occur transverse to the wind, and conditions for resonance would occur if the vortex shedding frequency coincides with the natural frequency of the structure. This is often referred to as the critical velocity effect.

The resonant response may be further exacerbated by the *lock-in* phenomena. Lock-in occurs when the sway of the building becomes sufficiently large such that it begins to drive the shedding vortices. As a result, the vortices adapt to the natural frequency of the building, causing the resonant response to be preserved over a wider range of approach wind speeds. This can give rise to very large oscillations and possible serviceability and ultimate limit failures.

A secondary mechanism that can contribute to crosswind response is the *incident turbulence mechanism*. In some cases, turbulence in the approaching wind can directly induce variations in crosswind forces. The ability of incident turbulence to produce significant contributions to crosswind response depends largely on the ability of longitudinal wind speed to generate crosswind forces on the structure. Sections with a high lift curve slope or a pitching moment curve slope, such as a streamlined bridge deck section or flat deck roof, are possibly susceptible to this effect, although this effect is not very significant when considering the overall crosswind response of tall buildings.

3 WIND PERFORMANCE CRITERIA

The interaction of wind with tall buildings gives rise to several challenges that must be considered in the wind design of tall buildings. These issues are addressed with the following three types of wind studies:

- ***Wind loads on structure*** – to determine wind loads for the design of the lateral load resisting structural system.
- ***Wind loads on cladding*** - to assess design wind pressures throughout the facade area of a tall building.
- ***Environmental wind studies*** - to assess the influence of a proposed tall building on the behaviour of wind in the surrounding environment. These include studies on pedestrian wind comfort and safety.

When assessing wind loads on the structure, the design should satisfy requirements for (1) *stability* of the overall structure against overturning or sliding; (2) *strength* of structural members to prevent their failure in bending and shear while effectively transferring the lateral wind load down to the foundation, and (3) *serviceability* criteria by controlling overall building deflection and inter-storey drift to limit damage and cracking of non-structural elements such as the facade and internal partitions and ceilings, and controlling sway accelerations to prevent occupant discomfort due to motion perception in the upper stories

A discussion of wind performance criteria relating to structural and cladding design is provided in the following sections, and further details are available in the design manual by Biswas and Peronto (2020).

3.1 *Wind drift design*

Wind drift refers to the lateral deflection of the building under wind loads and is commonly quantified in terms of the total building deflection or interstorey drift. The total building deflection is the simplest wind drift metric that describes the overall displacement of a single point on each floor. Drift limits based on this metric are specified as a ratio of top deflection to the building height, with typical limits ranging between H/600 and H/400. These limits are not based on specific performance criteria but are nonetheless useful for estimating the general building behaviour prior to performing a detailed analysis.

Interstorey drift is the most used deformation criterion in tall buildings. It is defined as the relative horizontal displacement between two adjacent floors in the building, and can also be expressed as a ratio of storey drift divided by floor height. Interstorey drift is an important metric in tall building design as it has implications on stability, serviceability, and strength (Biswas and Peronto, 2020). There are two main contributors to storey drift, the first is the racking drift component caused by shear forces, and the second is the chord drift component caused by flexure that results in relative rotation between floors. The sum of these two components gives the total interstorey drift. Depending on the building height and structural system, storey drifts will usually be dominated by either flexure or shear.

Because most building damage is caused by the shear drift component, it is useful to consider this effect in isolation. This can be achieved using the Drift Measurement Index (DMI) where the shear drift for a given storey is calculated by considering the combination of relative horizontal and vertical displacements of all four corners of the area of interest (Biswas and Peronto, 2020). Details of this method are presented by Griffis (1993)

The main reasons for adopting wind drift limits relate to serviceability criteria such as limiting damage to non-structural components such as facade elements and interior partitions, avoiding operational issues of vertical transport, controlling P-Δ or secondary loading effects, and reducing effects of motion perceptibility.

Drift limits for cladding and partitions should be specified in terms of serviceability wind speeds, and the limit should relate to the type of non-structural materials used and the methods of fixing. Due to a lack of information on the performance of partitions and cladding under racking loads, while a wide range of different systems is used in practice, it is difficult to establish a rational basis for specifying drift limits. Currently used limits appear to be based on judgement developed from satisfactory past performance, and deflection limits provided in structural design codes are given as recommendations rather than mandatory requirements. Drift criteria are left at the discretion of the structural engineer, and these must be discussed and agreed upon by relevant stakeholders during the design process. Table 2 lists suggested deflection limits in Appendix C of AS/NZS 1170.0:2002 (Standards Australia, 2002) for serviceability design under wind actions.

Table 2: Suggested deflection limits in AS1170.0:2002 for cladding and partitions elements under wind loading

Element	Phenomenon controlled	Serviceability parameter	Limit
Walls - general (face loading)	Discerned movement	Mid-height deflection	$H/150$
	Supported elements rattle	Mid-height deflection	$H/1000$
Brittle cladding (face loading)	Cracking	Mid-height deflection	$H/500$
Masonry walls (in plane)	Noticeable cracking	Deflection at top	$H/600$
Masonry walls (face loaded)	Noticeable cracking	Deflection at top	$H/400$
Plaster/gypsum walls (in plane)	Lining damage	Mid-height deflection	$H/300$
Plaster/gypsum walls (face loading)	Lining damage	Mid-height deflection	$H/200$
Glazing systems	Bowing	Mid-span deflection	$S/400$
Windows, facades, curtain walls	Façade damage	Mid-span deflection	$S/250$

H is the height of the wall or cladding unit. *S* is the span.

Drift limits may also be imposed for consideration of lift operation in tall buildings. Lifts cars are secured by vertical rails within the shafts of buildings and can therefore withstand considerable lateral slope. However, the swaying of lift cables under dynamic excitation can result in the cables snagging on fittings in the shaft. This is managed by parking the lifts during high wind events so that the cable length is reduced, and the natural frequency of the cable is separated from the natural frequency of the building. Lift considerations will vary from case to case depending on the geometry of the lift shaft and its deflected shape, but in general, if drift limits are relaxed then larger shaft sizes may be needed to provide adequate clearance between the cables and fittings in the shaft (Smith, 2011).

