Prediction of Wind Loads on Tall Buildings: Development and Applications of an Aerodynamic Database

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The Western University Aerodynamic Database (WAD) has been developed as an alternative means for structural engineers to estimate the preliminary design wind loads on tall buildings. The database consists of aerodynamic loads obtained from either the force-balance or pressure model tests on 56 tall buildings in their simulated actual surroundings carried out in the Boundary Layer Wind Tunnel Laboratory at Western University. The data for a given building include the statistics of the normalized aerodynamic loads such as the means, root-mean squares, and power spectral density functions of the base bending moments in two orthogonal directions and base torque. To estimate the preliminary design wind loads on a target building, the fuzzy logic theory is employed to select the reference buildings from the database whose aerodynamic characteristics and upstream conditions are similar to those of the target building. A modified three-dimensional moment gust loading factor approach is proposed to estimate the wind-induced responses of the target building for all wind directions. The WAD-based procedure for estimating the wind-induced responses is validated by comparing the estimated responses with the corresponding responses obtained from the force-balance or pressure model tests for 36 tall buildings included in WAD. The comparison suggests that the WAD-based procedure can provide reasonably accurate estimates of base moments and accelerations of tall buildings, and is therefore considered adequate to be used in their preliminary design. Finally, the wind-induced responses predicted using the WAD-based procedures are also compared with those obtained from the wind load provisions in three major design codes, i.e. ASCE 7-10, NBCC 2010 and AS/NZS 1170.2: 2011, as well as the NatHaz Aerodynamic Load Database developed at University of Notre Dame. The results of the comparison study show that the WAD-based predictions of the wind loads is a viable alternative to evaluating the preliminary design wind loads for tall buildings.

Wind Loading, Tall Buildings, Design Codes and Standards, Force-balance Method, Pressure Integration Method, Aerodynamic Database, Gust Loading Factor, Fuzzy Logic Theory
This thesis was carried out in the Civil and Environmental Engineering Department of Western University. Foremost, I would like to express my gratitude to my co-supervisors Dr. Wenxing Zhou and Dr. Eric Ho for their guidance and support throughout the completion of this thesis. Great appreciation goes to Western University and Dr. Wenxing Zhou for the financial assistance. I would also like to thank Dr. Gregory Kopp, Dr. Hanping Hong, and Dr. Kamran Siddiqui for being my examiners and offering constructive suggestions to the thesis.

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\( \hat{a}_{AC} \): peak across-wind acceleration
\( \hat{a}_{AL} \): peak along-wind acceleration
\( \hat{a}_j (z, \alpha) \): peak acceleration along \( j \)-axis, at height \( z \), and wind angle \( \alpha \)
\( \hat{a}^W_j (H, \alpha) \): peak acceleration at \( H \) determined from wind tunnel analysis along \( j \)-axis and wind angle \( \alpha \)
\( \hat{a}^W (H, \alpha) \): peak resultant acceleration at \( H \) and wind angle \( \alpha \) determined from wind tunnel analysis
\( \hat{a}^* (H, \alpha) \): peak resultant acceleration at \( H \) and wind angle \( \alpha \) determined from WAD/design code/NALD
\( \mathcal{B} \): background turbulence factor
\( B \): longer plan dimension of the building
\( B_i \): longer plan dimension at height \( i \)
\( b \): plan dimension of the building that is perpendicular to the direction of wind; across-wind body width
\( C_{eH} \): terrain factor at \( H \) of the building in NBCC 2010
\( C_{fs} \): across-wind force spectrum coefficient generalized for a linear mode shape
\( C_{fx} \): total drag force coefficient
\( \bar{C}_{M_j} (\alpha) \): non-dimensional mean base moment coefficient about \( j \)-axis and wind angle \( \alpha \)
\( C_j \): linearization factor for the dynamic components for ESWL for dynamic moments about \( j \)-axis
\( C_{P,LW} \): effective pressure coefficient at leeward wall for design code calculations
\( C_{P,WW} \): effective pressure coefficient at windward wall for design code calculations
\( D_i (r_i) \): fuzzy value for the aspect ratio \( (A) \) or slenderness ratio \( (S) \) and \( \bullet \) is a generic symbol for \( A \) or \( S \)
\( D_i \): shorter plan dimension at height \( i \)
\( D \): shorter plan dimension of the building
\( E \): gust energy ratio
\( e_R \): distance between the elastic shear center and the center of mass
\( e \): eccentricity
F(ξ): fuzzy value of final matching index

\( f_i \): fundamental natural frequency, \( i = \text{along-wind, across-wind, or torque} \)

\( f_r \): reduced frequency

\( f_i \): natural frequency of the fundamental along-wind mode of the building for design code calculations

\( f_2 \): natural frequency of the fundamental across-wind mode of the building for design code calculations

\( f_j \): natural frequency of the building at mode dominant along \( j \)-axis

\( f_n(\alpha) \): reduced frequency of the building at mode dominant along \( j \)-axis and at wind angle \( \alpha \)

\( f(n) \): frequency

\( \tilde{G}_i \): mean component of the three-dimensional moment gust loading factor, \( i = \text{along-wind, across-wind, or torque} \)

\( G_{Bi} \): background component of the three-dimensional moment gust loading factor, \( i = \text{along-wind, across-wind, or torque} \)

\( G_q \): gust factor of wind velocity pressure

\( G_{Ri} \): resonant component of the three-dimensional moment gust loading factor, \( i = \text{along-wind, across-wind, or torque} \)

\( G_i \): three-dimensional moment gust loading factor, \( i = \text{along-wind, across-wind, or torque} \)

\( G \): gust loading factor

\( g_B \): background peak factor

\( g_R \): resonant peak factor

\( H_s \): mode correction factor used for resonant response factor in AS/NZS 1170.2:2011

\( H_i \): height of increment \( i \)

\( H \): total height of structure

\( I_H \): turbulence intensity at \( H \)

\( I(z) \): mass moment of inertia per unit height

\( \mathcal{K} \): factor related to the surface roughness coefficient of the terrain in NBCC 2010

\( K^3 \): generalized stiffness

\( K_m \): mode shape correction for the across-wind acceleration for design code calculations

\( K \): along-wind mode shape correction factor

\( L \): plan dimension of the building that is in the direction of the modal displacement
\( L_Z \): turbulence length scale at \( Z \)

\( \bar{M}_j(\alpha) \): peak base moment and torque about \( j \)-axis and at wind angle \( \alpha \)

\( \bar{M}_{\text{Code}} \): peak base moment/torque calculated using design codes

\( \bar{M}_i \): peak base moment, \( i = \) along-wind, across-wind, or torque

\( \bar{M}_j^\alpha \): peak base moment evaluated by WAD, NALD, ASCE 7-10, AS/NZS 1170.2:2011, or NBCC 2010 about the \( j \)-axis

\( \bar{M}_j^{WT} \): peak base moment evaluated by wind tunnel analysis about the \( j \)-axis

\( \bar{M} \): peak along-wind base bending moment

\( \bar{M}_j(\alpha) \): mean component of the base moment about \( j \)-axis and at wind angle \( \alpha \)

\( \bar{M}_i \): mean base bending moment or torque, \( i = \) along-wind, across-wind, or torque

\( \bar{M} \): mean along-wind base bending moment

\( \bar{M}_j(t) \): dynamic base moments incorporating dynamic response of the building about \( j \)-axis and at time \( t \)

\( M_j' \): reference moment or torque, \( i = \) along-wind, across-wind, or torque

\( M_j^* \): modal mass for dominant mode along \( j \)-axis

\( M_j \): base moment about \( j \)-axis

\( m_1 \): generalized mass of the first mode

\( m_\infty \): average mass per unit height

\( m(z) \): mass per unit height

\( N(n) \): fuzzy value of near-field exposure input

\( n_s \): frequency of vortex shedding

\( \bar{P}_j(z, \alpha) \): peak equivalent static wind load along \( j \)-axis, at wind angle \( \alpha \), and elevation \( z \)

\( \bar{P}(z) \): peak along-wind load at \( z \)

\( \bar{P}_j(z, \alpha) \): mean component of the equivalent static wind load along \( j \)-axis, at wind angle \( \alpha \), and elevation \( z \)

\( \bar{P}(z) \): mean along-wind load

\( P_{Bj}(z, \alpha) \): background component of the equivalent static wind load along \( j \)-axis, at wind angle \( \alpha \), and elevation \( z \)

\( P_{Rj}(z, \alpha) \): resonant component of the equivalent static wind load along \( j \)-axis, at wind angle \( \alpha \), and elevation \( z \)
\( q_{\text{des}} \): design wind velocity pressure
\( q_{\text{des,ww}}(k\Delta H) \): design wind velocity pressure for the windward wall
\( q_{\text{des,lw}}(k\Delta H) \): design wind velocity pressure for the leeward wall
\( q_Z \): along-wind velocity pressure at equivalent height of the building
\( R_{h,B,L} \): factors for evaluating the size reduction factor in ASCE 7-10
\( R \): resonance response factor
\( r \): wind pressure turbulence factor
\( r_r \): denote the absolute difference between the aspect ratios of the target and candidate buildings divided by the aspect ratio of the target building; \( r_A \) is calculated in the same way as \( r_r \) but for the slenderness ratio
\( S M_i \): spectrum of the aerodynamic base bending moment or base torque, \( i = \) along-wind, across-wind, or torque
\( S M_j \): power spectrum of the aerodynamic base moment about \( j \)-axis
\( S M'_j \): normalized spectrum of base moment about \( j \)-axis
\( S_t \): Strouhal Number
\( s \): size reduction factor
\( T_d \): observation period
\( t \): time
\( \bar{V}_H(\alpha) \): mean hourly wind speed at \( H \) and at wind angle \( \alpha \)
\( \bar{V}(z) \): mean velocity of the approaching flow
\( V \): basic wind speed
\( W \): \( B \) or \( D \) for wind acting perpendicular or parallel to the longer dimension of the building, respectively
\( \bar{Z} \): equivalent height
\( z \): elevation
\( \Delta \): maximum deflection at the top of the building for NBCC 2010
\( \Delta H \): height of tributary area
\( \alpha \): wind angle
\( \beta \): power-law exponent of the mean wind speed profile
\( \phi_j(z) \): mode shape for fundamental mode along \( j \)-axis
\( \phi_1(H) \): mode shape value for the first fundamental mode at top of the building
\( y_j^* \): peak base moment coefficient about \( j \)-axis

\( \rho \): air density

\( \rho_B \): average bulk density of the building

\( \sigma_{CM_j} (\alpha) \): RMS base moment coefficient about \( j \)-axis and at wind angle \( \alpha \)

\( \sigma_{MB_i} \): background RMS base bending moment, \( i = \) along-wind, across-wind, or torque

\( \sigma_{MR_i} \): resonant RMS dynamic base moment, \( i = \) along-wind, across-wind, or torque

\( \sigma_{MR_j} \): resonant RMS dynamic base moment about \( j \)-axis

\( \sigma_{M_j}(\alpha) \): root-mean square component of the base moment about \( j \)-axis and at wind angle \( \alpha \)

\( \nu \): up-crossing rate

\( \omega^* \): peak resultant acceleration coefficient

\( \xi_1 \): damping ratio for the fundamental mode of the building

\( \xi_2 \): damping ratio for the across-wind mode of the building

\( \xi_i \): damping ratio, \( i = \) along-wind, across-wind, or torque

\( \xi_j \): damping ratio for fundamental mode along \( j \)-axis
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Slender and lightweight structures such as tall buildings are susceptible to the aerodynamic wind loads and corresponding wind-structure interaction. The aerodynamic forces on tall buildings, which are generally treated as bluff bodies, are governed by regions of separated flow, drag forces, and the formation of vortex shedding (Roshko 1993). A tall building submerged in the wind oscillates according to its dynamic properties. Currently, there are three main approaches to determine the design wind loads on tall buildings, namely design codes, the database-assisted approach, and wind tunnel testing.

All the major design codes utilize the gust loading factor approach developed by Davenport (1967) to estimate the design wind loads, but the design codes have limitations on the building’s height, dynamic and aerodynamic characteristics, and exposure conditions. The wind tunnel testing is the most accurate means of determining the design wind loads on a tall building; however, the wind tunnel testing is usually not used for determining the preliminary design wind loads. Detailed information with regards to the building’s dynamic properties such as natural frequencies, mode shapes, distribution of the mass and mass moment of inertia along the height of the building is required by the wind tunnel analysis but are typically unavailable during the preliminary design phase. Moreover, significant changes in the geometry of the building, which is not uncommon during the course of building design, will result in retests that can be time-consuming and costly.

A viable alternative to design codes for determining the preliminary design wind loads on tall buildings is the database-assisted approach. The basic premise of the database-assisted approach is that the quasi-steady wind loads on a building can be determined using the aerodynamic loads of another building that has similar aerodynamics previously tested in the wind tunnel. Buildings with similar aerodynamics have similar loads when normalized by the corresponding building geometric parameters and wind speeds, and
wind tunnel testing provides the best collection of the normalized aerodynamic loads. The vibration-induced responses can be determined by applying the random vibration analysis. Perhaps the most well-known example of the database-assisted approach is the NatHaz Aerodynamic Load Database (NALD) developed at the University of Notre Dame (Zhou et al. 2003), which is also recommended in ASCE7-10 (ASCE 2010) as an acceptable alternative. NALD allows a user to calculate wind-induced responses such as the across-wind and torsional responses, of which corresponding provisions are limited or do not exist in design codes. The theoretical basis for NALD-based wind load evaluation is the three-dimensional moment gust loading factor approach (3D MGLF) (Kareem and Zhou 2003). Other examples of the database-assisted approach include those developed by Main and Fritz (2006) for low-rise and high-rise structures, from the National Institute of Standards and Technology (NIST) of the US.

Note that NALD only considers the wind load on isolated tall buildings, i.e. no surrounding buildings; however, in reality a tall building is usually surrounded by other buildings with various orientations, sizes and distances from the building under consideration. The interference and shielding effects associated with a building’s surrounding can significantly impact wind-induced responses of the building and therefore should be taken into consideration in evaluating the design wind loads on the building (Khanduri 1997). Furthermore, the aerodynamic data stored in NALD are from a limited number of buildings. No guidance is provided in NALD as to which building to select to evaluate the wind loads on a given tall building under consideration; which is problematic if the building geometry does not exactly match one of the buildings stored in NALD.

The Boundary Layer Wind Tunnel Laboratory (BLWTL) at Western University has been conducting wind tunnel tests of tall buildings for the past four decades and accumulated a large number of test data, which contain highly valuable information on aerodynamic loads on tall buildings with a wide variety of geometric shapes exposed to diverse terrain and interference/shielding conditions. The motivation of the study reported in this thesis was to take advantage of the test data in BLWTL by compiling an aerodynamic database of tall buildings to facilitate the evaluation of wind loads for preliminary design of tall
buildings. A reasonably accurate estimate of the wind loads can help to set appropriate initial design parameters and avoid costly concept changes in later stages of the building development. The advantage of such a database, entitled the Western University Aerodynamic Database (WAD) of tall buildings, is that it incorporates major influencing factors for wind loads such as the wind attack angle, building geometry, general terrain conditions, and direct surrounding conditions as described in Chapter 3 of this thesis. It should be emphasized that WAD is intended to supplement the existing methods of estimating the preliminary design wind loads and not aimed at replacing wind tunnel testing.

The objectives of the study presented in this thesis were to 1) review the existing methods of calculating design wind loads on tall buildings, 2) develop WAD and the procedure to evaluate the wind-induced responses including the acceleration and base moments using WAD, and 3) compare the WAD-based wind responses with those determined from wind tunnel tests, design codes, and NALD for a range of tall buildings to validate the WAD-based procedure.

This thesis is prepared in a monograph format and consists of five chapters. References and appendices for all chapters are placed at the end of the thesis. The contents of each chapter are briefly described in the following:

Chapter 2 presents an overview of wind loads on tall buildings, which includes a brief discussion on bluff body aerodynamics, major factors that impact the aerodynamics of tall buildings, and current approaches to evaluate the wind load on tall buildings. The gust loading factor approach is also described in Chapter 2. Chapter 3 presents details of the development of WAD as well as the WAD-based procedure to evaluate the wind-induced responses of tall buildings including the base bending moments in two orthogonal directions and base torque as well as the equivalent static wind loads and acceleration. The validation of WAD-based procedure and comparison of different methods of evaluating the wind responses are described in Chapter 4. Chapter 5 includes summaries, main conclusions and recommendations for future work. The derivation of
the modified three-dimensional moment gust loading factor approach implemented in WAD is presented in Appendix A. Supplementary figures, plots, and tables can be found in subsequent appendices.
The aerodynamics of a tall building induced by the wind flow surrounding the building is characterized as that of a bluff body (Roshko 1993). The key factors affecting the aerodynamic loads on a bluff body include the approaching boundary layer wind, the characteristics of the bluff body, and the conditions of the direct surroundings of the body such as the presence of other bluff bodies.

Many tall buildings are slender and light structures that are sensitive to the gust buffeting from the wind (Davenport 1966; Lin et al. 2005; Holmes 2007). Therefore, in addition to the aerodynamics of a tall building, the vibration response of the structure is also a key consideration in the design for wind. Structural engineers at present primarily rely on the design codes and wind tunnel testing to determine the design wind loads on tall buildings. The wind load provisions in all major structural design codes and standards worldwide are based on the gust loading factor approach developed by Davenport (1967). Wind tunnel testing provides accurate estimates of the wind loads by testing scaled models of tall buildings in simulated atmospheric boundary layers. Recently, the database-assisted procedure has emerged as an alternative means to design codes for estimating the preliminary design wind loads (Main and Fritz 2006; Kwon et al. 2008).