P-Δ effects are important in checking the strength and stability of a tall building under ultimate limit state wind speeds, and methods for calculating these secondary effects are well established. However, there seems to be no need to control them by arbitrarily setting drift limits. Motion perception and human comfort are also related to drift limits, but it is best to relate these criteria to lateral accelerations as described in the following section.

3.2 *Authors' Recommendation*

Unfortunately, code guidance on overall top deflections for tall buildings is quite limited. Based on the authors' vast experience of many tall building projects, it is recommended that the allowable top deflections of tall buildings be limited to values between H/500 to H/1000 for serviceability limit state, where a more stringent design would maintain an allowable top deflection closer to H/1000.

3.3 *Comfort criteria and motion perceptibility*

A critical design criterion for tall buildings is the control of sway accelerations under serviceability wind speeds. Some building motion under wind effects is expected but this needs to be contained within acceptable limits. The general objective should be to avoid perception of motion under everyday conditions (0.1-year RP), to keep occupants comfortable under frequent wind events (1-year RP), and to limit discomfort to manageable levels under infrequent events (10-year RP) (Biswas and Peronto, 2020).

Establishing fixed criteria that satisfy everyone is difficult because human response to motion is known to vary significantly from person to person. Several physiological and psychological parameters affect human perception of motion in the low-frequency range of 0-1 Hz encountered in tall buildings. These 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 (Kwok et al., 2015; Kwok et al., 2009). Table 3 lists typical human responses to various levels of building acceleration.

Table 3: Human perception levels to building motion (Yamada and Goto, 1975)

Level	Acceleration (m/s^2)	Effect
1	< 0.05	Humans cannot perceive motion
2	0.05 - 0.1	Sensitive people can perceive motion, and hanging objects may move slightly.
3	0.1 - 0.25	Majority of people will perceive motion. May affect desk work, and long-term exposure may produce motion sickness.
4	0.25 - 0.4	Desk work becomes difficult or almost impossible, but ambulation is still possible.
5	0.4 - 0.5	People strongly perceive motion and walking naturally is difficult. 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

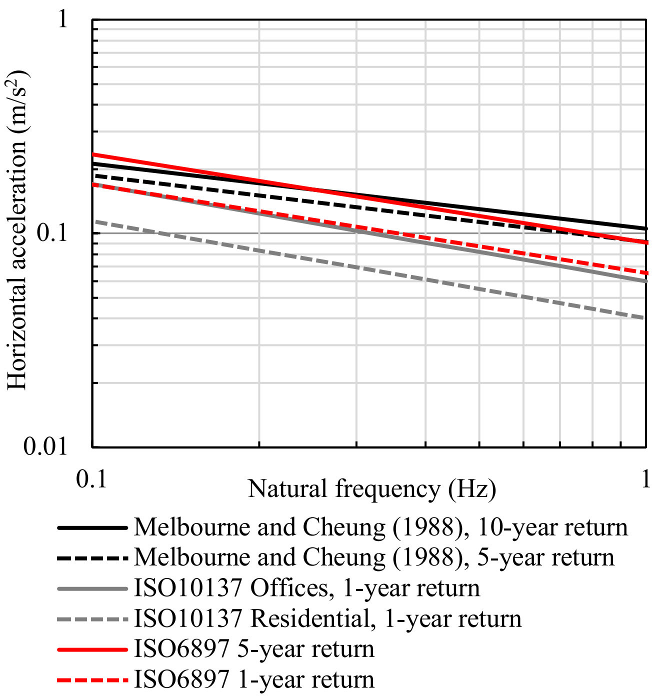


Figure 7: Horizontal acceleration limits for occupant comfort in buildings

Although motion perception levels in humans are generally well understood, there are no generally accepted standards for comfort criteria in tall buildings. Irwin (1978) proposed comfort criteria based on root mean squared (RMS) acceleration limits, which were later adopted in ISO6897 (International Organization for Standardization, 1984) guidelines for evaluating the response of occupants of fixed structures to low-frequency horizontal motion. Melbourne and Cheung (1988) converted these limits from RMS to peak accelerations, and these have been widely used in Australia. While early practice has mostly focused on limiting accelerations at 5-year and 10-year return periods, more recent standards such as the ISO10137 (International Organization for Standardization, 2007) specify limits for a lower return period of 1-year. A comparison of various acceleration guidelines is presented in Figure 7, where the vertical axis corresponds to the acceleration limit in m/s^2 and the horizontal axis is the natural frequency of the building in the direction of interest. A peak factor of 3.4 has been used to convert the ISO6897 limits from RMS to peak accelerations.

Due to the lack of uniformly accepted standards for comfort criteria in tall buildings, acceleration limits should be determined by the design team in conjunction with relevant stakeholders based upon agreed levels of performance. For further details on habitability requirements in tall buildings, the reader is referred to the text by Kwok et al. (2015).

3.4 *Wind loads on cladding*

Surface pressures induced by wind are unsteady and highly fluctuating, partly due to the gustiness of approaching wind, and partly because of dynamic flow behaviours such as buffeting and local shedding of vortices at the edges of the structure. Spatially, the pressures are not uniformly distributed but vary with position over the surface of the structure.

Due to the significant cost of facades in proportion to the overall cost of tall buildings, engineers cannot afford to be unnecessarily conservative when estimating design wind loads for facades. Wind design codes are unable to provide accurate predictions of cladding pressures in tall buildings because of the dynamic characteristics of wind and the complexity of modern tall building shapes. For this reason, wind tunnel testing is now a standard industry practice that is carried out to minimise initial capital costs associated with overdesign while avoiding expensive operating and maintenance costs associated with facade failure.

Wind tunnel studies have shown that the effects and factors producing wind loading design criteria for facades can vary significantly from those of the primary structure, even though both are subject to the same wind environment. This critical difference is directly related to the behavioural response characteristics of each system. The primary structure feels little of the specific effects of localised peak pressures such as those that may occur at building corners, setbacks, parapets, and other geometric complexities. The rigidity and degree of structural redundancy of a facade is usually low compared to the superstructure to which it is connected. Therefore, facades can be significantly impacted by localised peak wind loads.

With the global trend towards more sustainable buildings, it is also an added design criterion to maintain increased differential pressures between the interior and exterior environments. It should be noted that although the design wind loads for the primary structure generally decrease at lower elevations, they may not decrease nearly as significantly for the facade because of ground turbulence effects, "downwash" effects, and significant changes to the facade and building geometry towards the lower levels of the building.

Facades of tall buildings, which through wind tunnel studies are found to exhibit significant dynamic acceleration characteristics, should be even more carefully designed. Increases to the inherent or induced damping of the primary structure have been required in some buildings not only to modify the structure's dynamic behaviour for human perception of motion but also to achieve a satisfactory design of its facade.

4 DESIGN MEASURES FOR MITIGATING WIND-INDUCED BUILDING MOTION

There are two main approaches for mitigating wind-induced motion in tall buildings: modifying the aerodynamic performance of the building and modifying the structural properties of the building.

Aerodynamic performance is modified by altering the external geometry of the buildings. A common approach is to vary the building's cross-section with height to prevent vortex shedding from synchronising along the height of the building. This is achieved by tapering, twisting, or adding setbacks. Major geometric modifications are typically only feasible if considered during the concept design stages. Minor alterations, such as chamfering, rounding, recessing corners, or removing façade elements (Marsland et al., 2022) can be implemented later in the project, although these are generally less effective.

Alternatively, wind effects may be reduced by modifying the dynamic properties of the structure to shift the natural frequency of the structure away from aerodynamic loading frequencies, particularly those due to crosswind vortex shedding. Dynamic response due to wind can be improved by increasing the stiffness and mass of the building, but this often conflicts with requirements for earthquake design where lower mass and stiffness are desirable. Increasing the damping of the building can reduce both wind and earthquake responses simultaneously

4.1 *Damping systems*

Damping systems are often the only practical and economical means of controlling wind-induced vibrations in tall buildings. There are three main categories of damping systems: passive, active, and semi-active systems. Active and semi-active systems require an external power source to operate, while passive systems rely on the natural movement of the structure.

Although general design philosophy tends to favour passive damping systems due to their lower capital and operational costs, active or semi-active dampers may be the ideal solution for certain complex vibration problems. Examples of damping systems are listed in Table 4, and a full discussion of the theory and design of damping technologies in buildings can be found in texts by Lago et al. (2019) and Soong and Costantinou (1994).

Table 4: Examples of damping systems in buildings

Category	**Examples**
Passive damping systems	Tuned mass dampers Tuned liquid dampers Tuned liquid column dampers Friction dampers Viscous dampers Viscoelastic dampers Impact type dampers
Active damping systems	Active tuned mass dampers Active mass drivers

Semi-active damping systems	Variable stiffness dampers Variable friction dampers Hydraulic dampers Controllable fluid dampers Magneto-rheological dampers Electro-rheological dampers

5 WIND DESIGN TOOLS: ANALYTICAL METHODS AND WIND DESING CODES

Wind design tools are used to obtain estimates of wind loading on a given building. These tools can be divided into three main categories: analytical methods, experimental methods, and computational methods.

Wind design codes offer detailed analytical methods for predicting wind loads on structures. These analytical methods are subdivided into static analysis and dynamic analysis. The static approach is based on a quasi-steady assumption that the building is a fixed rigid body in the wind. This approach uses a single-value equivalent static wind pressure to represent the maximum pressure the structure would experience. Design wind pressures on building surfaces P are approximated by the product of the gust dynamic wind pressure q and the mean pressure coefficients C_P:

$$P = q \times C_P = [0.5\rho_{air}V^2] \times C_P \tag{5.1}$$

Mean pressure coefficients are provided in wind codes for simple geometries or may be measured in wind tunnels or by full-scale tests. The implied assumption is that the pressures on the building surface (external and internal) closely follow 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 is reliable for low to mid-rise structures but is not adequate for tall, slender, or vibration-prone buildings.

For vibration-sensitive structures, wind design standards require the use of dynamic analysis methods that consider the effects of resonance, acceleration, damping, and structural dynamics. Many wind codes require that certain structures must be analysed using dynamic methods, including buildings with certain aspect ratios (e.g. height to breadth ratio greater than 5) or natural frequencies less than 1 Hertz. The AS/NZS 1170.2:2021 standard (Standards Australia, 2021) dedicates an entire chapter to evaluating dynamic response in tall buildings, where dynamic effects are accounted for by multiplying static load estimates with a dynamic response factor C_{dyn}. The standard also provides guidance for estimating peak floor accelerations in the along-wind and cross-wind directions in Appendix E.

Nonetheless, it should be noted that wind codes have limitations when analysing buildings with extreme heights, complicated geometries, and complex terrain and shielding structures surrounding them. For instance, the AS/NZS 1170.2:2021 code does not apply to buildings taller than 200 m or with a natural frequency less than 0.2 Hz. For the design of taller buildings or structures with irregular geometry, most major standards recommend the use of wind tunnel testing.

6 WIND TUNNEL TESTING

In the many situations described previously where analytical methods cannot be used, more accurate estimates of wind effects on tall buildings can be obtained through model testing in a boundary layer wind tunnel. Wind tunnel testing is now common practice for the design of most tall buildings. Owners of proposed tall buildings are encouraged to allow for wind tunnel testing, as the costs associated with such testing can be offset by the substantial savings in the structural costs from often reduced wind loads. The Australian wind code allows wind tunnel testing as a suitable alternative to the code recommendations to determine design wind loads for any structure. To regulate the highly specialised area of wind tunnel

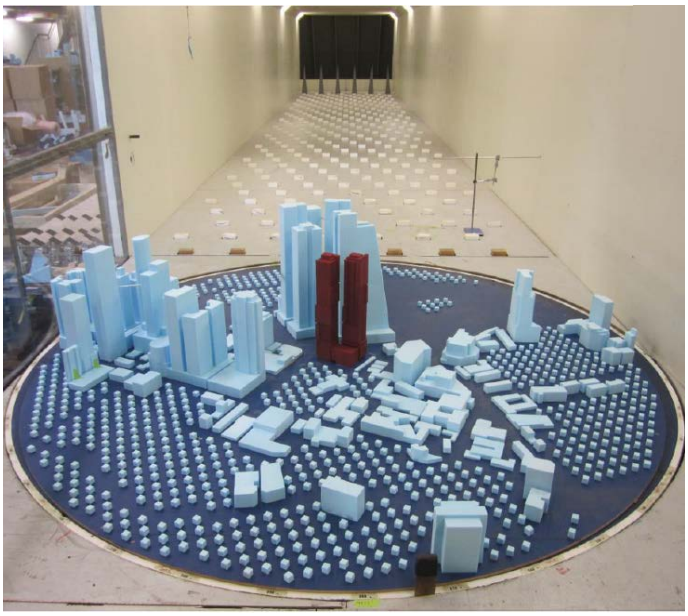


Figure 8: Test section in boundary layer wind tunnel with building models placed on a turntable