The rest of Chapter 2 is organized as follows: Section 2.2 provides a brief description of the bluff body aerodynamics; key factors that affect the aerodynamics of a tall building are discussed in Section 2.3; a review of the gust loading factor approach is included in Section 2.4; the three approaches for determining the design wind loads on tall buildings, i.e. the design codes, wind tunnel testing and the NALD database-assisted procedure, are discussed in detail in Section 2.5, and a summary of Chapter 2 is included in Section 2.6.
Tall buildings under the action of wind are generally treated as prismatic bluff bodies that have various plan dimensions and oscillate in the along-wind, across-wind, and torsional directions (Lin et al. 2005; Holmes 2007). Compared to streamlined bodies, where the flow streamlines follow the outlines of the body, bluff bodies are characterized by large regions of separated flow, large drag forces, and the formation of vortex shedding (Roshko 1993). Figure 2.1 is a schematic plan view of the average air flow around a bluff body with a rectangular cross-section (Roshko 1993; Simiu 1996; Holmes 2007). As indicated in the figure, the separated flow region consists of the outer region, where there are no viscous effects, and the inner region, where viscous effects govern. A thin region known as the free shear layer that has complex flow characteristics with high shear and vorticity separates the inner and outer regions (Holmes 2007). If the bluff body has a long after-body, the flow may reattach to the surface of the body and will be followed by a second separation point at the corners downstream of the body (Taylor et al. 2011); otherwise the flow will remain separated and generate a large wake at the lee of the body. A separation bubble is formed between the free shear layer and the body from the initial separation point and the reattachment point (Djilali 1992; Taylor et al. 2011). The wake region downstream of the body is characterized by a region with low velocity and turbulent flow (Holmes 2007).

The unstable nature of the flow surrounding the bluff body and the turbulent nature of the approaching air flow generate highly fluctuating loads. First, the approaching flow defined as the atmospheric boundary layer has natural turbulence or gustiness often called buffeting. Second, the bluff body itself can generate unsteady flows through separation of flow, reattachment, and vortex shedding. Finally, the movement of the body can also generate fluctuating forces also known as the aerodynamic damping, which can be significant for highly flexible vibration-prone aero-elastic structures. The response of a tall building under wind load consists of components in the along-wind, across-wind, and torsional directions, as illustrated in Figure 2.2. The along-wind response of a building is the response of the building parallel to the direction of wind. The across-wind response
is the building’s response perpendicular to the direction of wind. Lastly, the torsional response describes the twisting motion of the building about the vertical axis.

![Figure 2.1: Schematic of the Air Flow around a Bluff Body](image)

The along-wind load results from the net pressure fluctuations acting in the direction parallel to the wind. The along-wind response of a tall building is generally considered by applying the quasi-steady theory (Richards and Hoxey 2004), which assumes that the fluctuating pressure on the windward face on the structure varies directly with the fluctuation of the longitudinal wind velocity upstream. The total along-wind force is the sum of the forces acting on the windward and leeward faces of the structure. The load in the leeward face of the structure is generally caused by the pressure fluctuations in the wake recirculation region. As discussed in Section 2.2.1, the wake recirculation region is highly turbulent but has low velocities, and in turn low pressures.

![Figure 2.2: Wind Load Axes](image)
Vortex shedding is the primary mechanism for generating the across-wind response on a bluff body (Davenport 1966; Lin et al. 2005; Holmes 2007). Vortex shedding is described as the alternating shedding of vortices from the rolling up of separating shear layers into the wake and is influenced by the turbulence in the approaching flow (Davenport 1966; Holmes 2007). The frequency of the alternating forces, i.e. vortex shedding, can be expressed as a non-dimensional value known as the Strouhal number, $S_t$, defined as follows (Simiu 1996; Holmes 2007):

$$S_t = \frac{n_s b}{\bar{v}}$$

(2.1)

where $n_s$ is the frequency of the vortex shedding (i.e. the Strouhal frequency); $d$ is the across-wind characteristic length, i.e. plan dimension perpendicular to the direction of wind, and $\bar{v}$ is the mean velocity of the approaching flow. The forces generated by the shedding of vortices on the structure depend on the turbulence in the flow, the dimensions of the bluff body, and the natural frequency of the bluff body (Davenport 1966). The across-wind force can be much larger than the along-wind force if the Strouhal frequency is at resonance with the natural frequency of the bluff body within a uniform steady flow (Davenport 1966).

The twisting motion of a bluff body subjected to air flow results from the non-uniform pressure distribution around the wall faces of the bluff body. This mechanism was generally studied through measuring aerodynamic loads in wind tunnel tests on bluff bodies with varying shapes, presence of other interfering bodies, and various angles of the approaching flow (Boggs et al. 2000). Figure 2.3 shows an example of the streamlines and pressure distribution of the flow when the flow is approaching the bluff body at an angle to a face. As shown in Figure 2.3, a non-uniform pressure distribution around the rectangular cylindrical bluff body is formed, thus inducing torsion on the body. The pressure distribution around the bluff body can change when the shape of the bluff body is altered. For example, the torque on a bluff body with the same shape as
shown in Figure 2.3 but with two opposite corners curved increases due to the imbalance of pressures caused by separation of flow from one corner but not the other because of the curved edge (Boggs 2000). Boggs (2000) also showed that the presence of interfering bluff bodies can increase the effect of torsion.

![Figure 2.3: Wind Load Streamlines and Pressure Distribution around a Bluff Body](image)

**Factors Affecting the Aerodynamics of Tall Buildings**

The aerodynamics of a tall building is a complex phenomenon influenced by a number of factors including the approaching boundary layer flow, building dimensions and direct surroundings of the building. The impact of each of these factors on the aerodynamics of the building is briefly described in the following:

The boundary layer flow approaching a tall building is described using the mean wind speed and turbulence intensity profiles with respect to height; which govern the magnitudes of the mean and root-mean square (RMS) values of the aerodynamic forces on the building respectively. Note that the turbulence intensity is defined as the RMS value of the fluctuating wind speed divided by the corresponding mean wind speed. The boundary layer flow depends on the general roughness of the terrain characterized by the roughness length, $z_o$. Figure 2.4 depicts the profiles of the mean wind velocity
(normalized by the mean wind velocity at the elevation of 600 m) and turbulence intensity corresponding to different terrain roughness lengths, generated using the Engineering Sciences Data Unit (ESDU) model (ESDU 2005). Each roughness length is associated with certain terrain conditions as summarized in Table 2.1, which is adapted from the Davenport Classification of Effective Terrain Roughness table provided in ASCE 7-10 (ASCE 2010). As shown in Figure 2.4, as the general terrain becomes flat and smooth the mean wind speed at a given height increases but the corresponding turbulence intensity decreases. Therefore, with the application of the quasi-steady theory, an increase in the roughness length leads to a decrease in the mean component of the load but an increase in the fluctuating component of the load. The trade-off between the mean and fluctuating components of the wind speed in response to the increase in the roughness length results in similar peak wind speeds among different terrain conditions, but in general terrains with less roughness have higher peak wind speeds.

**Figure 2.4: Standard Profiles for Mean Wind Speed and Turbulence Intensity**
Table 2.1: Terrain Categories Defined in ASCE 7-10

<table>
<thead>
<tr>
<th>Terrain Category</th>
<th>Terrain Characteristics</th>
<th>Roughness Length, $z_o$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth</td>
<td>Flat land surface without any noticeable obstacles</td>
<td>0.005</td>
</tr>
<tr>
<td>Open</td>
<td>Open country with low vegetation and isolated obstacles</td>
<td>0.03</td>
</tr>
<tr>
<td>Roughly Open</td>
<td>Area with low crops or plant covers, or moderately open country with occasional obstacles</td>
<td>0.1</td>
</tr>
<tr>
<td>Very Rough</td>
<td>Intensely cultivated landscape with many large obstacles</td>
<td>0.5</td>
</tr>
<tr>
<td>Skimming</td>
<td>Landscape regularly covered with similar-size large obstacles</td>
<td>1.0</td>
</tr>
<tr>
<td>Chaotic</td>
<td>City centers with mixture of low-rise and high-rise buildings</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Vickery (1968) and Sakamoto (1985) conducted studies on the fluctuating loads on bluff bodies in turbulent flow. All of the bluff bodies were rectangular cylinders with varying plan dimensions and heights. The shapes of the bluff bodies were found to impact the patterns of the flow around the bodies and consequently the drag and lift forces on the bodies. The results reported by Vickery (1968) show that the mean drag force decreases as $B/D$ or $2H/B$ decreases, where in this reference $B$ and $D$ are defined as the plan dimensions perpendicular and parallel to the direction of flow, respectively, and $H$ is the overall height of the bluff body. In general, the total mean drag force coefficients increases as the surface area of the bluff body increases. Furthermore, the shapes of the power spectral density functions (or spectrum for simplicity) of the fluctuating drag and lift forces were observed to depend on $B/D$: the spectrum becomes more peaked as $B/D$ increases above unity and conversely becomes flatter as $B/D$ decreases below unity.

Tall buildings are often located in urban environments and surrounded by buildings of similar sizes. The aerodynamics of a tall building is impacted by its surrounding buildings, known as the interference and shielding effects (Khanduri 1997). The interference effects depends on many factors such as the size and shape of the interfering
building, distance between the interfering building and the building being interfered (i.e. downstream in the wind), the orientation of both interfering building and downstream building, lateral offset in building axes of the interfering building relative to the downstream building and wind flow direction, the number of interfering buildings, attack angle of the wind, and the roughness of the upstream terrain (Bailey 1985; Kareem 1987; Tamiike 1991; Khanduri 1997; Gu 2005; Xie 2009). In general, the interference effects increase the fluctuating load but decreases the mean load. The reduction of the wind load caused by surrounding buildings is referred to as the shielding effect (English 1990; English 1993; Khanduri 1997). A review and summary of the interference and shielding effects of surrounding buildings can be found in Khanduri (1997).

It has been reported that neither the along-wind nor across-wind response of a downstream building with a rectangular plan is significantly impacted by the interference of buildings of any size in highly turbulent environments (Kareem 1987; Tamiike 1991). This is attributed to the fact that higher turbulence in the flow reduces the organization of vortex shedding from the interfering building; which results in a reduction of the loads on the downstream building. The size of the interfering building impacts the interference effects as well, but an increase in the size of the interfering building does not always result in more pronounced interference effects. An interfering building that is significantly larger than the downstream building can shed vortices that are larger than the size of the downstream building, the downstream building is shielded by the interfering building and the overall wind loads are expected to be lower (Bailey 1985).

Studies of shielding and interference effects of grouped buildings are limited in the literature due to the difficulty in considering numerous combinations of building arrangements and sizes. Gu et al. (2005) and Xie and Gu (2009) studied the along-wind and across-wind interference effects of two and three tall building arrangements, respectively. Lam et al. (2008) studied the interference effects of tall buildings arranged in a row. Lam and Zhao (2008) studied the interference effects of tall buildings arranged in L- and T-shapes. All these studies focused on buildings of equal sizes. Stone (1987) studied the interference caused by different orientations of a group of buildings of equal size. Studies on the interference effect caused by large groups of buildings are also
reported by Soliman (1976), and Hussain and Lee (1980). All of the above-mentioned studies suggest that the wind loads on a tall building surrounded by a large group of buildings of equal size are generally less severe than those on the same building in an isolated condition.

For highly dynamic structures, the gust loading factor (GLF) approach proposed by Davenport (1967) is the basis of the procedures used to determine the along-wind responses specified in all major structural design codes around the world. The peak along-wind pressure ($\tilde{P}(z)$) at a given elevation $z$ of a structure is expressed as the gust loading factor ($G$) multiplied by the corresponding mean along-wind pressure ($\bar{P}(z)$) (Davenport 1967):

$$\tilde{P}(z) = G\bar{P}(z)$$ (2.2)

The GLF approach accounts for the gustiness of the wind as well as amplification of the wind load due to the dynamics of the structure. GLF can be computed based on the displacement response, i.e. the so-called displacement gust loading factor (DGLF), or other responses such as the bending moment and shear force at any elevation of the structure. The evaluation of DGLF is briefly described in the following based on Kareem and Zhou (2003):

Idealizing the structure as a single-degree-of-freedom system characterized by its fundamental sway mode shape, one can evaluate DGLF as the gust loading factor for the generalized coordinate associated with the first mode as follows:

$$G = 1 + 2I_H\sqrt{g_B^2B + g_R^2R}$$ (2.3)

where $I_H$ is the turbulence intensity at the top of the structure (i.e. $z = H$); $B$ is the background turbulence factor; $g_B$ and $g_R$ are the background peak factor which follows the wind pressure peak factor and resonant peak factor, respectively, and $R$ is the resonant
response factor. The resonant response factor is expressed as \( R = sE/\xi_1 \), where \( s \) is size reduction factor; \( E \) is the gust energy ratio, and \( \xi_1 \) is the critical damping ratio for the fundamental mode. The resonant peak factor is given by (Davenport 1964):

\[
g_R \approx \sqrt{2\ln(vT_d)} + \frac{0.5772}{\sqrt{2\ln(vT_d)}}
\]  

(2.4)

where \( v \) is the up-crossing rate that can be approximated as the fundamental frequency of the structure assuming the structural response to be a narrow-banded Gaussian process, and \( T_d \) is the observation period in seconds and typically equals 3600 s (one hour).

Based on the simplifying assumption used in codes that the structure can be idealized as a single-degree-of-freedom system characterized by its linear mode shape, the DGLF given by Eq. (2.3) is equal to the gust loading factor associated with the base bending moment (Kareem and Zhou 2003). Therefore, the peak along-wind base bending moment (\( \bar{M} \)) equals the mean along-wind base bending moment, \( \bar{M} \) (i.e. the base bending moment caused by the mean wind pressure), multiplied by \( G \), i.e. \( \bar{M} = G\bar{M} \).

In reality, slender structures such as tall buildings vibrate simultaneously in the along-wind, across-wind, and torsional directions under the action of wind. The traditional GLF approach described in Section 2.4.1 applies to the along-wind response only; little guidance is provided in the design codes to deal with the across-wind and torsional responses. Three-dimensional (3D) GLF approaches have been reported in literature (e.g. Piccardo and Solari 2002; Kareem and Zhou 2003) that apply to building responses in the along-wind, across-wind, and torsional directions. In particular, Kareem and Zhou (2003) proposed a 3D moment gust loading factor (MGLF) approach that applies to the base moments in the along-wind, across-wind and torsional directions. Because it is the basis of the WAD-based wind load evaluation procedure developed in this study, the 3D MGLF approach is discussed in the following:
Denote the base bending moments and base torque in a building by a generic symbol, \( M_i \), where \( i = AL, AC \) and \( T \), representing the along-wind, across-wind and torsional directions respectively. The peak value of \( M_i \), \( \bar{M}_i \), is given by

\[
\bar{M}_i = \bar{M}_i + \sqrt{g_B^2 \sigma_{MB_i}^2 + g_R^2 \sigma_{MR_i}^2}
\]

(2.5)

where \( \bar{M}_i \) is typically close to zero in the across-wind and torsional directions for most symmetrical buildings (Kareem and Zhou 2003), and \( \sigma_{MB_i} \) and \( \sigma_{MR_i} \) are the background and resonant components of the RMS value of the dynamic base moments in the \( i^{th} \) direction respectively. Introducing the reference moment or torque \( M'_i \), where \( M'_i \) is typically a function of the building plan dimensions and height, as well as the mean wind velocity at the roof top, one can recast Eq. (2.5) as follows (Kareem and Zhou 2003):

\[
G_i = \bar{G}_i + \sqrt{G_{Bi}^2 + G_{Ri}^2}
\]

(2.6)

\[
G_{Bi} = g_B \sigma_{MB_i} / M'_i
\]

(2.7)

\[
G_{Ri} = g_R \sigma_{MR_i} / M'_i
\]

(2.8)

\[
M'_{AL} = \frac{1}{2} \rho \bar{V}_H^2 b H^2
\]

(2.9)

\[
M'_{AC} = \frac{1}{2} \rho \bar{V}_H^2 L H^2
\]

(2.10)

\[
M'_T = \frac{1}{2} \rho \bar{V}_H^2 b LH
\]

(2.11)

where \( \bar{V}_H \) is the mean hourly wind speed at roof height, \( H \); \( \rho \) is the density of air; \( L \) is the plan dimension of the building that is in the direction of the modal displacement; \( G_i = \bar{M}_i / M'_i \) is the 3D MGLF; \( \bar{G}_i = \bar{M}_i / M'_i \), \( G_{Bi} \) and \( G_{Ri} \) are the mean, background and resonant components of \( G_i \) respectively. \( \sigma_{MB_i} \) can be set to equal the RMS value of the fluctuating aerodynamic base bending moment or base torque; \( \sigma_{MR_i} \) can be computed
based on the random vibration theory as follows, assuming that the mode shapes in the fundamental modes are linear and uncoupled (Kareem and Zhou 2003):

$$\sigma_{R_{M_i}} = \sqrt{\frac{\pi f_{1i}}{4 \xi_{1i}}} S_{M_i}(f_{1i})$$

(2.12)

where $f_{1i}$ and $\xi_{1i}$ are the fundamental natural frequencies and corresponding critical damping ratios in the along-wind, across-wind or torsional direction respectively, and $S_{M_i}(f)$ is the spectrum of the fluctuating aerodynamic base bending moment or base torque.

In the following sections, provisions for determining wind loads on tall buildings that are flexible, light-weight, and sensitive to the wind from three representative design codes are briefly discussed. The design codes considered in this study are the National Building Code of Canada NBCC 2010 (NRC 2010), Australian/New Zealand Standard AS/NZS 1170.2:2011 (Australian/New Zealand Standard 2011), and American Society of Civil Engineers Standard ASCE 7-10 (ASCE 2010) design codes. The general procedures in these design codes are similar in that the peak along-wind design pressures are determined as the product of the mean wind pressure and the gust loading factor (GLF); however, there are differences among the three design codes, which lead to a scatter in the predicted wind loads for a given structure. An in-depth study has recently been carried out by Kwon and Kareem (2013), who compared eight international design codes, including the above-mentioned three codes. The comparison carried out in this study is focused on the definition of wind field characteristics and approaches to deal with the wind-induced dynamic response of the structure in these three codes.