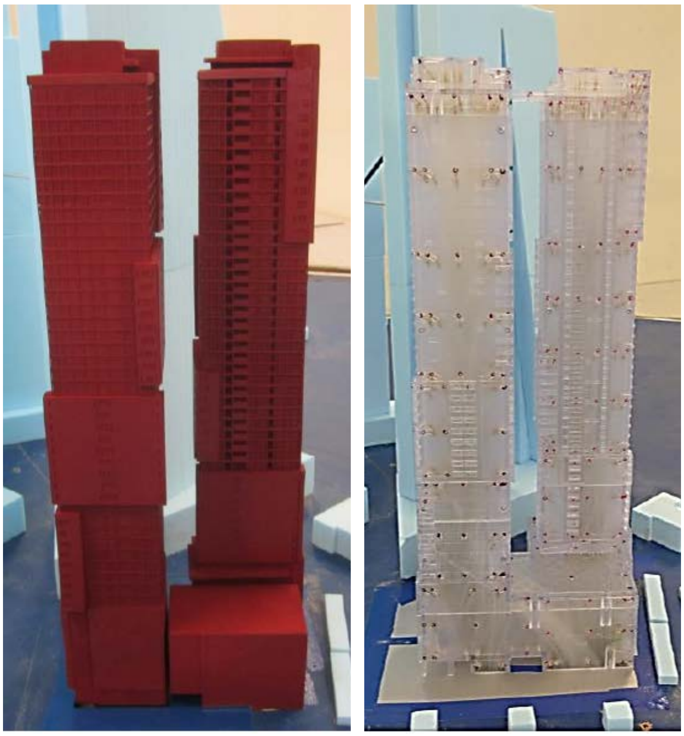


Figure 9: HFFB model (left) and pressure tap model (right) of Capitol TwinPeaks Towers, Colombo

testing in Australia, a quality assurance manual has been developed for wind engineering studies of buildings (AWES, 2019).

Wind tunnel testing involves blowing air on a scaled physical model of the building and its surroundings at various angles relative to the building orientation representing the wind directions. This is typically achieved by placing the scaled physical model on a turntable within the test section (Figure 8). Once testing is completed for a selected direction, the turntable is rotated by a chosen angular increment to represent a new wind direction.

6.1 *Types of wind tunnel tests for tall buildings*

Wind tunnel tests currently being conducted for determining overall structural loads and responses of tall buildings can be divided into two main categories. The first category involves the use of rigid building models (Figure 9), which includes the high-frequency force balance (HFFB) method and the high-frequency pressure integration (HFPI) method. The second category involves the use of flexible (aeroelastic) models.

6.1.1 *High-frequency force balance (HFFB)*

In HFFB testing, a rigid physical model of the building is mounted to a high-frequency force balance at its base. The balance is usually a strain gauge that can measure base reactions at a very high sampling rate. The building model needs to be extremely rigid so that the effect of wind loads on the building surface is fully accounted for in base reaction measurements. For very tall and slender models, carbon fibre rods may be added to the centre of the model to increase rigidity.

The model is tested in the wind tunnel for various wind speeds and approach angles. Time histories of base reactions are recorded which can be used to calculate peak structural loads as well as dynamic building responses such as peak accelerations and deflections. HFFB models are considerably easier and cheaper to prepare compared to other testing methods. The main drawback of this method is that it does not provide measurements of surface pressures which are needed for designing façade elements and other exposed components on the building envelope.

6.1.2 *High-frequency pressure integration (HFPI)*

In HFPI, a rigid physical model of the building is fitted with pressure taps distributed around the external surface area of the building. A minimum of 1 pressure tap per 120m^2 of building surface area is recommended (AWES, 2019). This process can be very time consuming depending on the size and geometric complexity of the building model. The pressure taps are connected to a high-frequency electronic pressure measuring system to capture instantaneous pressure readings at all locations simultaneously. The resulting time histories of pressure measurements can be used to estimate peak pressures needed for the design of facade elements. Furthermore, since instantaneous pressure measurements are synchronised, they can be integrated over the building surface to obtain time histories of base reactions. These can then be analysed in a similar approach to the results of the HFFB method to obtain peak base reactions and building responses.

6.1.3 *Aeroelastic testing*

A more advanced type of wind tunnel testing involves the use of aeroelastic models. The main objective of an aeroelastic study is to obtain more accurate predictions of wind loads by properly modelling the physical behaviour of both the wind and the structure in the wind tunnel. Aeroelastic testing takes the guesswork out of gust factor computations since it allows for direct measurement of dynamic building responses in the wind tunnel. It can also account for the effects of displacement-dependant excitations such as galloping, flutter, and lock-in.

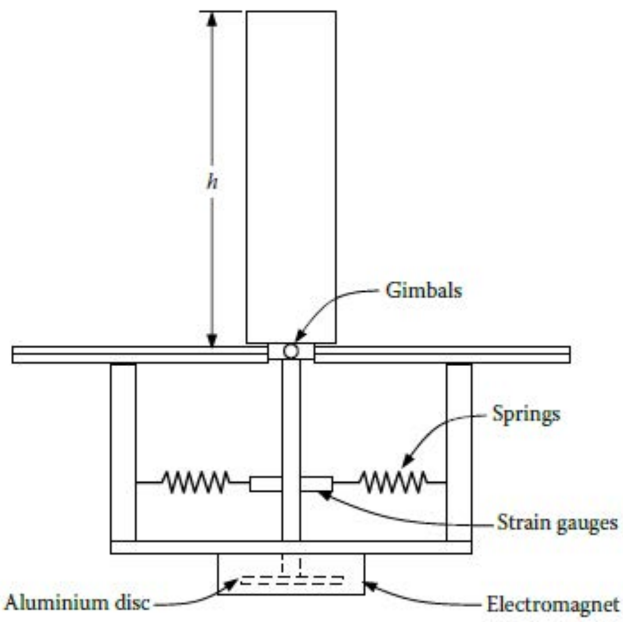


Figure 10: Components of base-pivoted aeroelastic building model (Holmes, 2015).

Aeroelastic testing is however significantly costlier than HFFB and HFPI because it requires scaling down of structural properties including the stiffness, mass distribution, and damping of the building. Aeroelastic tests are generally only considered after rigid model tests have been performed and the potential for negative aerodynamic damping has been identified (Irwin et al., 2013).

For very flexible and slender buildings, several spring pivots may be provided along the height of the building model to account for higher modes of vibration in several degrees of freedom. For stiffer buildings, where higher modes of vibration may be neglected, the aeroelastic models may be simplified by reducing the number of pivots. The simplest and cheapest aeroelastic models are base-pivoted models (Figure 10). These only consider the first mode of vibration and approximate the building's deflected shape with a straight line.

For base-pivoted models, it is generally not necessary to achieve a correct mass density distribution along the building height if the mass moment of inertia about the pivot point is equivalent to the model's density distribution. The pivot point is typically chosen to obtain a mode shape that provides the best agreement with the fundamental mode shapes of the model. For buildings with complex structures, computer software may be used to determine the fundamental model shapes of the building.

6.2 *Achieving similitude in wind tunnel tests*

To accurately estimate wind forces on the structure, the atmospheric boundary layer must be carefully replicated in the wind tunnel. Properties of the approach flow that are of particular importance are the mean velocity profile $\bar{V}(z)$, longitudinal turbulence intensity profile $I(z)$, and the power spectral density of velocity fluctuations $S_V(n)$. In boundary layer tunnels, large-scale turbulence can be generated by placing trip boards and spires upstream of the fetch length, while the required velocity and turbulence profiles may be generated by placing carpet or roughness blocks along the fetch length (Figure 8).

It is also important to maintain dynamic similarity between model and full-scale results. This can be achieved by ensuring that the following non-dimensional parameters are kept as near constant as possible between the natural wind and the wind tunnel:

- the velocity profile $\bar{V}(z)/\bar{V}(z_0)$ which is the variation of velocity with height normalised with respect to the velocity at height z_0,
- the turbulence intensity $I(z)$=$\sigma_V/\bar{V}$; and
- the normalised power spectral density, $nS_V(n)/\sigma_V^2$, which defines the energy present in the turbulence at various frequencies n.

The geometric scale of the wind profile must correspond to the geometric scale of the physical model of the building. A geometric scale of 1:400 is typically used to limit the blockage ratio in the wind tunnel to less than 10%. For larger tunnels, the use of 1:200 or 1:100 scale models may be possible.

To relate wind tunnel pressure measurements to full-scale values, length and time scales must be determined. If, say, the length scale λ_L is 1:400 and the velocity scale λ_V (ratio of wind speed in the tunnel to full-scale wind) is 1:3, the resulting time scale $\lambda_T = \lambda_L/\lambda_V$ would be approximately 1:133. This means that every second recorded in the wind tunnel corresponds to 133 seconds at full scale. Because the time scale is the inverse of the frequency scale, the scaled model would have a natural frequency equivalent to 133 times that of the full-scale building.

6.3 *Effect of Reynolds number*

Reynolds number is a non-dimensional parameter in fluid flow that describes the ratio of inertial forces to viscous forces in a fluid. Essentially, this parameter describes the flow regime which affects the magnitude and distribution of wind loads acting on an object.

Achieving Reynolds number similarity for buildings with sharp edges is generally not required since the flow around such buildings is dictated by geometrically induced separation at the building edges. However, for buildings with smooth surfaces where separation is induced by the curvature of the building surface, the flow pattern is sensitive to the Reynolds number, so it becomes important to account for Reynolds effects.

Achieving similarity of Reynolds number in the wind tunnel is not feasible due to the significant difference in geometric scale between the building model and the real structure. Nonetheless, Reynolds effects can be accounted for by roughening the model surface so that the flow behaviour mimics the flow expected at a higher Reynolds number. This approach was adopted in wind tunnel testing of the Colombo Lotus Tower in Sri Lanka which has a smooth cylindrical base (Figure 11). The surface roughness of the model was varied incrementally using adjustable roughness ribs until the flow pattern matched that expected in the real structure.

Figure 11: Wind tunnel testing of Colombo Lotus Tower

6.4 *Influence of surrounding buildings*

Another factor that needs to be considered in wind tunnel studies is the influence of surrounding buildings. Buildings of similar size located near the proposed building can cause large increases in cross-wind responses. Tests should not only consider the existing conditions but make allowances for future changes in the surrounding area during the design life of the structure. These can be easily incorporated into wind tunnel tests with a relatively minor increase in costs. The Australian quality assurance manual (AWES, 2019) requires that surrounding buildings within a 300 m radius from the building site should be modelled to the correct scale in the wind tunnel, although the required geometric accuracy for these buildings is low (within 10%).

7 COMPUTATIONAL FLUID DYNAMICS

Computational fluid dynamics (CFD) is a numerical tool for simulating fluid flow that involves solving the fundamental equations that govern the behaviour of fluids known as the Navier-Stokes equations. The use of CFD for building studies has traditionally been restricted due to the high computational cost of simulating turbulent flow in urban environments, but with considerable advancement in computing resources and simulation tools over the past decade, CFD is nowadays becoming increasingly popular for analysing wind effects on buildings. CFD models are highly versatile and can be used for a wide range of wind engineering studies including wind load predictions on the structure and cladding, environmental studies such as pedestrian wind comfort, simulation of wind-driven rain exposure (Abu-Zidan et al., 2021b), and wind effects on façade fire spread (Abu-Zidan et al., 2022).

CFD offers unique advantages over wind-tunnel testing, including lower cost and time requirements and ease of design parametrisation. CFD models can account for complex architectural forms and are not restricted by similitude requirements. Moreover, unlike wind tunnel experiments where results are only available at instrumented locations, CFD results are available at all points in the computational domain, making CFD a powerful tool for visualising flow structures and performing qualitative analysis during early design stages.