The design wind velocity pressure in all three design codes is calculated from the basic wind speed, $V$, and factors that account for the terrain conditions, topographic conditions,
surface roughness, and, for AS/NZS 1170: 2011 only, shielding conditions. The averaging times for the basic wind speed and factors applied to the design wind velocity pressure in the three codes are summarized in Table 2.2.

### Table 2.2: Averaging Times for Basic Wind Speed and Formulas for Calculating Design Wind Velocity Pressure

<table>
<thead>
<tr>
<th>Terrain Factor</th>
<th>Basic Wind Speed Averaging Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_z$</td>
<td>3 seconds</td>
</tr>
<tr>
<td>$M_{z,cat}$</td>
<td>3 seconds</td>
</tr>
<tr>
<td>$C_e$</td>
<td>one hour</td>
</tr>
</tbody>
</table>

### Table 2.3: Exposure Categories in Design Codes

<table>
<thead>
<tr>
<th>Exposure</th>
<th>ASCE 7-10</th>
<th>AS/NZS 1170.2: 2011</th>
<th>NBCC 2010</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open Water</td>
<td>D</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Open Country</td>
<td>C</td>
<td>2</td>
<td>A</td>
</tr>
<tr>
<td>Suburban</td>
<td>B</td>
<td>3</td>
<td>B</td>
</tr>
<tr>
<td>Urban</td>
<td>-</td>
<td>4</td>
<td>-</td>
</tr>
</tbody>
</table>

1 When the NBCC 2010 topographic factor is used, $C_e^*$ is used in place of $C_e$.

The basic wind speeds in all of the three design codes are defined at 10 m above ground in the flat/open exposure condition but have different averaging times. NBCC 2010 is based on the mean hourly wind speed, while ASCE7-10 and AS/NZS 1170: 2011 are based on the 3-second gust peak wind speed. ASCE 7-10 and NBCC 2010 adopt a power-law wind velocity profile, whereas AS/NZS 1170.2: 2011 adopts a logarithmic wind velocity profile. The three codes consider two to four exposure categories that range from open water to urban exposures, which are summarized in Table 2.3.

### Table 2.4: GLF Defined in the Three Design Codes

Table 2.4 compares the GLF defined in the three design codes. As shown in the table, the GLF is calculated at the equivalent height ($\bar{z}$) of 0.6$H$ in ASCE 7-10 and at $\bar{z} = H$ in
NBCC 2010 and AS/NZS 1170.2:2011. In NBCC 2010 the background \( (g_B) \) and resonant \( (g_R) \) peak factors are equal where an additional term (i.e. the term in the square root) is included in the average fluctuation rate \( (\nu) \) to account for both the background and resonant effects. As for ASCE 7-10 and AS/NZS 1170.2:2011, the background \( (g_B) \) peak factor is a constant value.

The duration of the peak associated with the wind-induced dynamic response are one hour and ten minutes in ASCE 7-10 and AS/NZS 1170.2:2011, respectively. For designs based on the mean hourly wind speed, the effect of gustiness of the wind is included by factoring up the mean wind speed; on the other hand, the designs based on the 3-second gust wind speeds inherently include the effects of gustiness. For this reason, a so-called gust effect factor (GEF) or dynamic response factor (DRF) is defined in design codes based on the 3-second gust wind speed to differentiate from the GLF employed in the design codes based on the mean hourly wind speed (Holmes et al. 2009; Solari and Kareem 1998). The general format of the GEF can be written as

\[
GEF = \frac{GLF}{G_q}
\]  
(2.13)

\[
G_q = 1 + rg_B
\]  
(2.14)

where \( r \) is the wind pressure turbulence factor (equal to \( 2I_H \) for AS/NZS 1170.2:2011 and \( 1.7I_H \) for ASCE 7-10; where \( I_H \) is the turbulence intensity at the roof height); \( G_q \) is the gust factor for the wind velocity pressure and is used to convert the GEF from the duration of the wind-induced dynamic response to the averaging time of the basic wind velocity. In calculating the gust energy ratio \( E \), ASCE 7-10 uses Kaimal’s spectrum, whereas AS/NZS 1170.2:2011 and NBCC 2010 use Karman’s spectrum and Davenport’s spectrum, respectively. The resonant response factor \( (R) \) is a function of the size reduction factor \( (s) \), gust energy ratio \( (E) \), and the damping ratio \( (\xi) \). Clear differences between the three codes can also be found in the evaluation of the background response factor, \( \mathcal{B} \). The background factor for all design codes is a function of the building’s dimensions and the turbulence length scale \( (L_g) \).
In Table 2.4, $I_z$ is the turbulence intensity at the equivalent height $z$; $\mathcal{K}$ is a factor related to the surface roughness coefficient of the terrain; $C_{eH}$ is the terrain factor at the roof height of the structure; $f_1$ is the fundamental frequency of the structure; $b$ is the plan dimension of the building that is perpendicular to the direction of the wind, and $H_s$ is an additional mode correction factor used for the resonant response factor in AS/NZS 1170.2:2011; which is equal to 1.0 for linear modes. The factors in the evaluation of the size reduction factor using ASCE 7-10 are determined using the following equations:

$$R_{h,B,L} = \begin{cases} \frac{1}{\eta_{h,B,L}} - \frac{1}{2\eta_{h,B,L}^2} (1 - e^{-2\eta_{h,B,L}}) & \eta_{h,B,L} > 0 \\ \frac{1}{\eta_{h,B,L}} & \eta_{h,B,L} = 0 \end{cases}$$  \hspace{1cm} (2.15)$$

$$\eta_h = \frac{4.6f_1 H}{\bar{v}_2}$$  \hspace{1cm} (2.16)$$

$$\eta_B = \frac{4.6f_1 B}{\bar{v}_2}$$  \hspace{1cm} (2.17)$$

$$\eta_L = \frac{15.4f_1 L}{\bar{v}_2}$$  \hspace{1cm} (2.18)$$

AS/NZS 1170.2:2011 also have provisions for both evaluating the across-wind base bending moments and accelerations. Across-wind loads may be determined for buildings with proportions of $H:B:D$ of 3:1:1, 6:1:1, 6:1:2, and 6:2:1 and turbulence intensities of 0.12 and 0.2; and interpolation is used for buildings with intermediate values of $H/B$, $B/D$, and turbulence intensity. More details on the across-wind base moments can be found in literature (e.g. Kwon and Kareem 2013) and in the design code, AS/NZS 1170.2:2011.
Table 2.4: Gust Loading Factor

<table>
<thead>
<tr>
<th></th>
<th>ASCE 7-10</th>
<th>AS/NZS 1170.2:2011</th>
<th>NBCC 2010</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gust Loading Factor, $G$</td>
<td>$0.925 \left( 1 + 1.71f_x \sqrt{\frac{g_B^2 \mathcal{B} + g_R^2 R}{1 + 1.7 g_B l_z}} \right)$</td>
<td>$1 + 2 g_B \sqrt{\frac{g_B^2 \mathcal{B} + g_R^2 R}{1 + 2 g_B l_z}}$</td>
<td>$1 + g_R \sqrt{\frac{K}{C_{eh} (\mathcal{B} + R)}}$</td>
</tr>
<tr>
<td>Equivalent Height, $\bar{z}$</td>
<td>$0.6H$</td>
<td>$H$</td>
<td>$H$</td>
</tr>
<tr>
<td>Wind Velocity Peak Factor, $g_B$</td>
<td>3.4</td>
<td>3.7</td>
<td>$\sqrt{2 \ln(3600v)} + \frac{0.577}{\sqrt{2 \ln(3600v)}}$</td>
</tr>
<tr>
<td>Resonant Peak Factor, $g_R$</td>
<td>$\sqrt{2 \ln(3600v)} + \frac{0.577}{\sqrt{2 \ln(3600v)}}$</td>
<td>$\sqrt{2 \ln(600v)}$</td>
<td>$\sqrt{2 \ln(3600v)} + \frac{0.577}{\sqrt{2 \ln(3600v)}}$</td>
</tr>
<tr>
<td>Average Fluctuation Rate, $f$</td>
<td>$f_i$</td>
<td>$f_i$</td>
<td>$f_i \frac{sE}{sE + \xi \mathcal{B}}$</td>
</tr>
<tr>
<td>Background Factor, $\mathcal{B}$</td>
<td>$\frac{1}{1 + 0.63 \left( \frac{b + H}{L_x} \right)^{0.63}}$</td>
<td>$\frac{1}{1 + \sqrt{0.26 (H)^2 + 0.46 b^2}}$</td>
<td>$\frac{4}{3} \int_0^x \left[ \frac{1}{1 + \frac{xH}{457}} \left[ \frac{1}{1 + \frac{xb}{122}} \right] \frac{x}{(1 + x^2)^{4/3}} \right] dx$</td>
</tr>
<tr>
<td>Resonant Factor, $R$</td>
<td>$\frac{sE}{\xi}$</td>
<td>$\frac{H_s sE}{\xi}$</td>
<td>$\frac{sE}{\xi}$</td>
</tr>
<tr>
<td>Size Reduction Factor, $s$</td>
<td>$R_h R_\theta (0.53 + 0.47 f_n)$</td>
<td>$\left[ 1 + \frac{3.5 f_i H (1 + g_B l_z)}{V_x} \right] \left[ 1 + \frac{4 f_i (1 + g_B l_z)}{V_x} \right]$</td>
<td>$\frac{\pi}{3} \left[ \frac{1}{1 + \frac{8 f_i H}{3 V_x}} \right] \left[ \frac{1}{1 + \frac{10 f_i b}{V_x}} \right]$</td>
</tr>
<tr>
<td>Spectrum of Turbulence (Gust Energy Ratio), $E$</td>
<td>$\frac{7.47 f_n}{(1 + 10.3 f_n)^{5/3}}$ (Kaimal’s spectrum)</td>
<td>$\frac{\pi f_n}{(1 + 70.8 f_n^2)^{5/6}}$ (Karman’s spectrum)</td>
<td>$\frac{f_n^2}{(1 + f_n^2)^{4/3}}$ (Davenport’s spectrum)</td>
</tr>
<tr>
<td>Reduced Frequency, $f_n$</td>
<td>$\frac{f_i L_x}{V_x}$</td>
<td>$\frac{f_i L_x (1 + g_B l_z)}{V_x}$</td>
<td>$\frac{1220 f_i}{V_x}$</td>
</tr>
</tbody>
</table>
For serviceability design, the acceleration response under wind is of concern. As discussed in Kwon and Kareem (2013), the acceleration response in the along-wind direction at the top of the building can be expressed in the following general form per ASCE 7-10 and AS/NZS 1170.2:2011:

$$\hat{a}_{AL}(H) = \frac{q_zG_R C_{f_x} b H K}{m_1} \phi_1(H)$$

(2.19)

where $q_z$ is the velocity pressure at the equivalent height; $C_{f_x}$ is the drag force coefficient of the absolute sum of the windward and leeward pressure coefficients (equal to 1.3); $m_1$ is the generalized mass of the first mode; $K$ is the mode shape correction factor; $\phi_1(H)$ is the modal displacement of the first mode at the top of the building (typically equal to 1.0), and $G_R$ is the resonant gust factor. Both $G_R$ and $K$ are summarized in Table 2.5.

<table>
<thead>
<tr>
<th>Table 2.5: Along-wind Acceleration Terms</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_R$</td>
</tr>
<tr>
<td>ASCE 7-10&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>AS/NZS 1170.2:2011&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>1</sup>$\hat{a}$ and $k$ are the exposure exponent of the wind speed profile and along-wind mode shape exponent respectively

<sup>2</sup>the term used for $K$ is an expression from Kwon and Kareem (2013)

The expression for estimating the peak along-wind acceleration in NBCC 2010 is not included in Table 2.5 because the peak along-wind acceleration does not follow the same approach and is calculated in terms of the maximum deflection of the building, $\Delta$, as follows:

$$\hat{a}_{AL}(H) = 4\pi^2 q_1^2 g_R \sqrt{\frac{\kappa S E \Delta}{C e H_{f_1} g}}$$

(2.20)

NBCC 2010 does not include equations for evaluating $\Delta$, but suggests that it be evaluated from a static analysis of the structure using the peak wind pressure on the structure.
The peak across-wind acceleration, $\hat{a}_{AC}(H)$, is estimated based on the base bending moment as follows in AS/NZS 1170.2:2011:

$$\hat{a}_{AC}(H) = \frac{1.5bg_B}{m_o} \left[ \frac{0.5\rho v^2}{(1+g_B l_2)^2} \right] K_m \sqrt{\frac{\pi C_{fs}}{\xi_2}}$$

(2.21)

where $m_o$ is the average mass per unit height; $K_m$ is the mode shape correction for the across-wind acceleration; $\xi_2$ is the critical damping ratio for the across-wind mode, and $C_{fs}$ is the across-wind force spectrum coefficient generalized for a linear mode shape. In NBCC 2010, the peak across-wind acceleration is estimated using the following empirical formula derived from wind tunnel measurements:

$$\hat{a}_{AC}(H) = f_2^2 g_B \sqrt{bL} \left( \frac{0.0785 \left[ \frac{V_H}{f_2 b L} \right]^{3.3}}{9.81 \rho_B \sqrt{\xi_2 b}} \right)$$

(2.22)

where $f_2$ is the fundamental natural frequency of the structure in the across-wind direction; $\xi_2$ is the damping ratio for the across-wind mode, and $\rho_B$ is the average bulk density of the building. ASCE 7-10 wind load provisions do not include formulas for evaluating the across-wind accelerations.

Although there are differences among design codes in the evaluation of the parameters involved in calculating the GLF, Kwon and Kareem (2013) found that the overall loads and accelerations are reasonably consistent in the along-wind direction.

All three codes recommend that the wind tunnel testing be carried out if a building meets any of the following conditions:

- total height is above 200 m;
- building is immersed in an urban/chaotic exposure;
- building is highly susceptible to across-wind, vortex shedding, or instability due to galloping or flutter;
• site is prone to channeling effects or buffeting in the wake of upstream obstructions such as surrounding buildings that may require special attention, and
• building is irregularly shaped.

In addition to the above, NBCC 2010 recommends that the wind tunnel testing be carried out if the fundamental frequency of a building is less than 0.25 Hz.

The wind tunnel testing of a tall building involves testing scaled models of the building in a boundary layer wind tunnel. The simulated boundary layer is developed using roughness elements, spires, and/or barriers as shown in Figure 2.5. Models of the surroundings of the building being tested are also geometrically scaled in the wind tunnel. Two methods commonly used to carry out the wind tunnel testing of tall buildings and determine the overall structural wind loads and responses are the force-balance model and pressure model tests (Tschanz 1982; Ho et al. 1999).

Figure 2.5: Simulation of the Atmospheric Boundary Layer
Force-balance models at the Boundary Layer Wind Tunnel Laboratory (BLWTL) of Western University are constructed using a lightweight high density foam material where only geometric scaling of the test building is considered when constructing the model (Tschanz 1982). The aerodynamic forces at the base of the model are directly measured by a high frequency ultra-sensitive force balance during the wind tunnel test. The aerodynamic loads recorded by the force balance are quasi-steady loads and do not include the effect of dynamic amplification. Care is taken either by model design or by control of the testing parameters to avoid contamination of the aerodynamic force spectra by resonance at the natural frequency of the balance-model system. The random vibration theory is then used to analytically evaluate the dynamic response that includes the resonant amplification at the natural frequencies for the fundamental modes of the building, assuming that aerodynamic damping is negligible. This is based on the fact that the generalized forces are directly related to the base moments given that the mode shapes for the fundamental sway modes of the building are approximately linear. Although the mode shape for the torsional mode is also approximately linear, the influence function for the base torque is unity along the height, thus an empirical correction is used to include the base torque in the generalized force. Detailed formulations of the force-balance method for determining the wind loads can be found in the literature (e.g. Ho and Jeong 2008).

In a pressure model test, pressure transducers are placed on the surface of a geometrically scaled rigid model of the building. The pressure taps are connected through vinyl tubes to pressure scanners that simultaneously measure pressures at various locations during the test. Similar to the force balance technique, the pressure model test only measures the quasi-steady aerodynamic loads. The aerodynamic force is determined from measurements at each pressure transducer by multiplying the pressure by the corresponding tributary area. The aerodynamic force can then be directly combined with the mode shapes of the building to calculate the generalized forces without the need to make specific assumptions about the mode shapes (Ho et al. 1999). The dynamic
amplification is derived using the same random vibration theory-based procedure as employed in the force-balance test.

The database-assisted approach provides a viable alternative to the design codes for determining the wind loads on tall buildings at the preliminary stage of the design process. The basic premise of the database-assisted approach is that the wind loads for a target building can be estimated by using the aerodynamic loads of another building with similar aerodynamics that has been previously measured in a wind tunnel test. The NatHaz Aerodynamic Loads Database (NALD) (Kwon et al. 2008), developed at the University of Notre Dame, as well as the databases developed at NIST are briefly described in the following.

NALD stores aerodynamic data from 54 different force-balance tests, which include nine different model cross-sections, three model heights (16, 20, and 24 inches) and two exposure categories (open and urban exposures with the power law exponents for the mean wind velocity profile equal to 0.16 and 0.35, respectively). All the test models were built with balsa wood and tested under isolated conditions (i.e. no surrounding buildings) in a 3×1.5×18 m boundary layer wind tunnel. In NALD, the mean components of the responses in the along-wind, across-wind and torsional directions corresponding to orthogonal angles of attack (i.e. wind acting perpendicular to a face) are determined using the ASCE 7-98 (ASCE 1998) procedures. The background components of the dynamic base moments are determined using the RMS aerodynamic base moment coefficients stored in NALD, whereas the resonant components of the dynamic base moments are determined using the non-dimensional aerodynamic base moment power spectra either stored in NALD or input by the user, and the 3D MGLF approach.

The current version of NALD (v. 2.0) is hosted by an Apache Web server found at http://www.nd.edu/~nathaz/. To use NALD to determine the wind loads on a tall building, the user is required to select the appropriate building from NALD, input the building’s dynamic properties and exposure condition, and select one of the three options for inputting the aerodynamic non-dimensionalized power spectral density functions for
the base moment: namely selecting the power spectrum stored in NALD, entering a user-defined spectrum, or inputting the spectrum obtained from the wind tunnel testing. Given these inputs, a Matlab® script embedded in the database is then executed to determine the wind loads on the building using the 3D MGLF approach (Kareem and Zhou 2003). The output from the database includes the peak base bending moments and torque, the displacement and accelerations at the top of the building, and the equivalent static wind loads that accounts for the fluctuation in the wind load and amplification of the wind load due to the dynamics of the structure.

Main and Fritz (2006) from NIST discuss in detail the so-called Database-Assisted Design (DAD) for low-rise and high-rise buildings. The DAD used for rigid, gable-roofed buildings, called the windPRESSURE, uses the time history of pressures at various tap locations measured from wind tunnel tests in conjunction with structural influence coefficients to determine the peak wind loads such as bending moments, shear forces, and displacements at various locations. The database is composed of four buildings with the same roof slope of 1:12, same width of 36.6 m, and heights of 3.7 m (in open country terrain), 5.5 m (in open country and suburban terrains), and 7.3 m (in open country terrain). The testing configurations, wind simulation, standard archiving format, distribution of the data, and the analysis of the data are discussed in detail in Ho et al. (2005), wherein seven buildings models were tested. The windPRESSURE requires inputs with regards to the building dimensions, terrain conditions, and information on the structural system such as frame locations and sizes and influence functions. An interpolation procedure is used when the dimensions of the building of interest do not match those of the buildings in the database.

The High-Rise Database Assisted Design for Reinforced Concrete Structures (HR_DAD_RC) employs a time-domain analysis that combines the time histories of the wind pressure measurements and climatological data to determine the wind-induced response of tall buildings. The methodology and procedures used by HR_DAD_RC are provided in detail in Yeo (2010). This approach accounts for the dynamic effects of wind loads on tall buildings by considering the actual vibration modes of the buildings, and does not require simplifying assumptions with respect to the mode shapes such as
assuming uncoupled linear modes. Given the time histories of the pressure measurements and climatological data, the structural engineer can calculate the building’s loads and responses. The design based on DAD accounts for both the wind and gravity loads and is carried out iteratively until the design specifications are met. The inputs required for HR_DAD_RC consists of the details of the structural members, wind-induced pressure coefficients on the external surfaces, climatological data at the location of the structure, mass and mass moment of inertial at the center of mass at each floor, mode shapes, natural frequencies, and damping ratios. Given the input parameters, HR_DAD_RC computes the peak loads and responses of the building, i.e. demand-to-capacity indexes, inter-storey drift, and top floor acceleration. This database-assisted approach is not practical for preliminary design of tall buildings due to the requirement of the external pressure coefficients as one of the inputs.

This chapter presents a brief review of the basic bluff body aerodynamics and current state of the practice of determining wind loads on tall buildings. The basic bluff body aerodynamics is discussed with respect to the along-wind, across-wind, and torsional loads influenced by separating shear layers and pressure distributions; which lay the foundation for aerodynamic wind loads on tall buildings. The basic formulation associated with the gust loading factor approach for the along-wind response and its extension to the three-dimensional condition is described. The wind load provisions in three design codes, namely NBCC 2010, ASCE 7-10, and AS/NZS 1170.2:2002 are compared primarily in terms of the definition of wind field characteristics and formulations of the gust loading factor approach. The fundamentals of two main wind tunnel testing methods for tall buildings, namely the force-balance model and pressure model tests, are briefly described. Finally, the database-assisted approach, which is an alternative to the design codes for determining the preliminary design wind loads on tall buildings, is reviewed. The NatHaz Aerodynamic Loads Database (NALD) developed at the University of Notre Dame, and the Database-Assisted Design (DAD) developed at NIST are briefly reviewed.
The Western University Aerodynamic Database (WAD) established in this study contains aerodynamic data for tall buildings that were tested in its simulated natural surroundings in the BLWTL; therefore, the interference and shielding effects are included in the corresponding test results. WAD consists of data collected from 56 different wind tunnel tests that were carried out using either the force balance or pressure model tests as described in Section 2.5.2. The establishment of the database allows the user to evaluate the preliminary design wind loads on a tall building, referred to as the target building, based on its aerodynamic and dynamic properties. An overview of the WAD methodology is summarized as a flow chart in Figure 3.1. The procedure carried out by the WAD program to estimate the wind loads on the target building begins with the user providing the geometric and dynamic properties of the building. The WAD program consists of the following two components: the reference building selection process and the wind load evaluation. A fuzzy logic inference system (Sivanandam et al. 2007; and Yen and Langari 1999) is employed in the reference building selection process to compare the geometry, terrain conditions, and surrounding building configurations of the target building with those of a candidate building in WAD. This process is repeated for every 10° around the target building, for every mirrored angle corresponding to each of four 90° quadrants, and all 56 buildings in WAD. Once a number of appropriate buildings from WAD, referred to as the reference buildings, are selected, a set of normalized aerodynamic loads are obtained. A modified three-dimensional moment gust loading factor (3D MGLF) approach is then employed to evaluate the wind loads on the target building based on each set of normalized aerodynamic loads. The maximum peak base moment over all sets of aerodynamic loads is used as the final output of the program. A Matlab®-based script program is also developed to act as an graphic interface between the user and the database.
The rest of this chapter is organized as follows: Section 3.2 presents details of the development of WAD; Section 3.3 introduces the fuzzy logic rule-based inference process and its application in the reference building selection process; the modified 3D MGLF approach for determining the preliminary design wind loads is described in Section 3.4; Section 3.5 describes the graphical user interface developed using MatLAB®; the applicable scope of WAD is stated in Section 3.6, and Section 3.7 is a summary of this chapter.
The heights of the buildings included in WAD range from 60 m to 350 m. The aspect ratios (A) of the buildings, defined as the long plan dimension, $B$, divided by the short plan dimension, $D$, range from 1.0 to 3.4. The slenderness ratios (S), defined as the height of the building, $H$, divided by $B$, range from 1.2 to 10.4. In the case where a building’s cross-section is not rectangular, it is approximated by an equivalent rectangle defined by the projected lengths along the two orthogonal primary axes of the cross-section. If a building’s plan dimensions change with height, $B$ and $D$ were evaluated as the weighted average values given by Eqs. (3.1) and (3.2) respectively

$$B = \frac{\sum_i (H_i B_i)}{\sum_i H_i} \quad \text{(3.1)}$$

$$D = \frac{\sum_i (H_i D_i)}{\sum_i H_i} \quad \text{(3.2)}$$

where $B_i$ and $D_i$ are the plan dimensions of the building at the $i^{th}$ story, and $H_i$ is the height of the building up to storey $i$. The particular form of the weighted average employed in the above two equations is used based on the consideration that the wind loads applied at higher levels on the building in general have a larger impact on the base bending moment due to the increased lever arm. Figs. 3.2 through 3.4 are the histograms for the aspect ratios, slenderness ratios, and heights of the buildings included in WAD, respectively. As shown in these figures, almost 50% of the buildings in WAD have aspect ratios between 1.0 and 1.5; over 60% of the buildings have heights between 150 and 250 m, and the slenderness ratios of the buildings distribute relatively uniformly between 1.5 and 7.0 with a few outliers. The aerodynamic properties of a target building consist of the aspect and slenderness ratios and the total height of the building, as defined in this section.
Figure 3.2: Summary of Aspect Ratios of Buildings in WAD

Figure 3.3: Summary of Slenderness Ratios of Buildings in WAD

Figure 3.4: Summary of Building’s Heights in WAD
Each building entered into WAD is associated with the following two categories of upstream exposure conditions: the far-field and near-field exposures. The far-field exposure classifies the conditions of the general terrain surrounding the building. Buildings are classified into four far-field exposure categories as summarized in Table 3.1 (ASCE 2010; Holmes 2007) by comparing the mean wind speed and turbulence intensity profiles used in the wind tunnel test with the standard profiles generated using ESDU (ESDU 2005) for the four categories. The terrain roughness lengths corresponding to these categories used in the ESDU analysis are also summarized in Table 3.1. The roughness lengths used in literature (e.g. ASCE 2010; Australian/New Zealand Standard 2011; Holmes 2007) for the suburban exposure typically vary within the range of 0.1 to 0.5 m. Therefore, two sets of profiles are generated for the suburban exposure category using roughness lengths of 0.1 and 0.5 m respectively. Note that the mean wind speed and turbulence intensity profiles employed in wind tunnel tests do not necessarily match the standard ESDU profiles. In these cases, the far-field exposure that corresponds to the closest match with the profiles employed in the tests was used. Each far-field exposure category is also tagged with a numerical value in WAD (see Table 3.1), which is used as one of the inputs in the reference building selection process.

Table 3.1: Far-Field Exposure Categories in WAD

<table>
<thead>
<tr>
<th>Far-Field Exposure</th>
<th>Terrain Characteristics</th>
<th>Roughness Length, $z_0$ (m)</th>
<th>Assigned Numerical Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth</td>
<td>Flat land surface without any noticeable obstacles and little vegetation</td>
<td>0.005</td>
<td>1</td>
</tr>
<tr>
<td>Open</td>
<td>Open country fields with little vegetation and some trees</td>
<td>0.03</td>
<td>2</td>
</tr>
<tr>
<td>Suburban</td>
<td>From open country with groups of trees and vegetation to areas with closely spaced obstructions such as suburban housing</td>
<td>0.1 and 0.5</td>
<td>3</td>
</tr>
<tr>
<td>Urban</td>
<td>Large city centers with large buildings</td>
<td>1.0</td>
<td>4</td>
</tr>
</tbody>
</table>

Each of the buildings in WAD was tested in the wind tunnel within its natural surroundings that are generally composed of models of geometrically scaled surrounding.
buildings. The near-field exposure classification is an attempt at characterizing the general sizes of the buildings surrounding the test building, generally within the radius that is modeled in the wind tunnel test (approximately 500 m). The near-field exposure categories used in WAD and corresponding numerical values assigned to these categories are summarized in Table 3.2. The rationales for the selected numerical values are further discussed in Section 3.3. The classification of each building to the near-field exposure category is more subjective than the far-field exposure category, because the categories summarized in Table 3.2 only generally define the configurations of surrounding buildings. Furthermore, accurate information about the number, orientations, sizes, and locations of the surrounding buildings is not readily available and can only be approximately inferred based on the photographs of the test model in its natural surroundings found in the final reports issued by BLWTI. The near-field exposure categories as summarized in Table 3.2 were proposed based on information in the literature concerning the interference and shielding effects of individual and multiple surrounding buildings.

English (1990) studied the shielding effect by changing the height and width of a single shielding building. The considered heights of the shielding building include 150% and 50% the height of the downstream building. The mean along-wind force on the downstream building shielded by a building with half the height was found to be closer to the along-wind force on the downstream building under the isolated condition. A reduction of the mean along-wind force was observed if the height of the shielding building is 150% that of the downstream building. This is due to the fact that the wind is blocked by the interfering building as a result of the downstream building located in the wake recirculation region of the interfering building with low velocities. Melbourne and Sharp (1976) studied the interference effect by changing the height of a single interfering building and found that the interference effect is significantly reduced if the height of the interfering building is less than 2/3 of the height of the downstream building. This is because the size of the vortices shed by the interfering building is small compared to the downstream building and the vortices do not affect the top of the target building where the load effects are larger. Saunders and Melbourne (1979) reported that the fluctuating along-wind and across-wind base moments of the downstream building significantly
increase once the interfering building’s height is greater than or equal to the height of the downstream building. It can be inferred from the above-mentioned studies that there approximately exist two transition values of the ratio between the heights of the interfering building and downstream building: one between $1/2 - 2/3$ and the other being unity.

The buildings included in WAD are usually surrounded by multiple instead of single interfering buildings. However, as described in Section 2.3.4, studies reported in the literature on the interference effect due to multiple buildings were all focused on groups of buildings (including the interfering building and downstream building) of equal height. Given the above, 50% of the downstream building’s height was selected in this study as the threshold for high shielding. An additional threshold value of 20% of the height of the downstream building was further included to separate low and moderate shielding conditions.

Table 3.2: Near-Field Exposure Categories

<table>
<thead>
<tr>
<th>Near-Field Exposure</th>
<th>General Size of Surrounding Buildings</th>
<th>Assigned Numerical Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isolated</td>
<td>No surrounding buildings in all directions</td>
<td>-1</td>
</tr>
<tr>
<td>No Shielding</td>
<td>No surrounding buildings</td>
<td>-1</td>
</tr>
<tr>
<td>Low Shielding</td>
<td>Heights of surrounding buildings are 0-20% of the height of the downstream building</td>
<td>1</td>
</tr>
<tr>
<td>Moderate Shielding</td>
<td>Heights of surrounding buildings are 20-50% of the height of the downstream building</td>
<td>2</td>
</tr>
<tr>
<td>High Shielding</td>
<td>Heights of surrounding buildings are 50-100% of the height of the downstream building</td>
<td>3</td>
</tr>
<tr>
<td>Immersed</td>
<td>Building is surrounded by buildings of similar or greater heights in all directions</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 3.3 is a summary of the number of buildings in WAD that are associated with each of the far-field and near-field exposure categories at 36 different wind attack angles. It can be seen that the majority of the buildings in the database are categorized in the suburban and urban far-field exposure categories, and most buildings are categorized as high shielding near-field exposure categories.
Table 3.3: Distribution of Buildings in Upstream Exposure Categories at Different Wind Angles in WAD

<table>
<thead>
<tr>
<th>Angle</th>
<th>Far-field Exposure</th>
<th>Near-field Exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 2 3 4 -1 1 2 3 5</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>14 9 44 45 21 30 12 39 10</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>24 20 85 95 44 60 22 78 20</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>27 18 85 94 42 60 22 80 20</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>23 18 83 100 42 61 22 79 20</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>23 18 85 98 42 59 22 81 20</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>22 18 88 96 43 55 21 85 20</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>24 19 89 92 43 55 21 85 20</td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>21 20 90 93 43 55 21 85 20</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>20 22 88 94 43 56 21 84 20</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>9 11 43 49 21 29 11 41 10</td>
<td></td>
</tr>
</tbody>
</table>

The normalized aerodynamic loads discussed in this section are used for the calculations of the peak base moments. The sign conventions for the aerodynamic base bending moments and torque of the buildings included in WAD are shown in Fig. 3.5, where $M_j (j = x, y, \text{ or } T)$ denotes the base bending moment about the $x$- or $y$-axis (i.e. following the right-hand rule) or base torque (i.e. $j = T$). Note that the $x$-axis is set to be parallel to the long plan dimension, i.e. $B$, of the building. A zero-degree wind angle is defined as the condition where the direction of the wind flow coincides with the direction of the negative $y$-axis, and the wind angle measures clockwise from the $y$-axis (see Fig. 3.5).

Figure 3.5: Sign Conventions for Base Bending Moments and Torque
The mean base moments are expressed in the following non-dimensional forms:

\[
\tilde{C}_{M_x}(\alpha) = \frac{M_x(\alpha)}{\frac{\pi}{2} \rho \bar{V}_H(\alpha)^2 BH^2}
\] (3.3)

\[
\tilde{C}_{M_y}(\alpha) = \frac{M_y(\alpha)}{\frac{\pi}{2} \rho \bar{V}_H(\alpha)^2 DH^2}
\] (3.4)

\[
\tilde{C}_{M_T}(\alpha) = \frac{M_T(\alpha)}{\frac{\pi}{2} \rho \bar{V}_H(\alpha)^2 BDH}
\] (3.5)

where \( \bar{M}_j(\alpha) \) is the mean base bending moment or torque about the \( j \)-axis for a given wind angle \( \alpha \); \( \rho \) is the air density and taken as 1.23 kg/m\(^3\), and \( \bar{V}_H(\alpha) \) is the mean hourly wind speed at the roof height of the building corresponding to the wind angle \( \alpha \). The RMS base moments are expressed in the following non-dimensional forms:

\[
\sigma_{C_{M_x}}(\alpha) = \frac{\sigma_{M_x}(\alpha)}{\frac{\pi}{2} \rho \bar{V}_H(\alpha)^2 BH^2}
\] (3.6)

\[
\sigma_{C_{M_y}}(\alpha) = \frac{\sigma_{M_y}(\alpha)}{\frac{\pi}{2} \rho \bar{V}_H(\alpha)^2 DH^2}
\] (3.7)

\[
\sigma_{C_{M_T}}(\alpha) = \frac{\sigma_{M_T}(\alpha)}{\frac{\pi}{2} \rho \bar{V}_H(\alpha)^2 BDH}
\] (3.8)

where \( \sigma_{M_j}(\alpha) \) is the RMS base bending moment or torque about the \( j \)-axis corresponding to the wind angle \( \alpha \). The non-dimensional spectra of the aerodynamic base moment and torque are defined as follows:

\[
S'_{M_j}(f_n(\alpha), \alpha) = \frac{f_{S_{M_j}}(f_n(\alpha), \alpha)}{(\sigma_{M_j}(\alpha))^2}
\] (3.9)

\[
f_n(\alpha) = \frac{f_{\sqrt{BD}}}{\bar{V}_H(\alpha)}
\] (3.10)

where \( f \) is the frequency; \( \sqrt{BD} \) is the characteristic plan dimension of the building; \( f_n(\alpha) \) is the reduced frequency corresponding to a given wind angle \( \alpha \), and \( S_{M_j}(f_n(\alpha), \alpha) \) is the
spectrum of the aerodynamic base moment and torque as a function of the reduced frequency. A set of input normalized aerodynamic loads used in the evaluation of the peak base moments consist of the mean and RMS base moment coefficients and the normalized spectral coordinates at the reduced frequencies of the building.