Despite these advantages, confidence in the reliability and accuracy of CFD predictions remains a major challenge. CFD results are highly sensitive to a wide range of simulation parameters that must be set by the user, including turbulence model parameters, boundary conditions, computational grids, and discretisation schemes. This introduces a significant risk of degraded model performance due to error or uncertainty in the input data or model parameter set by the user (Blocken, 2015)

Confidence in numerical predictions can be established through the verification and validation (V&V) framework (AIAA, 1998; Oberkampf and Trucano, 2002). *Verification* assesses whether the numerical model has been correctly implemented per the modeller's conceptual description of the problem, while *validation* determines the degree to which the numerical model can replicate physical reality. V&V guidelines emphasise the importance of a two-step approach where verification is performed prior to, and independent of validation.

Although V&V guidelines are available for wind engineering applications, these mostly focus on environmental studies such as pedestrian wind comfort and pollutant dispersion [e.g. Franke (2010), Blocken and Gualtieri (2012), and Blocken (2015)]. Recently, Abu-Zidan (2019) demonstrated the application of V&V principles for CFD simulations of structural wind loading on tall buildings.

7.1 *Components of a CFD model of a tall building*

The main components of a typical CFD model for building studies are presented in Figure 12. The model consists of an external computational domain, typically of cubic shape, that contains the computational cells or elements in which the fluid equations are solved, and flow is simulated. Within the domain lies the building of interest. This may be an isolated building or may include surrounding structures. Often the geometric complexity of the building is simplified to reduce meshing requirements

The internal volume of the computational domain is spatially discretised into a finite number of computational cells through the process of meshing. A finer mesh with a larger number of computational elements provides higher solution fidelity but increases the computational cost. A common practice to optimise the cell count is to selectively refine the mesh at specific regions in the domain. At the surface of the building and within its immediate vicinity, a high level of refinement is provided to capture critical flow features such as flow separation and recirculation. In the regions upstream and downstream of the building, moderate refinement is specified to maintain inflow turbulence and building-generated turbulence, while elsewhere in the domain low refinement is used to reduce the computational cost.

The boundaries of the computational domain are designated specific boundary conditions as shown in Figure 12. The atmospheric boundary layer profile is specified at the inlet boundary. This profile must be maintained as it travels through the upstream fetch to the location of the building to avoid errors in the simulation (Abu-Zidan et al., 2020). The domain boundaries should be placed as close to the building model as possible without interfering with the results. Guidance on sizing the computational domain for tall

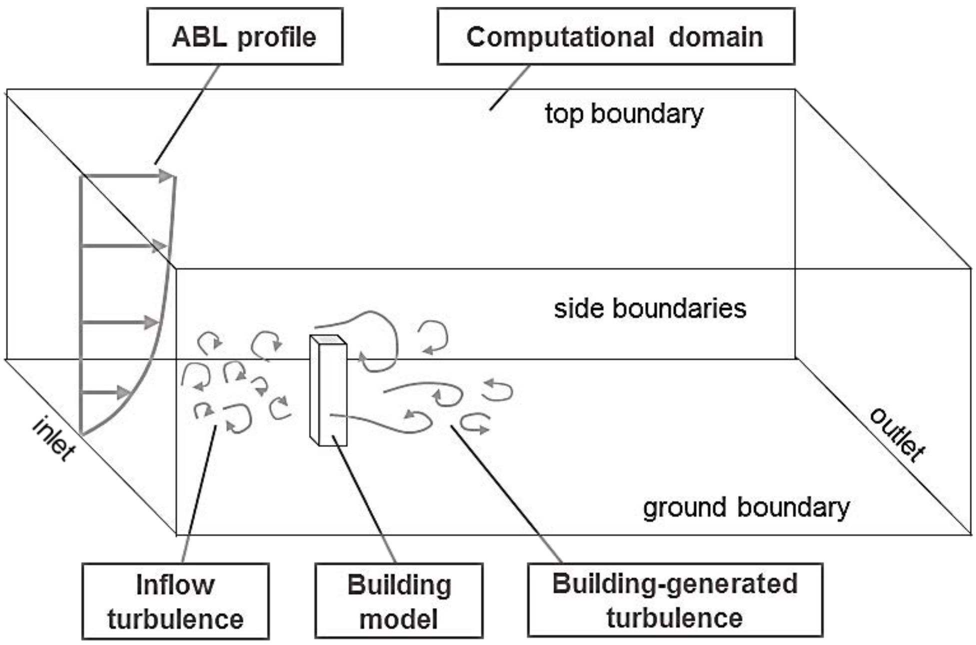


Figure 12: Components of a CFD model for simulating wind effects on buildings (Abu-Zidan, 2019)

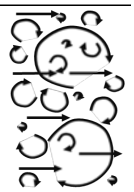
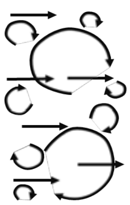
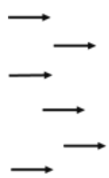
building simulations can be found in Abu-Zidan et al. (2021a).

7.2 *Types of CFD simulations*

The various types of CFD simulations are listed in Table 5 and can be distributed on a spectrum of accuracy vs computational cost. At the high end of accuracy are Direct Numerical Simulations (DNS), where turbulence at all scales is resolved in time and space, from the largest eddies down to the viscosity scale. This requires extremely fine spatial and temporal discretisation which comes at an extremely large computational cost, making DNS impracticable for industrial-scale problems.

At the lower end of accuracy are Reynolds-averaged Navier-Stokes (RANS) models where, rather than resolving turbulence directly, turbulence models are used to estimate the effect of turbulence on the mean flow field. RANS models may be simplified further by ignoring temporal terms in the Navier-Stokes equations, leading to steady-state simulations. These are highly computationally efficient but are inherently incapable of predicting dynamic wind effects which are critical in the structural design of tall buildings. Nevertheless, steady RANS is suitable for other applications such as environmental wind studies including pedestrian wind comfort and wind-driven rain exposure.

Table 5: Types of CFD simulations

Direct numerical simulation (DNS)	All turbulence is resolved down to viscosity Very high computational cost, not feasible for industrial-scale problems	
Large eddy simulation (LES)	Large eddies are resolved while small eddies are modelled High computational cost, but feasible for building applications Directly simulates dynamic wind behaviour such as vortex shedding and buffeting.	
Reynolds Averaged Navier Stokes (RANS)	Turbulence is modelled at all scales Low computational cost Cannot simulate dynamic wind behaviour.	

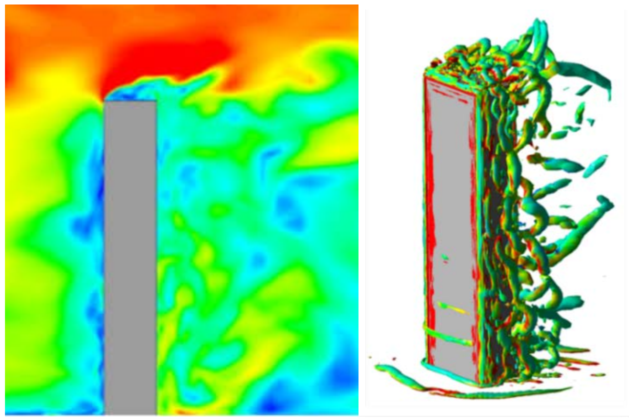


Figure 13: LES results of instantaneous velocity field around building (left) and building-generated turbulence (right)

Large Eddy Simulations (LES) provide a middle ground between the computational efficiency of RANS and the accuracy of DNS. LES adopts a filtering approach where larger eddies are resolved while smaller eddies are modelled using sub-grid scale (SGS) models. The size of the computational cells will dictate the smallest scale of resolved turbulence, and for LES models, it is recommended that at least 80% of kinetic turbulence energy in the simulation is resolved (Pope, 2000). LES models can simulate dynamic wind and are therefore suitable for predicting wind loads on tall buildings. Figure 13 presents LES results for wind flow around a tall building showing explicit modelling of atmospheric and building-generated turbulence.

7.3 *Fluid-structure interaction*

To reduce the computational cost, CFD simulations are typically performed with a rigid building model that does experience deformation under the effects of wind. More complex simulations may be performed by coupling a transient CFD model such as LES with a Finite element (FE) model of the building structure. This is commonly referred to as a *Fluid-structure interaction (FSI)* model which can involve either one-way or two-way coupling. One-way FSI involves a one-way transfer of results from the CFD model to the FE model where time histories of CFD- generated wind pressures are fed as an input into the FE solver to predict the dynamic response of the structure. The effect of building displacement on the wind flow field is however not considered in one-way FSI.

In two-way FSI, results of structural displacements from the FE model are fed back into the CFD solver at regular time intervals during the simulation to account for the effect of building motion on the flow field. This approach is akin to aeroelastic testing in wind tunnels and allows for predicting higher-order effects such as lock-in and flutter. The computational cost of two-way FSI is considerably greater than rigid CFD models and one-way FSI.

Although research on FSI modelling for tall buildings remains very limited, a recent study suggests that one-way FSI can produce accurate results for a super-slender structure compared to wind tunnel testing (Wijesooriya et al., 2021). Two-way coupling may prove pertinent when considering wind-induced responses of structural elements prone to very large deflections where secondary effects are magnified, such as in the structural design of tower masts against fatigue failure (Mendis et al., 2018).

8 CASE STUDY: CAPITOL TWINPEAKS TOWERS, COLOMBO

The use of CFD modelling for tall building design has traditionally been limited in industry projects due to the high level of complexity involved and the limited availability of computing resources. However, with recent advancements in computing technologies including multicore processors, cloud computing services, and open-source CFD software, CFD modelling for industry scale projects is now becoming increasingly feasible.

Here we present a case study of CFD modelling for the Capitol TwinPeaks Towers project in Colombo, Sri Lanka. The project features a complex geometry of two fifty-storey towers (182 meters tall) connected at a podium structure eight levels above the ground (Figure 15). The project is also surrounded by high-rise developments that will influence wind flow and should therefore be included in the analysis. With sponsoring from Sanken Constructions (Pvt) Limited, wind tunnel testing was performed using both HFFB and pressure tap models (Figure 9) to determine wind loads on the structure and façade. The test included surrounding structures within a 500 m radius of the building as shown in Figure 8.

An equivalent numerical model was developed at the University of Melbourne with support from CSEC (Civil & Structural Engineering Consultants Private Limited) using open-source CFD software OpenFOAM v7 (Figure 14). The model features a high level of geometric detail and also includes surrounding structures within a 500 m radius. This required a large computational grid with 6.5 million elements. The pimpleFOAM solver was used with second-order discretisation schemes and the realizable k-ε model was used to model turbulence.

To demonstrate practical feasibility, the model was solved on a relatively low-cost single-node machine with two 14-core Intel(R) Xeon(R) E5-2697 v3 processors at 2.6 GHz and a total RAM of 64 GB (considerably larger computing resources can nowadays be accessed via high-performance computing (HPC) facilities or on cloud computing services). The model was solved until all residuals dropped below 10^{-4} to achieve convergence, and the solution time was < 3 hrs. The resulting surface pressure contours on the windward and leeward surfaces of the building are shown in Figure 15.

To validate the CFD model, surface pressure predictions were compared with wind tunnel measurements in Figure 16 at 855 locations on the building surfaces including the walls, roof, and soffit.