In using WAD to evaluate the preliminary design wind loads on a target building, it would be ideal to select reference buildings from the database whose aerodynamic properties and exposure conditions are the same as those of the target building. However, an exact match is generally not possible due to the variability of the aerodynamic characteristics and surroundings of the potential target buildings, as well as the limited number of entries included in WAD. A fuzzy rule-based inference process was therefore developed using the Matlab® Fuzzy Logic Toolbox to select the reference buildings whose aerodynamics and exposure conditions are similar to those of the target building. The fuzzy rule-based inference process is described in detail in such references as Sivanandam et al. (2007) and Yen and Langari (1999).

The fuzzy logic theory (Sivanadam et al. 2007) deals with reasoning that is approximate as opposed to being fixed and exact. The fuzzy rule-based inference process consists of four components: namely (1) fuzzy matching, (2) inference, (3) combination, and (4) defuzzification. The fuzzy matching step converts the input data into fuzzy values that are in the range of 0 to 1, representing the degree to which the input matches the fuzzy rules through membership functions. WAD compares the following four parameters of the target building and a given building in the database (referred to as the candidate building): the aspect ratio, slenderness ratio, far-field exposure condition, and near-field exposure condition. Note that the aspect and slenderness ratios of the target building must be calculated based on definitions given in Section 3.2.1, and Tables 3.1 and 3.2 must be used to classify the target building into appropriate far-field and near-field exposure categories respectively.

Let \( n \) denote the absolute difference between the numerical values of the near-field categories corresponding to the target and candidate buildings. The following
membership function (see Fig. 3.6) is used to convert \( n \) into the corresponding fuzzy value \( N \):

\[
N(n) = \begin{cases} 
1 & n \leq 0.5 \\
\frac{2-n}{1.5} & 0.5 < n \leq 2 \\
0 & n > 2 
\end{cases}
\]  

Figure 3.6: Near-field Category Membership Function

The membership function, \( N(n) \), is the trapezoidal-shaped built-in membership function in MatLab\textsuperscript{®} (Sivanandam et al. 2007). The absolute difference of the numerical values of the near-field exposure category will always be either an integer or zero; therefore, as shown in Fig. 3.6, \( N(n) \) only have three possible values: 1.0, 0.67 or zero (\( n > 2 \)). The numerical values of -1 and 5 for the isolated/no shielding and immersed near-field categories respectively as shown in Table 3.2 were selected to ensure that only candidate buildings within the respective specified near-field categories will result in \( N(n) > 0 \). Target buildings with the low shielding near-field exposure can potentially be associated with a reference buildings with the low or moderate shielding near-field exposure; target buildings with the moderate shielding near-field exposure can potentially be associated with reference buildings with the low, moderate or high shielding category, and lastly,
target buildings with a high shielding exposure category can potentially be associated with reference buildings with the moderate or high shielding exposure.

A candidate building in WAD will not be selected as a reference building if the far-field exposure category of the building does not match that of the target building. This is based on the fact that the mean wind speed and turbulence intensity profiles that were used to test each building in WAD may not have exactly matched one of the standard profiles generated by ESDU (discussed in Section 3.2.2) for each far-field category.

Let \( r_A \) denote the absolute difference between the aspect ratios of the target and candidate buildings divided by the aspect ratio of the target building; \( r_S \) is calculated in the same way as \( r_A \) but for the slenderness ratio. \( r_A \) and \( r_S \) are converted into fuzzy values according to the following membership function:

\[
D_*(r_*) = \begin{cases} 
1 & r_* \leq 0.1 \\
1 - 2 \left( \frac{r_* - 0.1}{0.1} \right)^2 & 0.1 < r_* \leq 0.15 \\
2 \left( \frac{r_* - 0.2}{0.1} \right)^2 & 0.15 < r_* \leq 0.2 \\
0 & r_* > 0.2 
\end{cases}
\]  

(3.12)

Figure 3.7: Dimensional Membership Function
where \( D.(r.) \) is the fuzzy value for the aspect ratio \( (A) \) or slenderness ratio \( (S) \) and \( \bullet \) is a generic symbol for \( A \) or \( S \). The function, \( D.(r.) \), is the \( z \)-shaped built-in membership function (see Fig. 3.7) in MatLab\textsuperscript{®} and was selected such that the fuzzy value decreases more rapidly than that of a linear membership function once the relative difference between the aspect (slenderness) ratios of the target and candidate buildings is relatively large, say greater than 15%. The control points of 0.1, 0.15 and 0.2 for \( r \) were selected based on inferences made from the literature (Sakamoto 1985; Vickery 1968). In the most conservative scenario, a 15% difference in the aspect or slenderness ratio can result in up to a 20% difference in the mean and RMS drag force coefficients.

The inference step evaluates the conclusions corresponding to individual fuzzy rules. The only fuzzy rule currently used in the reference building selection process is as follows: “If \( n < 2 \) and \( r_A < 0.2 \) and \( r_S < 0.2 \), then a candidate building is included in the combination step.” In the combination step, the conclusions from individual fuzzy rules are combined into the final fuzzy conclusion. Because only one fuzzy rule is included in the selection process, this step is ignored. For the final step, i.e. defuzzification, let \( F(x) \) define the membership function that relates the index, \( x \), that characterizes the degree of overall matching between the target and candidate buildings to a fuzzy value. Based on one of the standard defuzzification methods implemented in MatLab\textsuperscript{®}, namely the smallest of maximum (Sivanandam et al. 2007), the final matching index, \( x \), can be evaluated as \( x = F^{-1}(\Pi) \), where \( \Pi = \min\{N(n), D_A(r_A), D_S(r_S)\} \). The membership function \( F(x) \) was selected to be the MatLab\textsuperscript{®} built-in S-shaped function (see Fig. 3.8) defined as follows:

\[
F(x) = \begin{cases} 
2x^2 & 0 \leq x \leq 0.5 \\
1 - 2(x - 1)^2 & 0.5 < x < 1 \\
1 & x = 1 
\end{cases}
\]  
(3.13)
The S-shaped function was selected as opposed to a linear function to be more conservative compared with a linear function if an $x_{\text{min}} > 0.5$ is selected. The final matching index, $x$ (ranging from 0 to 1), is the value used to define the degree to which a candidate building matches the target building. Those candidate buildings satisfying $x \geq x_{\text{min}}$ are accepted as the reference buildings. Currently, $x_{\text{min}}$ is set to equal 0.5 based on the consideration that setting $x_{\text{min}}$ too high may markedly limit the number of reference buildings available for a given target building while setting $x_{\text{min}}$ too low may result in poor accuracy of the estimated wind loads. This value may be adjusted in the future if more buildings are included in the database and/or there is a need to adjust the level of accuracy in the estimated wind loads. The current value of $x_{\text{min}} = 0.5$ means that only those candidate buildings satisfying $r < 0.15$ and $n < 1.0$ may be selected as reference buildings. For a given target building, the fuzzy logic rule-based reference building selection process is executed for every 10° interval around the building and four mirrored angles corresponding to each 90° quadrant around the candidate building. For example, as shown in the example in Fig. 3.9, a wind angle of 20° for the target building is compared with angle 20°, 160°, 200°, and 340° for each candidate building in WAD.
Once the sets of normalized aerodynamic loads are obtained, the peak base moments for the target building can be evaluated. The 3D MGLF approach proposed by Zhou and Kareem (2001) was adopted as the basis for determining the design wind loads based on WAD. The application of this approach in NALD has been discussed by Kwon et al. (2008). The 3D MGLF approach applies to the responses in the along-wind, across-wind, and torsional directions for wind directions orthogonal to the \( x \) and \( y \) building axes, but is not applicable to the more general loading condition, where the combined effects of wind loads under non-orthogonal wind direction may be the critical loading case. Therefore, the approach has been slightly modified in this study to calculate the peak base bending moments in two orthogonal directions (i.e. \( x \) and \( y \) directions) and base torque corresponding to arbitrary wind attack angles. The modified 3D MGLF approach is described in the following.

By assuming the response of a tall building under the wind load to be a stationary Gaussian process, the expected peak base moment and torque for a tall building can be calculated as follows (Kwon et al. 2008):
\begin{equation}
\tilde{M}_x(\alpha) = \bar{M}_x(\alpha) + \sqrt{\left( g_B \sigma_{MB_x}(\alpha) \right)^2 + \left( g_{R_y} \sigma_{MR_x}(\alpha) \right)^2} 
\end{equation}

\begin{equation}
\tilde{M}_y(\alpha) = \bar{M}_y(\alpha) + \sqrt{\left( g_B \sigma_{MB_y}(\alpha) \right)^2 + \left( g_{R_x} \sigma_{MR_y}(\alpha) \right)^2} 
\end{equation}

\begin{equation}
\tilde{M}_T(\alpha) = \bar{M}_T(\alpha) + \sqrt{\left( g_B \sigma_{MB_T}(\alpha) \right)^2 + \left( g_{R_T} \sigma_{MR_T}(\alpha) \right)^2} 
\end{equation}

\begin{equation}
g_{R_j} = \sqrt{2\ln(f_jT_d)} + \frac{0.5772}{\sqrt{2\ln(f_jT_d)}} 
\end{equation}

where \( \tilde{M}_j(\alpha) \) (\( j = x, y \) or \( T \)) is the peak base moment or torque corresponding to wind angle \( \alpha \); \( \bar{M}_j(\alpha) \), \( g_B \sigma_{MB_j}(\alpha) \), and \( g_{R_j} \sigma_{MR_j}(\alpha) \) are the mean, background dynamic, and resonant dynamic components of the peak base moments respectively; \( g_B \) is the background peak factor and set to 3.4 in WAD (Zhou et al. 2003), and \( g_{R_j} \) is the resonant peak factor along the \( j \)-axis. The mean base moments can be further written as:

\begin{equation}
\bar{M}_x(\alpha) = 0.5\rho(\bar{V}_H(\alpha))^2BH^2\bar{C}_{Mx}(\alpha) 
\end{equation}

\begin{equation}
\bar{M}_y(\alpha) = 0.5\rho(\bar{V}_H(\alpha))^2DH^2\bar{C}_{My}(\alpha) 
\end{equation}

\begin{equation}
\bar{M}_T(\alpha) = 0.5\rho(\bar{V}_H(\alpha))^2BDH\bar{C}_{MT}(\alpha) 
\end{equation}

where \( \bar{C}_{Mj}(\alpha) \) is the dimensionless mean moment or torque coefficient as defined in Eqs. (3.3) to (3.5). The quantities \( \sigma_{MB_j}(\alpha) \) are given by:

\begin{equation}
\sigma_{MB_x}(\alpha) = 0.5\rho(\bar{V}_H(\alpha))^2BH^2\bar{C}_{\sigma Mx}(\alpha) 
\end{equation}

\begin{equation}
\sigma_{MB_y}(\alpha) = 0.5\rho(\bar{V}_H(\alpha))^2DH^2\bar{C}_{\sigma My}(\alpha) 
\end{equation}

\begin{equation}
\sigma_{MB_T}(\alpha) = 0.5\rho(\bar{V}_H(\alpha))^2BDH\bar{C}_{\sigma MT}(\alpha) 
\end{equation}
where \( C_{\sigma_M} \) is the dimensionless coefficient for the RMS fluctuating aerodynamic moment about the \( j \)-axis of the building as defined in Eqs. (3.6) to (3.8).

Based on the modal analysis and random vibration theory (Kwon et al. 2008) and the assumption of uncoupled and linear fundamental mode shapes in all directions, \( \sigma_{MR_j}(\alpha) \) can be calculated using Eqs. (3.24) to (3.26). Detailed derivations of Eqs. (3.24) to (3.26) are given in Appendix A. Note that the mode shape correction factors (Zhou et al. 2002) can be used for the impact of non-linear mode shapes on the base moment and torque. Mode shape corrections are ignored in this study because the analysis carried out by Zhou et al. (2002) suggests that it is conservative, in most cases, to ignore the mode shape correction factor: the correction factors applied to the resonant components of the base bending moments and base torque reported by Zhou et al. (2002) range from 0.84 to 1.06 and from 0.55 to 1.07, respectively.

\[
\sigma_{MR_x}(\alpha) = \sigma_{MB_x}(\alpha) \sqrt{\frac{\pi}{4 \bar{\xi}_y}} S'_{M_x}(f_{nx}(\alpha), \alpha) \tag{3.24}
\]
\[
\sigma_{MR_y}(\alpha) = \sigma_{MB_y}(\alpha) \sqrt{\frac{\pi}{4 \bar{\xi}_x}} S'_{M_y}(f_{nx}(\alpha), \alpha) \tag{3.25}
\]
\[
\sigma_{MR_T}(\alpha) = \sigma_{MB_T}(\alpha) \sqrt{\frac{\pi}{4 \bar{\xi}_T}} S'_{M_T}(f_{nt}(\alpha), \alpha) \tag{3.26}
\]

where \( S'_j(f_n(\alpha), \alpha) \) is the non-dimensional spectrum of the aerodynamic base moment about the \( j \)-axis as a function of the reduced frequency, \( f_n \), as defined in Eq. (3.10), and \( f_j \) and \( \bar{\xi}_j \) are the fundamental natural frequency and critical modal damping ratio, respectively, of the building along the \( j \)-axis (x, y, or T).

To evaluate \( \bar{M}_j(\alpha) \) for a target building based on WAD, \( \bar{C}_{M_j}(\alpha) \), \( C_{\sigma_M} \) and \( S'_j(f_n(\alpha), \alpha) \) of the target building are assumed to equal a given set of normalized aerodynamic loads obtained from the reference building selection process. Substituting the values of \( B, D, H, \bar{V}_H(\alpha), \) and the dynamic properties, i.e. \( f_j \) and \( \bar{\xi}_j \), associated with the target building into Eqs. (3.14) - (3.26) then results in estimated values of \( \bar{M}_j(\alpha) \) as well
as $\bar{M}_j(\alpha), g_B\sigma_{MB_j}(\alpha)$ and $g_R\sigma_{MR_j}(\alpha)$ for the target building. The final estimated $\bar{M}_j(\alpha)$ output from WAD for the target building is the maximum value over all sets of normalized aerodynamic load inputs.

Once the mean, background dynamic and resonant dynamic base moments and torque for a target building have been estimated, they can subsequently be distributed along the height of the building as equivalent static wind loads (ESWL) according to Eqs. (3.27) through (3.37). The mean and background dynamic components of the base bending moments can be distributed according to the mean wind profile, assuming a power law relationship with height (Kareem and Zhou 2003). The resonant dynamic components of the base bending moments and torque are distributed according to the building’s mode shapes and distributions of the mass and mass moment of inertia along the height (Kareem and Zhou 2003). Note that the term $C_j$ defined in Eq. (3.37) is used to linearize the contributions to the overall ESWL from the dynamic components of the base bending moments and torque.

For sway directions:

$$\hat{P}_j(z, \alpha) = \bar{P}_j(z, \alpha) + P_{B_j}(z, \alpha) + P_{R_j}(z, \alpha), j = x, y, T \quad (3.27)$$

$$\bar{P}_x(z, \alpha) = \bar{M}_y(\alpha) \left( \frac{2+2\beta}{H^2} \left( \frac{x}{H} \right) \right)^{2\beta} H_i \quad (3.28)$$

$$\bar{P}_y(z, \alpha) = \bar{M}_x(\alpha) \left( \frac{2+2\beta}{H^2} \left( \frac{x}{H} \right) \right)^{2\beta} H_i \quad (3.29)$$

$$P_{B_x}(z, \alpha) = C_y \left( g_B\sigma_{MB_y}(\alpha) \right)^2 \left( \frac{2+2\beta}{H^2} \left( \frac{x}{H} \right) \right)^{2\beta} H_i \quad (3.30)$$

$$P_{B_y}(z, \alpha) = C_x \left( g_B\sigma_{MB_x}(\alpha) \right)^2 \left( \frac{2+2\beta}{H^2} \left( \frac{x}{H} \right) \right)^{2\beta} H_i \quad (3.31)$$

$$P_{R_x}(z, \alpha) = C_y \left( g_R\sigma_{MR_y}(\alpha) \right)^2 \frac{m(z)\phi_x(z)}{\sum m(z)z\phi_x(z)} \quad (3.32)$$
\[ P_{R_y}(z, \alpha) = C_x \left( g_R \sigma_{MR_x}(\alpha) \right)^2 \frac{m(z) \phi_y(z)}{\sum m(z) z \phi_y(z)} \]  

(3.33)

For torsion:

\[ \bar{P}_T(z, \alpha) = \bar{M}_T(\alpha) \frac{(1+2\beta)}{H} \left( \frac{z}{H} \right)^2 H_i \]  

(3.34)

\[ P_{B_T}(z, \alpha) = C_T \left( g_B \sigma_{MB_T}(\alpha) \right)^2 \frac{(1+2\beta)}{H} \left( \frac{z}{H} \right)^2 H_i \]  

(3.35)

\[ P_{R_T}(z, \alpha) = C_T \left( g_R \sigma_{MR_T}(\alpha) \right)^2 \frac{I(z) \phi_T(z)}{\sum I(z) \phi_T(z)} \]  

(3.36)

\[ C_j = \sqrt{\frac{1}{\left( g_B \sigma_{MB_j}(\alpha) \right)^2 + \left( g_R \sigma_{MR_j}(\alpha) \right)^2}}, \quad j = x, y, T \]  

(3.37)

In Eqs. (3.27) to (3.36), \( \bar{P}_j(z, \alpha), \bar{P}_j(z, \alpha), P_{B_j}(z, \alpha), \) and \( P_{R_j}(z, \alpha) \) are the peak, mean, background, and resonant ESWL along the x-, y-, or torsional axis respectively at height \( z \); \( m(z) \) is the mass per unit height; \( I(z) \) is the mass moment of inertia per unit height; \( H_i \) is the height of storey \( i \); \( \phi_j(z) \) is the mode shape that is assumed to be linear with height in all directions (i.e. \( \phi_j(z) = z/H \)) (note that mode shapes are assumed to be uncoupled), and \( \beta \) is the power-law exponent of the mean wind speed profile. It should be noted that the equivalent static wind loads are not calculated in WAD and will need to be determined by the user.