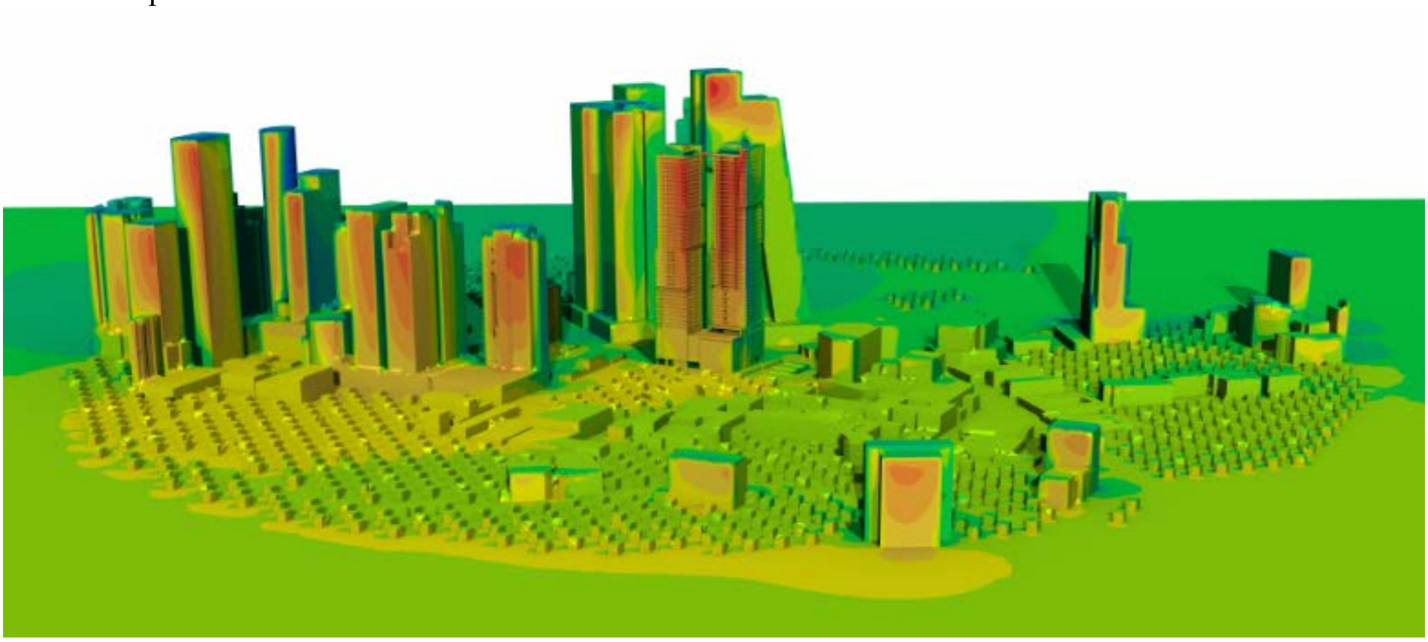


Figure 14: CFD model of Capitol TwinPeaks Towers with surrounding buildings within a 500 m radius

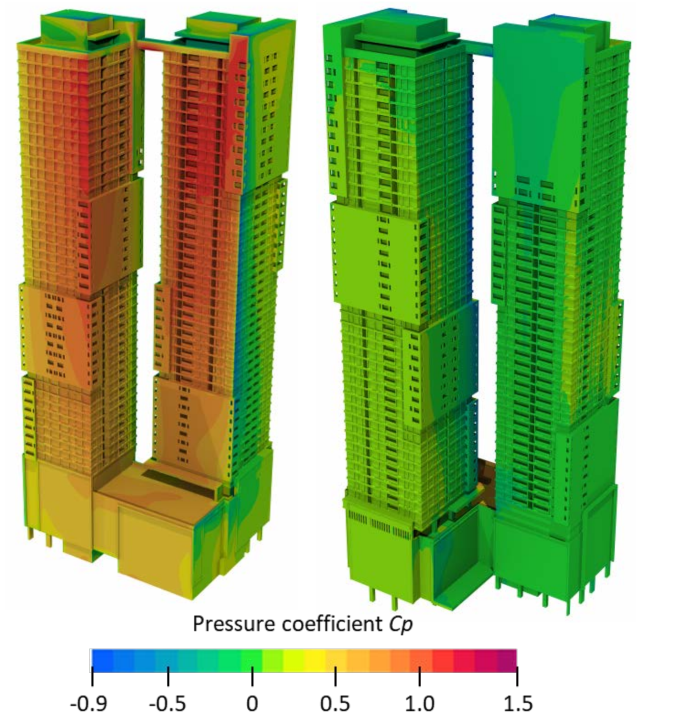


Figure 15: Surface pressure contours on the windward side (left) and leeward side (right) of Capitol TwinPeaks Towers

The comparison shows a strong correlation with an R-squared value of 0.97, despite a slight overprediction of CFD pressures which is a known limitation of the k-ε model used in this study.

Overall, the case study demonstrated the practical feasibility of CFD modelling for industry projects of tall buildings. The accuracy of CFD predictions can be improved by using LES models which come at the expense of a larger computational demand. Ongoing research is being conducted at the University of Melbourne to improve the efficiency of LES models for tall building applications.

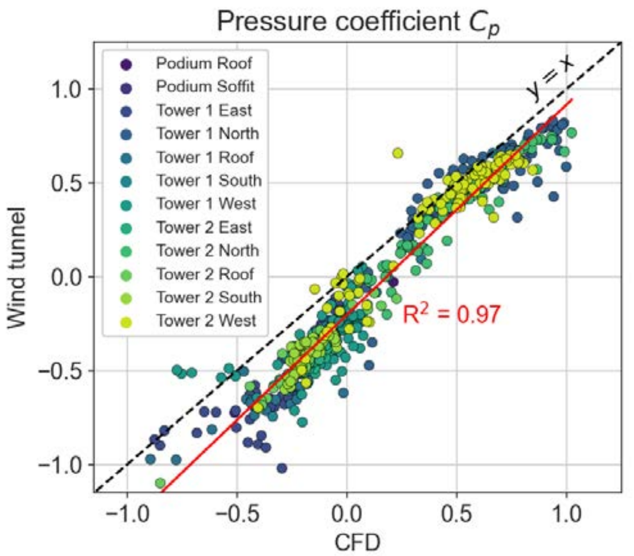


Figure 16: Comparison of CFD and wind tunnel results of surface pressures at 855 locations on Capitol TwinPeaks Towers

9 CONCLUDING REMARKS

This paper has considered several factors associated with the wind design of tall buildings. Design requirements for structural strength, stability, and serviceability, assume particular importance because tall buildings are susceptible to significant dynamic responses under wind-excitation mechanisms such as buffeting and vortex shedding. Tall building design is often governed by serviceability criteria for occupant comfort and perception of building motion, requiring the introduction of damping systems to reduce wind-induced vibrations to acceptable levels. Other design considerations include ensuring adequate facade performance under localised wind pressures and limiting wind drift which can lead to structural and serviceability issues.

Accurate estimation of wind effects requires considering the effects of the local climate and terrain conditions, as well as the geometrical and dynamic properties of the structure. Analytical methods provided in wind design codes and standards are largely inapplicable in the case of modern tall buildings which tend to be very slender with highly irregular geometries. Wind tunnel testing is recognised as being particularly useful for determining wind forces and dynamic responses in tall buildings, all while considering the effects of wind directionality, terrain and topological features, and nearby structures. The emerging use of CFD is noted as gaining importance in the design of tall buildings, particularly at the concept design stage. The importance of careful verification and validation of numerical results is highlighted as a vital prerequisite for adopting CFD in the design of tall buildings.

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