For serviceability design, the acceleration response is of interest. Using random vibration analysis and adopting the same assumptions involved in the modified 3D MGLF approach, one can calculate the peak acceleration at the center of mass as follows (Kwon and Kareem 2013):

\[ \hat{a}_x(z, \alpha) = \frac{M_{R_y}(\alpha)}{HM_x^2} \phi_x(z) \]  

(3.38)

\[ \hat{a}_y(z, \alpha) = \frac{M_{R_x}(\alpha)}{HM_y^2} \phi_y(z) \]  

(3.39)
where $\hat{a}_j(z, \alpha)$ is the peak acceleration along the $x$ and $y$ axis at height $z$ at wind attack angle $\alpha$, and $M_j^*$ is the generalized mass corresponding to the fundamental mode along the $j$-axis ($x$ or $y$). Because the acceleration is only evaluated at the center of mass in WAD, the base torque is not included in the acceleration calculation.

**WAD Graphical User Interface**

The following sections present the graphical user interface developed for the application of the WAD-based wind load calculations. Figure 3.10 and 3.11 are screenshots of the primary MatLab®-based input user interfaces developed for WAD with a set of hypothetical input parameters. Screenshots of other user interfaces associated with WAD are shown in Appendix B. The input required to estimate the preliminary design wind loads on a target building are divided into the following parts: the geometric and dynamic properties of the building, the design wind speed, and the exposure criteria. The user is provided with instructions and guidelines for the appropriate input through the “Input Guide” pushbutton.

![Figure 3.10: Main Graphical User Interface for WAD](image)
Within the “Dimensional” rectangle in the user interface, the user is required to input the aspect and slenderness ratios and the overall height of the target building as defined in Section 3.2.1. The user has the option of entering the height of the building either in S.I. or U.S. customary units. The dynamic properties of the target building include the frequencies for the fundamental modes that are dominant in -x-, -y-, and torsional axes, respectively; which are entered in the “Structural Dynamic Properties” rectangle in the user interface. The user will also have the option of including or excluding the acceleration calculations. If the acceleration estimation option is selected, the user also needs to input the average bulk density of the building. The evaluation of the fundamental frequencies is not provided in WAD, and therefore must be carried out by the user, for example, from an eigenvalue analysis of the building. However, if such an analysis is not feasible due to insufficient information, approximate approaches and/or empirical formulas (e.g. Rayleigh’s method) can be used to estimate the fundamental frequencies. These approaches are widely reported in literature (e.g. Goel and Chopra 1997, 1998; Holmes 2007; NRCC 2010; Saatcioglu and Humar 2013). Attention, however, must be paid to the limitations and applicability of these approaches. If there is no suitable approximate approach to estimate the frequency associated with the torsion-dominant fundamental mode, the user can simply leave the input for the torsional frequency blank, and the torque responses will not be calculated in WAD.
**Upstream Exposure Criteria**

The upstream exposures of the target building are input for eight equally divided quadrants around the building. The input for each quadrant is carried out by pressing the “Define Exposure” pushbutton which opens the user interface shown in Figure 3.11. The input for each quadrant includes the far-field and near-field exposure categories, as well as the design wind speed at the roof height corresponding to the far-field exposure category. WAD further allows the user to input two design wind speeds (corresponding to two different return periods) for a given quadrant for estimating the preliminary design wind loads and accelerations, respectively. The averaging time for the design wind speed can be entered as either one hour, 10 minutes, or 3 seconds using the pull-down menu in Fig. 3.10. Wind speeds entered as either the 10-minute mean or 3-second gust will be converted to the mean hourly wind speed according to the Durst Curve (Durst 1960). The far-field and near-field exposure categories for the target building can be determined through observation using satellite mapping software such as Google Earth.

After entering all of the input parameters, the user can press the “Calculate Wind Loads” pushbutton to obtain the output. Figure 3.12 is a screenshot of the output from WAD upon completion of the analysis. The key output consists of the following three components: results of the reference building selection process, peak base bending moments and torque, and peak accelerations.
Reference Building Selection Process Output

The reference building selection results panel displays the total number of reference buildings selected for the target building, the maximum and minimum values of the matching index $x$ indicating the degree to which the reference building matches the target building, and the total number of wind angles at which no reference building is selected.

Peak Base Moments and Accelerations Response

The base moment output provides the maximum estimated peak base bending moments and torque over all wind attack angles around the building and among all reference buildings, including the corresponding means, background dynamic and resonant dynamic components. The output also includes the plots depicting the distributions of the mean, background, and resonant moments with respect to the wind angle (see example shown in Figure 3.13) and the angle at which the maximum moments occur. It should be noted that Fig. 3.13 exemplifies a hypothetical scenario and may not be representative of
realistic situations. For example, caution would be taken with the unusual drop in the peak base moment at 90° and 270°. The assessment and interpretation of the output is further discussed in Chapter 4. If the building selection process does not yield a reference building for the analysis, the user may refer to the moment vs. wind angle plots to determine the particular angles for which no reference buildings were selected. Supplementary to the wind loads calculated using the WAD-based procedure, peak moments calculated using the dynamic procedure specified in NBCC 2010 are also available to the user for comparison. The peak acceleration output provides the peak accelerations along the x and y–axis of the building at the center of mass and top of the building.

![Figure 3.13: Example of Peak Base Moment (x-axis) vs. Wind Angle Plot Output in WAD](image)

The estimated peak base moments and their corresponding mean, background dynamic, and resonant dynamic components can be used to distribute along the height of the building as equivalent static wind loads following the formulations presented in Section 3.4.2. It is recommended that the following three scenarios be considered in evaluating the equivalent static wind loads, namely the peak base bending moment about the x-axis reaching the maximum value, the peak base bending moment about the y-axis reaching the maximum value, and the peak torque reaching the maximum value.
The application of WAD is suited for rectangular slender tall buildings with relatively low natural frequencies, e.g. below 0.25 Hz, but is not suited for circular buildings or buildings of unusual shapes that cannot be adequately simplified as rectangular cylinders. The near-field categories in WAD are quite general. Caution should be exercised for cases with nearby isolated individual buildings that can potentially amplify the wind responses. This generally occurs in cities with developing skylines. Caution should also be used for buildings subjected to reduced frequencies that are close to the Strouhal number. Because the modified 3D MGLF approach employed in WAD assumes linear and uncoupled fundamental modes, it is not recommended to apply WAD to estimate the preliminary design wind loads for buildings that have highly non-linear and/or coupled fundamental modes. Attention should also be given to highly flexible structures that are prone to negative aerodynamic damping.

This chapter describes the detailed development of the Western University Aerodynamic Database (WAD) of tall buildings. The data stored in WAD include the overall heights, aspect ratios, slenderness ratios, far-field and near-field exposure categories, non-dimensional aerodynamic base moment coefficients, and non-dimensional spectrums of the base moments for 56 tall buildings that have been tested in BLWTL at Western University. To estimate the preliminary design wind loads on a target building, reference buildings whose geometric and aerodynamic properties are similar to those of the target building are selected from WAD. A fuzzy theory-based process for selecting the reference building is developed and presented in this chapter. The four parameters involved in the selection include the aspect ratio, slenderness ratio, far-field exposure category, and near-field exposure category. Furthermore, a so-called modified 3D MGLF approach is proposed to evaluate the peak base moments and torque in the target building based on the aerodynamic information associated with the reference building. The formulations for computing the equivalent static wind loads and accelerations from the estimated peak base moments and torque are presented. Finally, the MatLab®-based graphical user interface associated with WAD as well as the scope of WAD is described.
The objective of the work reported in this chapter was to validate the WAD-based procedure for evaluating wind-induced responses of tall buildings and compare the accuracy of the procedure with that of other methods such as the design codes and NALD. An illustration of the reference building selection process in WAD is also presented. To this end, the buildings included in WAD were considered as the “test specimens” on which the validation and comparison are based. The advantage of such an approach is that all of the buildings in WAD have already been tested in the wind tunnel, with the corresponding test results readily available as the benchmark values for validation and comparison.

For each of the 56 buildings included in WAD, the reference building selection process as described in Section 3.3 was carried out to identify the corresponding reference building(s) from the other 55 buildings in WAD. This resulted in 36 buildings that are each associated with at least one reference building; no reference buildings were identified for the other 20 buildings, which imply that the WAD-based procedure cannot be applied to these buildings. Therefore, a total of 36 buildings in WAD were included in the subsequent analyses. The heights of the 36 buildings range from 67 m to 322 m; the aspect and slenderness ratios range from 1.03 to 3.41 and from 1.55 to 7.10 respectively, and the natural frequencies of the fundamental vibration modes of the buildings range from 0.12 to 0.7 Hz. The geometric properties of each of the 36 buildings are tabulated in Appendix C.

The reference building selection process is illustrated using the normalized aerodynamic loads from all remaining 55 buildings in WAD including the buildings that were selected as reference buildings for a given target building. Additionally, the peak base moments were also calculated following the procedures described in Section 3.4. In addition, the wind provisions in three design codes, namely ASCE7-10, NBCC 2010 and AS/NZS 1170.2: 2011, as well as the NatHaZ Aerodynamic Load Database (NALD) were
employed to evaluate the wind responses of the 36 buildings, which were then compared with those evaluated from the WAD-based procedure. It should be noted that many of the 36 buildings are outside the scope of the design codes (see Section 2.5.1.4); however, for the interest of the validation analysis the wind loads were evaluated for comparison purposes.

This chapter is organized as follows. Section 4.2 presents details involved in various approaches (i.e. WAD-based procedure, design codes and NALD) used to evaluate the wind responses of the selected 36 buildings; Section 4.3 presents illustrations of the reference building selection process using three examples; Section 4.4 presents the validation analysis results and the comparison between the results corresponding to different approaches, and the summary of this chapter is presented Section 4.5.

The analysis procedures used in the force-balance and pressure model tests for determining the wind responses of a tall building are detailed in Ho and Jeong (2008) and Ho et al. (1999) respectively. The maximum peak base moments over all wind angles were obtained from the wind tunnel analysis result and used for the validation and comparison. The peak resultant acceleration at the roof height from the wind tunnel analysis, \( \hat{a}^{WT}(H, \alpha) \), corresponding to a wind angle \( \alpha \) is calculated using the following approximate expression:

\[
\sqrt{\left(\hat{a}_x^{WT}(H, \alpha)\right)^2 + \left(\hat{a}_y^{WT}(H, \alpha)\right)^2},
\]

where \( \hat{a}_x^{WT}(H, \alpha) \) and \( \hat{a}_y^{WT}(H, \alpha) \) are the peak accelerations along the \( x \)- and \( y \)-axes, respectively. Note that \( \hat{a}_x^{WT}(H, \alpha) \) (\( \hat{a}_y^{WT}(H, \alpha) \)) is obtained as the square root of the sum of squares (SRSS) of the \( x \)-axis (\( y \)-axis) modal peak accelerations corresponding to the three fundamental modes. The maximum of \( \hat{a}^{WT}(H, \alpha) \) over all wind angles was used for the comparison.

Wind loads and response corresponding to the design codes, NALD, and WAD were calculated by simplifying each building as a rectangular cylinder with uniform projected weighted average plan dimensions along the height of the building defined in Section 3.2.1. The estimated average bulk densities (\( \rho_B \)) of the buildings for the acceleration
calculations are tabulated in Appendix C. The natural frequencies and damping ratios for the first three fundamental modes used in the analysis are also tabulated in Appendix C. The peak base moments were calculated as the moments about the $x$, $y$, and torsional axes, following the same sign convention as defined in Section 3.2.3.

The normalizing wind speeds used in the wind tunnel testing, WAD and NALD, and the reference wind speeds used in the three selected design codes are referenced at different heights corresponding to different exposures. To make the validation and comparison meaningful, the design wind speeds used in the above-mentioned approaches therefore must be made consistent with each other. The aerodynamic forces measured from the wind tunnel testing are normalized by the mean hourly wind speed at the gradient height at approximately 500 m above ground. Wind speeds used to normalize the aerodynamic loads in both NALD and WAD are referenced to the roof height of the building at their respective exposure categories. Lastly, the design wind speeds used in the three design codes are referenced to 10 m above ground in the open country terrain, but with different averaging times (one hour for NBCC 2010 and 3 seconds for ASCE7-10 and AS/NZS 1170.2:2011).

The mean hourly roof height wind speed for the WAD-based procedure was obtained from the mean hourly gradient height wind speed and normalized wind velocity at the roof height, both of which are reported as part of the wind tunnel test results. The mean hourly gradient height wind speeds were converted to the mean hourly wind speed at 10 m above ground corresponding to the open country terrain using the ESDU model (ESDU 2005) with an assumed terrain roughness length of 0.03m (ASCE 2010). The mean hourly wind speeds are multiplied by a factor of 1.53, obtained from the Durst Curve (Durst 1960), to be converted to the 3-second gust wind speeds. NALD converts the 10 m wind speed in open country into the roof height wind speed associated with the building’s particular terrain category using the corresponding power-law profile specified in ASCE7-10. Although wind speeds with different return periods are typically used to calculate the peak base moments (i.e. ultimate limit states) and accelerations (serviceability limit states), for simplicity, a single design wind speed was used for the
calculation of both the base moments and accelerations for a given building. This is considered adequate for the purpose of this study.

The peak base moments and the peak acceleration at the top of the building were evaluated according to the modified three-dimensional moment gust loading factor procedures described in Section 3.4. The base moment coefficients and the normalized base moment spectrums were obtained from the reference buildings corresponding to each of the 36 buildings considered. For a given building, the peak base moments and acceleration were calculated for wind attacking angles at every 10° around the building. When calculating the resonant dynamic base moment, if the reduced frequency of the target building is outside the range of the normalized spectrum of the base moments of the reference building, the last spectral value is used as a conservative estimation. The maximum peak base moments and peak resultant acceleration over all wind angles and all reference buildings (if there are more than one reference building selected for the target building) were used for the validation and comparison. Note that the peak resultant acceleration corresponding to a given wind angle \( \alpha \) and a given reference building was approximately evaluated as \( \sqrt{\left(\hat{a}_x(H, \alpha)\right)^2 + \left(\hat{a}_y(H, \alpha)\right)^2} \), where \( \hat{a}_x(H, \alpha) \) and \( \hat{a}_y(H, \alpha) \) are the peak accelerations along the \( x \)- and \( y \)-axes at the center of mass and the top of the building calculated according to Eqs. (3.38) and (3.39), respectively.

The three design codes, i.e. ASCE 7-10, NBCC 2010, and AS/NZS 1170.2:2011, follow the gust loading factor approach, in which the peak along-wind pressure is equal to the mean along-wind pressure multiplied by the gust loading factor. For each of the 36 buildings considered, the along-wind loads were calculated for the conditions where the wind is oriented parallel and perpendicular to the longer plan dimension of the building, i.e. wind angles of 0°, 90°, 180°, and 270° (see Fig. 3.5). The maximum values of the base bending moments about the \( x \)- and \( y \)-axes corresponding to these wind angles were used in the comparison. For consistency, all correction factors (see Table 2.2) for the
design wind velocity pressure except the terrain factor, were set to unity. The effective pressure coefficients for the windward ($C_{P,WW}$) and leeward ($C_{P, LW}$) walls were set to equal 0.8 and -0.5 respectively; which is consistent among all three codes. It should be noted that a constant terrain factor, namely the terrain factor at the equivalent height ($\bar{z}$) of the building, was used in calculating the pressure distribution of the leeward wall according to the three codes: $\bar{z} = H$ in ASCE 7-10 and AS/NZS 1170.2:2011, and $\bar{z} = H/2$ in NBCC 2010. The following general expression was used to calculate the peak along-wind base bending moment, $\bar{M}_{Code}$, according to the design codes:

$$\bar{M}_{Code} = \Delta HW G \sum_{k=1}^{100} k\Delta H \left[ (q_{des,WW}(k\Delta H)C_{P,WW}) - (q_{des,LW}(k\Delta H)C_{P, LW}) \right]$$ (4.1)

where $q_{des,WW}(k\Delta H)$ and $q_{des,LW}(k\Delta H)$ are the design wind velocity pressures for the windward and leeward walls, respectively, at the elevation of $k\Delta H$; $\Delta H = H/100$; $G$ is either the gust loading factor or the gust effect factor depending on the averaging time of the basic wind speed (see Section 2.5.1.2), and $W = B$ or $D$ for wind acting perpendicular or parallel to the longer dimension of the building, respectively, as defined in Section 3.2.1.

The partial loading cases (see Figs. 4.1 to 4.3) specified in NBCC 2010 were used to evaluate the peak base torque corresponding to NBCC 2010. The base torque was calculated with respect to the center of geometry, and set to equal the maximum value of the torques corresponding to the loading cases in Figs. 4.1 to 4.3. The wind load cases (see Figs. 4.4 to 4.6) specified in ASCE 7-10 were used to evaluate the peak base torque corresponding to ASCE 7-10. For the loading cases shown in Fig. 4.4 and 4.5, the torque at a given elevation was calculated as the resultant force multiplied by an eccentricity, $e$, evaluated as follows:

$$e = \frac{0.15b + 1.7l_{\bar{z}}(g_B + 0.15b)^2 + (g_R e_R)^2}{1 + 1.7l_{\bar{z}}(g_B + 0.15b)^2 + (g_R R)^2}$$ (4.2)

where $b$ is the plan dimension perpendicular to the direction of the force; $e_R$ is the distance between the elastic shear center and the center of mass, and is assumed to be zero, and $l_{\bar{z}}, g_B, R, g_R,$ and $R$ are defined in Section 2.5.1.2. For the loading case shown
in Fig. 4.6, the torque at a given elevation was calculated as the sum of the resultant forces in the two directions, each multiplied by the corresponding eccentricity calculated from Eq. (4.2). The maximum value of the base torques corresponding to the loading cases in Figs. 4.4 to 4.6 was selected as the base torque associated with ASCE 7-10. Partial loading cases or procedures for calculating the base torque are not provided in AS/NZS 1170.2: 2011 and therefore the base torque for AS/NZS 1170.2:2011 was not calculated.

Figure 4.1: NBCC 2010 Partial Loading Case B for Torsion Wind Acting
Perpendicular to D

Figure 4.2: NBCC 2010 Partial Loading Case B for Torsion Wind Acting
Perpendicular to B
Figure 4.3: NBCC 2010 Partial Loading Case D for Torsion

Figure 4.4: ASCE 7-10 Wind Load Case 2 for Torsion Wind Acting Perpendicular to D

Figure 4.5: ASCE 7-10 Wind Load Case 2 for Torsion Wind Acting Perpendicular to B
The peak accelerations corresponding to the design codes were calculated according to the equations given in Section 2.5.1.3. According to NBCC 2010, the calculation of the peak along-wind acceleration involves the peak displacement ($\Delta$) at the top of the building. Ideally, the peak displacement should be calculated from a static analysis of the building subjected to the peak along-wind loads evaluated according to NBCC 2010. However, it is not feasible to carry out such an analysis in this study due to very limited information about the structural systems of the buildings considered. Therefore, the peak displacement was approximately evaluated as

$$
\Delta = \frac{\Delta_{NBCC}}{K^*}
$$

(4.3)

where $K^*$ is the generalized stiffness associated with the fundamental sway mode in the along-wind direction. Implicit in the above equation is the assumption that the along-wind deflection of the building is approximated by the fundamental linear sway mode in the along-wind direction. The final peak acceleration corresponding to NBCC 2010 for a given wind direction was calculated as the resultant of the along-wind peak acceleration ($\hat{a}_{AL}(H, \alpha)$) and across-wind peak acceleration ($\hat{a}_{AC}(H, \alpha)$) as

$$
\sqrt{(\hat{a}_{AL}(H, \alpha))^2 + (\hat{a}_{AC}(H, \alpha))^2}
$$

where $\hat{a}_{AL}(H, \alpha)$ and $\hat{a}_{AC}(H, \alpha)$ were calculated from Eqs. (2.17) and (2.19) respectively.
As described in Section 2.5.1, ASCE 7-10 includes equations to estimate the along-wind peak acceleration, but does not provide equations to estimate the across-wind peak acceleration. Although AS/NZS 1170.2:2011 includes equations for estimating both the along-wind and across-wind peak accelerations, the equation for the across-wind acceleration is limited to the building’s aspect and slenderness ratios and cannot be applied to the majority of the 36 buildings considered in this study. Given the above, only the along-wind peak accelerations were calculated according to ASCE 7-10 and AS/NZS 1170.2:2011. For all three design codes, the maximum value of the peak accelerations corresponding to wind angles of 0°, 90°, 180°, and 270° was used in the subsequent comparison.

The exposure categories in accordance with the design code specifications associated with the buildings considered were mapped from the far-field exposure categories assigned to the same buildings in accordance with the criteria adopted in WAD (i.e. Table 2.3). The following mapping criteria were used: the open water exposure category was assigned to buildings in the smooth far-field exposure category; the open country exposure category was assigned to buildings in the open far-field exposure category, and suburban and urban exposure categories were assigned to buildings in the suburban and urban far-field exposure categories, respectively. However, ASCE 7-10 does not have provisions for an urban exposure category; therefore, the suburban exposure category was used instead. For NBCC 2010, terrain factors for open water and urban exposures are not provided; therefore the open country category was used in place of open water category and suburban category was used in place of urban category.

Based on NALD, wind loads were calculated for the wind oriented parallel and perpendicular to the longer plan dimension of each of the 36 buildings considered, i.e. wind angles of 0°, 90°, 180°, and 270°. The aerodynamic data stored in NALD at present are corresponding to the open and urban exposure categories specified in ASCE 7-98 (ASCE 1998) only. Therefore, buildings classified to the smooth and open far-field exposure categories in WAD were assigned the ASCE 7-98 open country exposure in NALD, and buildings classified to the urban far-field exposure category in WAD were
assigned the ASCE 7-98 urban exposure in NALD. If a given building in WAD belongs to the suburban exposure category, the peak base moments and accelerations for the building evaluated based on NALD were calculated as the average values of those corresponding to the open country and urban exposure conditions in NALD. Implicit in the NALD-based analysis is that there are no surrounding buildings to the building for which the wind responses are evaluated (i.e. the target building).

The peak base bending moments and torque are evaluated according to Eq. (2.5) in NALD. The mean component of the along-wind base moment is calculated according to the ASCE 7-98 provisions; which is the same as the corresponding ASCE 7-10 provisions as described in Section 2.5.1, except that ASCE 7-10 does not have provisions for urban exposure. The mean components of the across-wind and torsional moments of the building are assumed to equal zero. To calculate the background and resonant components of the RMS dynamic base moments, the RMS aerodynamic base moment coefficients and the normalized spectrums of the aerodynamic base moments need to be obtained from a reference building stored in NALD. It should be noted that there are a limited number of reference buildings stored in NALD, and no provisions for selecting a reference building are provided, if the geometry of the target building does not exactly match that of the reference buildings. In this study, the reference building was selected such that the proportions of $H:B:D$ of the reference building are closest to those of the target building. In situations where the reduced frequency of the target building is outside the range of the normalized spectrum of the base bending moment or torque, the last spectral value is used as a conservative estimation. The peak resultant acceleration for a given wind direction was calculated as $\sqrt{\left(\hat{a}_{AL}(H, \alpha)\right)^2 + \left(\hat{a}_{AC}(H, \alpha)\right)^2}$, where $\hat{a}_{AL}(H, \alpha)$ and $\hat{a}_{AC}(H, \alpha)$ are the along-wind and across-wind peak accelerations calculated using Eqs. (3.38) and (3.39), respectively.

The maximum of the peak base moments (about the $x$ and $y$-axes) and torque and the peak resultant accelerations corresponding to the four wind angles were used for the comparison.
Once reference buildings have been selected for an angle, the mean and RMS base moment coefficients and normalized spectral value of the base moment from each reference building are obtained from database; which are referred to as the input and are used to evaluate the peak base moments. A target building considered in the validation study can have up to 220 (i.e. 55 buildings and 4 angles) sets of data for each angle. The following sections discuss the comparisons of the input parameters and the peak base moment evaluated using data from each remaining 55 buildings in WAD including the buildings that were selected as reference buildings for a particular wind angle. Ideally, because the building selection process compares the aerodynamic characteristics between two buildings, the normalized aerodynamic loads should be similar between reference buildings; however scatter is expected. As mentioned in Section 3.2.2, the mean wind speed and turbulence intensity profiles used to test a building might not exactly match one of the standard profiles generated by ESDU that is used to categorize a building for the far-field exposure. Therefore the closest match between the test profile and a standard profile is used to categorize the building. Additionally, the near-field exposure for each building is inferred based on the photographs taken of the test building in its natural surroundings. It is thus expected that there will be differences in the normalized aerodynamic data between reference buildings.

Buildings #9, #21, and #25 were selected to illustrate the reference building selection process at wind angles of 30°, 280°, and 230° respectively. Building #9, #21, and #25, with heights of 169 m, 145 m, and 138 m respectively, have aspect and slenderness ratios of 1.35 and 5.60, 1.29 and 4.93, and 1.64 and 3.36 respectively. All three buildings are classified as the urban and high shielding for the far-field and near-field exposures respectively at the specified wind angles. Table 4.1 is a summary of the aerodynamic and dynamic properties of the buildings considered and presents the peak base moments obtained from WAD and the wind tunnel analysis. Figures 4.7 to 4.9 present the
comparisons made of the collected input values and the outputs evaluated using data from each of the remaining 55 buildings in WAD at the specified wind angles for the three buildings considered. The points within the same outlined shape that corresponds to a building number in the figures for the input parameters are the points that were used to calculate the peak base moments. The points within the circles in the figures for the output (i.e. peak base moment) is the final output value, and the horizontal lines in Figs. 4.7a), 4.8a) and 4.9a) are the peak base moments from the wind tunnel tests for the particular angles of interest.

<table>
<thead>
<tr>
<th>Building Number</th>
<th>Aerodynamic Properties</th>
<th>Dynamic Properties</th>
<th>Peak Base Moment</th>
</tr>
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<tr>
<td></td>
<td>Height (m)</td>
<td>Natural Frequency (Hz)</td>
<td>Wind angle (degree)</td>
</tr>
<tr>
<td>9</td>
<td>169</td>
<td>0.23</td>
<td>30</td>
</tr>
<tr>
<td>21</td>
<td>138</td>
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<td>230</td>
</tr>
<tr>
<td></td>
<td>Aspect Ratio</td>
<td>Damping Ratio</td>
<td>Response Axis</td>
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<tr>
<td></td>
<td>1.35</td>
<td>0.02</td>
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</tr>
<tr>
<td></td>
<td>1.64</td>
<td></td>
<td>y-axis</td>
</tr>
<tr>
<td></td>
<td>1.35</td>
<td></td>
<td>x-axis</td>
</tr>
<tr>
<td></td>
<td>Slenderness Ratio</td>
<td></td>
<td>WAD Output Peak Base Moment (MNm)</td>
</tr>
<tr>
<td></td>
<td>5.60</td>
<td></td>
<td>359</td>
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<td></td>
<td>Near-Field Exposure</td>
<td></td>
<td>Wind Tunnel Peak Base Moment (MNm)</td>
</tr>
<tr>
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<td>High Shielding</td>
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<td>333</td>
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<tr>
<td></td>
<td>Far-Field Exposure</td>
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<td>107</td>
</tr>
<tr>
<td></td>
<td>Urban</td>
<td></td>
<td>593</td>
</tr>
</tbody>
</table>

The comparison made for building #9 consists of the aerodynamic data and the peak base moment about the x-axis; for which eight sets of data were selected from three reference buildings to calculate the peak base moments. The maximum value (359 MNm) agrees well with the peak base moment obtained from the wind tunnel test (333 MNm). The fact that the peak base moments obtained from the other seven sets of data are all less than the wind tunnel peak base moment can be attributed to the overgeneralization of the near-field exposure. Although building #9 was classified as the high shielding at an angle of 30°, there are narrow passages between surrounding buildings with less shielding resulting in large wind loads (see Fig. 4.10). Figure 4.11 shows that the level of
shielding of the reference buildings (i.e. buildings within the ovals) is much greater than that of the target building, with the exception of building #6, which also has a narrow passage between surrounding buildings resulting in a similar peak base moment as the target building.

The reference building selection process carried out for building #21 resulted in twelve sets of inputs obtained from four reference buildings. Building #21 presents an example of an extreme case where all of the estimated peak base moments about the y-axis were greater than the wind tunnel peak base moment. At a wind angle of 280°, building #21 is completely covered by surrounding buildings as can be seen from Fig. 4.12, causing low wind loads. Although all of the reference buildings are categorized as high shielding at this wind angle, the shielding effects associated with the reference buildings are not as severe as those associated with building #21 (see Fig. 4.13). As a result, all of the peak base moments calculated using the twelve sets of input parameters from the four reference buildings are larger than the peak base moment (107 MNm) obtained from the wind tunnel test. Furthermore, because the maximum value of the peak base moments obtained from all sets of data from the reference buildings at a given wind angle is selected as the WAD estimate, the WAD output of the peak base bending moment about the y-axis for building #21 is an overly conservative estimate (by a factor of about 3) of the corresponding wind tunnel result. In addition, it is also important to note that the wind tunnel load at 280° is low compared with those corresponding to the other angles. The over-estimation of the final output for building #21 is significantly reduced from a factor of about 3 to a factor of 1.37 when considering the maximum peak base moment over all wind angles. When one of building #21’s reference buildings is used as a target building a similar grouping of reference buildings will be selected and the aerodynamic loads for building #21 at 280° will always be lower than the other reference buildings at the same angle. Therefore, the aerodynamic loads for building #21 at a wind angle of 280° are an anomaly and will rarely, if ever, be chosen as the final output.

Because the near-field exposure classified for each building is inferred based on the photographs of the test model in its natural surroundings, the method can lead to an overgeneralization of the configuration of the surrounding buildings and result in scatter
between the input values. This suggests that better categorization of the near-field exposures for the buildings in WAD are beneficial for improving the performance of the reference building selection process. On the other hand, there is comfort in that the current selection procedure tends to result in conservative results. Ideally, to determine a building’s near-field category accurately, information on the exact location and size of surrounding buildings would be needed; however, this approach is impractical because this information is not readily available. The categorization of a building’s near-field exposure can be improved by using three-dimensional satellite imaging software such as Google Earth®. This would provide a better mapping of the size and location of surrounding buildings rather than inferring it based on photographs.

The peak base moments calculated for building #25 were calculated using four sets of input data obtained from two reference buildings. The maximum peak base moment of 683 MNm is 15% higher than the wind tunnel peak base moment (593 MNm), but is considered reasonably accurate. Similar to example 1, the maximum peak base moment is the only value that is larger than the wind tunnel peak base moment. The two peak base moments (491 MNm and 566 MNm) obtained from the input data associated with building #27 are lower than the wind tunnel peak base moment by 17% and 4%, respectively, and are therefore considered fairly accurate estimates. The peak base moment (202 MNm) obtained from the input data associated with building #10 is 66% lower than the wind tunnel peak base moment. This can be attributed to the very high level of shielding for building #10 at the wind angle of 230° as shown in Fig. 4.14, which is similar to the shielding condition for the target building (i.e. building #21) in example 2. It can be seen from the three examples that using the maximum peak base moment is in general reasonably accurate and in the worst case may lead to an overly conservative estimation when compared to the wind tunnel peak base moments.
Figure 4.7: Peak Base Moment and Normalized Aerodynamic Loads for Building 9 at Wind Angle 30° - Base Moment (x-axis)
Figure 4.8: Peak Base Moment and Normalized Aerodynamic Loads for Building 21 at Wind Angle 280° - Base Moment (y-axis)
Figure 4.9: Peak Base Moment and Normalized Aerodynamic Loads for Building 25 at Wind Angle 230° - Base Moment (x-axis)
Figure 4.10: Example 1 Target Building

Figure 4.11: Example 1 Reference Buildings
Figure 4.12: Example 2 Target Building

Figure 4.13: Example 2 Reference Buildings

Figure 4.14: Example 3 Building # 10
The analysis results are depicted and discussed in terms of the ratios between the wind responses evaluated for a given target building from a given approach (i.e. WAD, design codes, and NALD) and the corresponding responses obtained from the wind tunnel test. The ratio for the peak base bending moments and torque, $\gamma_j^*$, is defined as follows:

$$\gamma_j^* = \frac{M_j^*}{M_{jWT}^*}$$  \hspace{1cm} (4.4)

where $M_j^*$ is the peak base moment about the $j$-axis ($j = x, y, T$) associated with a given method; $\cdot$ is a generic symbol to denote the method used to evaluate the wind loads, i.e. WAD, ASCE 7-10, AS/NZS 1170.2:2011, NBCC 2010, NALD, and $M_{jWT}^*$ is the peak base moment about the $j$-axis obtained from the wind tunnel test. The ratio for the acceleration, $\omega^*$, is defined as

$$\omega^* = \frac{\hat{a}^*}{\hat{a}_{WT}^*}$$  \hspace{1cm} (4.5)

where $\hat{a}^*$ is the peak resultant/max acceleration along the $j$-axis ($j = x, y, T$) associated with a given method (i.e. WAD, ASCE 7-10, AS/NZS 1170.2:2011, NBCC 2010 and NALD), and $\hat{a}_{WT}^*$ is the peak resultant acceleration along the $j$-axis obtained from the wind tunnel test.

Table 4.2 is a summary of the basic statistics of $\gamma_j^*$, whereas Fig. 4.16 is a box plot showing the overall spread of $\gamma_j^*$. In this figure, the left end of the “whisker” to the left of the box is the minimum value, the left side of the box is the 1st quartile (i.e. 25-percentile), the middle line is the median, the right side of the box is the 3rd quartile (i.e. 75-percentile), and the right end of the right “whisker” is the maximum value. The black dot is the mean value of the data set. Detailed tables and plots of $\gamma_j^*$ for each of the 36 buildings considered are given in Appendix D.
The results in Table 4.2 suggest that the WAD-based procedure provides reasonably accurate estimates of the peak base moments for the buildings considered. The mean values of $\gamma_x^{WAD}$, $\gamma_y^{WAD}$, and $\gamma_T^{WAD}$ are 1.01, 0.94, and 1.03, respectively, and the corresponding standard deviations are 0.24, 0.28, and 0.48 respectively. Figure 4.16 indicates that the peak base moments predicted based on WAD are within ±20% of the corresponding wind tunnel results for approximately 50% of the buildings considered. WAD markedly under-predicts the base torque for one building with the corresponding $\gamma_T^{WAD}$ equal to 0.39; however, it should be noted that the magnitude of the peak base torque for this building is negligibly small (only 84 MNm) according to the wind tunnel results, which may explain the low value of $\gamma_T^{WAD}$.

The peak base moments estimated based on WAD are on average slightly more conservative than those estimated from the three design codes considered. The design codes, on average, result in reasonably accurate estimates of the peak base moments; for example, the mean values of $\gamma_x^{ASCE}$, $\gamma_y^{ASCE}$, $\gamma_T^{ASCE}$, $\gamma_x^{AS/NZS}$, $\gamma_y^{AS/NZS}$, $\gamma_x^{NBCC}$, $\gamma_y^{NBCC}$ and $\gamma_T^{NBCC}$ are equal to 0.97, 0.91, 0.95, 1.00, 0.95, 0.92, 0.92 and 0.97 respectively. However, the variability of the predictions corresponding to the design codes is markedly higher than that of the predictions corresponding to WAD. This is reflected in the larger standard deviations of $\gamma_j^*$ associated with the design codes: the standard deviations of $\gamma_x^{ASCE}$, $\gamma_y^{ASCE}$, $\gamma_T^{ASCE}$, $\gamma_x^{AS/NZS}$, $\gamma_y^{AS/NZS}$, $\gamma_x^{NBCC}$, $\gamma_y^{NBCC}$ and $\gamma_T^{NBCC}$ equal 0.60, 0.45, 0.60, 0.49, 0.39, 0.49, 0.40 and 0.49, respectively.

The mean values of $\gamma_j^{WAD}$ are also higher than those of $\gamma_j^{NALD}$ except for the mean value of $\gamma_x^{NALD}$. The mean values of $\gamma_x^{NALD}$, $\gamma_y^{NALD}$, and $\gamma_T^{NALD}$ equal 1.23, 1.03, and 0.67 respectively. It is noted that the relatively large mean value of $\gamma_x^{NALD}$ is mainly attributed to a single data point with a very high value of $\gamma_x^{NALD}$ (6.83). For this building, the reduced frequency corresponding to the design wind speed considered is close to the Strouhal number of the reference building (i.e. the building selected from NALD), at which the spectrum of the across-wind base bending moment (i.e. the base bending moment about the x-axis in this case) has a sharp peak. The reduced frequency of the fundamental mode in the x-axis for the target building is equal to 0.08 and the peak of the
across-wind spectrum of the reference building is located at a reduced frequency of 0.89. This leads to significant errors in the results, because the Strouhal number of the target building is likely to be either different from that of the reference building or the magnitude of the peak of the across-wind spectrum is lower due to the influence of the surrounding buildings that is not taken into account in NALD. Zhou et al. (2003) also state that the estimation of the across-wind response based on the NALD approach is not advocated for buildings with a reduced frequency that is close to the Strouhal number of the reference building. The variability of the NALD-based predictions of the peak base bending moments is higher than the WAD-based predictions: the standard deviations of $\gamma^\text{NALD}_X$ and $\gamma^\text{NALD}_Y$ equal 1.14 and 0.54 respectively. The standard deviation of $\gamma^\text{NALD}_T$ is equal to 0.44.

For both NALD and WAD the last spectral value is used when the reduced frequency of the target building is outside the range of the normalized power spectrum of the base moment of the reference building. This does not occur in the WAD-based predictions; however, this occurs in about 40 cases (each case corresponding to a given wind attack angle) in NALD and in the majority of these cases $\gamma^\text{NALD}_T \geq 1.2$ or $\leq 0.8$.

In general, the WAD-based predictions of the peak base moments are more accurate than the predictions of the peak base moments made by the design codes and NALD. However, it should be noted that for five of the 36 buildings considered, namely buildings #5, #7, #8, #20 and #30, the corresponding values of $\gamma^\text{WAD}_j$ are all less than 0.8 except $\gamma^\text{WAD}_X$ for building #20 and $\gamma^\text{WAD}_T$ for building #8 and #30. For these five buildings, reference buildings were identified for wind angles at which the near-field exposure of the target building is moderate or high shielding, but no reference buildings were identified for wind angles at which the near-field exposure of the target building is no shielding or low shielding. For example, Figure 4.15 gives a scenario in which reference buildings were not found for angles with no shielding or low shielding but were found for angles that were moderate or high shielding. Given that an increase in the shielding effect tends to reduce the overall wind loads on the target building, as discussed in Section 2.3.4, the low values of $\gamma^\text{WAD}_j$ for these five buildings can therefore be
explained from the fact that the wind loads corresponding to wind angles associated with the low shielding condition were not calculated due to a lack of the number of reference buildings. Therefore, in situations such as these the user will be prompted with a warning that the peak base moment output from WAD has not considered angles with less shielding and will likely be non-conservative.

Figure 4.15: Scenario of Reference Building Selection Coverage (Moderate or high shielding = coverage, No shielding or low shielding = no coverage)

Values of $\gamma^WAD_f$ greater than 1.0 can occur when the reduced frequency of the target building is close to the reduced frequency that corresponds to the peak in the across-wind base bending moment of the reference building, e.g. $\gamma^WAD_f = 1.07$ for building #9. This value is not nearly as high as the value predicted by NALD (6.83) because of the impact of turbulence generated by the wake of the surrounding buildings. In the case when the reduced frequency of the target building, exposed to high or immersed conditions, is close to the peak of the across-wind spectrum of the reference building, the inclusion of the near-field exposure is expected to result in a better estimate of the across-wind response than NALD. In addition, values of $\gamma^WAD_f$ greater than 1.0 can also result from the overgeneralization of the near-field exposure categorization where local effects and interference can lead to large wind loads, as discussed in Section 4.3.2 for building #21: the value of $\gamma^WAD_y$ for building #21 is 1.37.

The variation in the predictions made by WAD raises the question of what the user should use as the final value for preliminary design. Although on average the WAD-
based predictions are expected to be close to 1.0, individual values of $\gamma_x^{WAD}$, $\gamma_y^{WAD}$, and $\gamma^T_{WAD}$ may differ from unity markedly. In addition, providing the user with underestimated peak base moments would be undesirable; however, as discussed previously, the primary source of under-predictions result when buildings have reference buildings at angles categorized as moderate or high shielding but not at angles categorized as no shielding and low shielding. Therefore, WAD can be updated so that conservative estimations are provided; possibly by using the next closest match candidate building that is classified as no shielding or low shielding near-field categories. It can also be suggested that the user use the output $\tilde{M}_x^{WAD}$, $\tilde{M}_y^{WAD}$, and $\tilde{M}_T^{WAD}$ and increase it by a factor of approximately 1.20, 1.30, and 1.20 so that the value reaches past the 75th percentile of the WAD-based predictions; however, this is subject to further discussion in the future.
Table 4.2: Basic Statistics of Peak Base Moment Ratios ($\gamma_j^*$)

<table>
<thead>
<tr>
<th></th>
<th>ASCE 7-10</th>
<th>AS/NZS 1170.2:2011</th>
<th>NBCC 2010</th>
<th>NALD</th>
<th>WAD</th>
</tr>
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<tr>
<td></td>
<td>x</td>
<td>y</td>
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<td>Maximum</td>
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<td>0.45</td>
<td>0.60</td>
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Table 4.3 is a summary of the basic statistics of $\omega^*$, whereas Fig. 4.17 is a box plot, similar to Fig. 4.16, showing the overall spread of $\omega^*$. Tables and plots of the peak accelerations determined from all the methods can be found in Appendix E. The mean value and standard deviation of $\omega^{WAD}$ equal 1.01 and 0.44, respectively, which suggest that the WAD-based procedure is reasonably accurate in predicting the acceleration compared with the wind tunnel test. Figure 4.17 further indicates that the accelerations
predicted based on WAD are within ±20% of the corresponding wind tunnel results for close to 50% of the buildings considered.

In comparison, the mean values of $\omega^{\text{ASCE}}$, $\omega^{\text{AS/NZS}}$, and $\omega^{\text{NBCC}}$ equal 0.34, 0.27, and 0.74, respectively. It should be noted that the marked under-predictions corresponding to ASCE 7-10 and AS/NZS 1170.2:2011 are partly due to the fact that the corresponding predictions only include the along-wind peak accelerations. The under-predictions corresponding to the three design codes can be also attributed to the fact that the codes do not take into account the impact of the surrounding buildings in evaluating the wind responses, which generally leads to over-prediction of the mean components of the responses as a result of ignoring the shielding effect, but under-prediction of the dynamic components of the responses, which directly impacts the acceleration, as a result of ignoring the large turbulence generated by the surrounding buildings. The variability of $\omega^{\text{ASCE}}$, $\omega^{\text{AS/NZS}}$, and $\omega^{\text{NBCC}}$ is higher than that of $\omega^{\text{WAD}}$: the standard deviations of $\omega^{\text{ASCE}}$, $\omega^{\text{AS/NZS}}$, and $\omega^{\text{NBCC}}$ equal 0.20, 0.14, and 0.73, respectively, which result in the coefficients of variation (COV, defined as the standard deviation divided by the mean) of 59, 52 and 99%, respectively. In contrast, the COV of $\omega^{\text{WAD}}$ equals 44%.

The mean value of $\omega^{\text{NALD}}$ is equal to 1.02; however, this value is skewed by a single data point with a high value of $\omega^{\text{NALD}}$ (7.29), because the median value of $\omega^{\text{NALD}}$ is only 0.67 as shown in Table 4.3. This data point is associated with the same building for which a high value of $\gamma^{\text{NALD}}$ (6.83) was obtained as described in Section 4.4.2. The high value of $\omega^{\text{NALD}}$ for this building can be attributed to the same reason that causes the high value of $\gamma^{\text{NALD}}$, as described in Section 4.4.2. Finally, the standard deviation of $\omega^{\text{NALD}}$, which equals 1.19, is almost three times that of $\omega^{\text{WAD}}$.

The results shown in Tables 4.2 and 4.3, and Figs. 4.16 and 4.17 suggest that the-WAD based procedure is an improvement on the existing methods and is a viable database-assisted approach to evaluate the wind responses for the preliminary design of tall buildings. The peak base moments and acceleration evaluated from WAD agree reasonably well with the corresponding wind tunnel test results, and are more accurate
than those corresponding to ASCE 7-10, AS/NZS 1170.2:2011, NBCC 2010 and NALD for the 36 buildings considered in this study.

Finally, it should be noted that the results summarized in Tables 4.2 and 4.3 are based on the fact that the structural dynamic properties of the considered buildings (i.e. buildings in WAD) have been evaluated from detailed structural analyses; this will likely not be the case during the preliminary design of a given target building. However, the relative accuracies of wind loads and responses predicted from the design codes, NALD and WAD as observed from Tables 4.2 and 4.3 are still considered valid.

Table 4.3: Basic Statistics of Peak Acceleration Ratios ($\omega^*$)

<table>
<thead>
<tr>
<th></th>
<th>ASCE 7-10</th>
<th>AS/NZS 1170.2:2011</th>
<th>NBCC 2010</th>
<th>NALD</th>
<th>WAD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>0.92</td>
<td>0.59</td>
<td>3.94</td>
<td>7.29</td>
<td>2.32</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.08</td>
<td>0.04</td>
<td>0.16</td>
<td>0.18</td>
<td>0.26</td>
</tr>
<tr>
<td>Mean</td>
<td>0.34</td>
<td>0.27</td>
<td>0.74</td>
<td>1.02</td>
<td>1.01</td>
</tr>
<tr>
<td>Median</td>
<td>0.27</td>
<td>0.22</td>
<td>0.52</td>
<td>0.67</td>
<td>0.92</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.20</td>
<td>0.14</td>
<td>0.73</td>
<td>1.19</td>
<td>0.44</td>
</tr>
</tbody>
</table>

Figure 4.17: Box Plot for the Peak Acceleration Ratios
This chapter describes the validation of the WAD-based procedure for evaluating the wind-induced responses of tall buildings, namely the peak base moments and peak accelerations. The validation was carried out by comparing the peak base moments and peak acceleration at the top of the building evaluated using the WAD-based procedure with those obtained from the wind tunnel test for 36 buildings selected from WAD. Furthermore, the peak base moments and peak accelerations evaluated from the WAD-based procedure were also compared with the corresponding values obtained from other approaches for evaluating the wind responses that include three well recognized design codes, ASCE7-10, NBCC 2010 and AS/NZS 1170.2:2011, as well as NALD.

Examples illustrating the reference building selection process indicated that improvements should be made on the categorization of the near-field exposure for each building to more accurately map the configuration of the surrounding buildings. It is also important that new buildings are continually added to the database to encompass a larger range of combinations of geometric properties and exposure conditions, and so that more reference buildings can be matched with a target building. The selection of the maximum peak base moment over all wind angles and between all reference buildings is found to be sufficient for the current application of WAD.

The wind load analysis results suggest that the WAD-based procedure can predict the peak base moments and accelerations with reasonable accuracy. The mean values of the ratios between the predicted wind responses and the corresponding responses evaluated from the wind tunnel test, denoted by $\gamma_x^{WAD}$, $\gamma_y^{WAD}$, $\gamma_T^{WAD}$, and $\omega^{WAD}$, equal 1.01, 0.94, 1.03, and 1.01, respectively, and the standard deviations of $\gamma_x^{WAD}$, $\gamma_y^{WAD}$, $\gamma_T^{WAD}$, and $\omega^{WAD}$ equal 0.24, 0.28, 0.48, and 0.44, respectively. The peak base moments and accelerations predicted based on WAD are within ±20% of the corresponding wind tunnel results for close to 50% of the buildings considered. The majority of the under-predictions made by WAD are caused by the insufficient coverage of the angles around the target building with lower shielding conditions while the other wind angles are categorized as moderate or high shielding. In these situations, the user will be prompted
The analysis results further indicate that although the peak base moments evaluated from ASCE 7-10, AS/NZS 1170.2:2011, NBCC2010 and NALD are on average close to the wind tunnel test results, the variability of the predictions corresponding to these methods is markedly higher than that of the WAD-based predictions. The three design codes also tend to result in under-predictions of the peak acceleration at the top of the building, and the variability of the accelerations predicted from the three design codes and NALD is markedly higher than that of the WAD-based predictions. The WAD-based predictions are in general an improvement on the existing methods of evaluating the wind loads. The results of both the peak base moments and acceleration predictions made by WAD indicate that the WAD-based procedure is a reasonable approach to evaluating the preliminary design wind loads for tall buildings.
Tall buildings are often susceptible to the vibration induced by the natural wind especially if the buildings are lightweight and slender. Generally treated as a bluff body, a tall building oscillates in the along-wind, across-wind, and torsional directions under the action of wind. Aerodynamic factors such as the approaching boundary layer flow, building geometry, and interference and shielding effects are important in considering the aerodynamic loads. In addition, the vibration response of a tall building is an important consideration for highly dynamic structures.

The existing methods of determining the design wind loads on tall buildings consist of the design codes, the wind tunnel test and the database-assisted approach represented by the NatHaz Aerodynamic Load Database (NALD) developed at the University of Notre Dame. The design codes considered in this thesis, i.e. ASCE 7-10, AS/NZS 1170.2:2011, and NBCC 2010, follow the gust loading factor approach developed by Davenport (1967) with some variation in the details of each code. The along-wind peak wind pressure is evaluated by multiplying the design wind pressure by the gust loading factor, GLF (Davenport 1967). In NBCC 2010, the effect of gustiness in the wind is included by factoring up the mean wind speed; however, design codes that use 3-second gust wind speeds inherently include the effects of gustiness. Therefore, ASCE 7-10 and AS/NZS 1170.2:2011 use a so-called gust effect factor (GEF) or dynamic response factor (DRF); which is equal to the GEF divided by the gust factor for the wind velocity pressure (Solari and Kareem 1998; Holmes et al. 2009; Kwon and Kareem 2013). The design codes lack in provisions for the across-wind responses, effects caused by presence of surrounding buildings, and consideration of wind acting at angles other than the angles perpendicular to a wall; all of which can potentially lead to an over- or under-estimation of the wind loads. The design codes also have limited applicability in terms of the building height, exposures, and the dynamic properties of the building.
Buildings that are out of the scope of the design codes are recommended for the wind tunnel test. The wind tunnel testing involves the physical measurement of the wind loads using scaled models. Two commonly used wind tunnel testing methods are the force balance and pressure model tests; in which the aerodynamic loads are directly measured (Tschanz 1982; Ho et al. 1999). In the force balance model test carried out by the Boundary Layer Wind Tunnel Laboratory of Western University, the aerodynamic loads are measured at the base of a scaled rigid model by a high frequency ultra-sensitive force balance. The aerodynamic loads in the pressure model test are measured by pressure taps located on the surface of the geometrically scaled rigid model. In both tests the resonant component of the response is evaluated using the random vibration analysis, assuming that aerodynamic damping is negligible. Although the wind tunnel testing is the most accurate method of evaluating the design wind loads for a tall building, it may not be practical during the preliminary design phase when the design of the building is not finalized.

Alternatively, the database-assisted approach such as NALD can be used in the preliminary design phase (Kwon et al. 2008). The basic premise of the database-assisted approach is that the wind responses of a tall building can be evaluated from the aerodynamic data of a reference building with similar aerodynamic characteristics and exposure conditions. NALD employs the three-dimensional moment gust loading factor (3D MGLF) approach (Kareem and Zhou 2003) to evaluate the peak base bending moments and accelerations. NALD is an improvement on the design codes in that the along-wind, across-wind, and torsional responses can be determined. However, the aerodynamic data stored in NALD are corresponding to a limited number of building geometries, and wind loads acting along the primary axes of buildings under the isolated condition only. Additionally guidance is selecting a building in NALD to extract the normalized aerodynamic loads from the database is not provided, making it difficult when the building in consideration does not exactly match one of the buildings in NALD.

The limitations of the design codes and NALD motivated the development of the Western University Aerodynamic Database (WAD) to supplement existing methods for determining preliminary design wind loads. The database currently contains
aerodynamic data of 56 different tests carried out in the Boundary Layer Wind Tunnel Laboratory at Western University. The aspect ratio, slenderness ratio and heights of the buildings in WAD range from 1.0 to 3.4, 1.2 to 10.4, and 60 to 350 m respectively. The collected aerodynamic data consist of the mean and RMS fluctuating base moment coefficients and the normalized power spectrums of the base moments, acting about three primary axes (i.e. x-, y- and torsional-axes) of the building, corresponding to wind acting from 36 different angles around the building. Each building’s upstream exposure is characterized by the far-field and near-field exposures around the building. The far-field exposure characterizes the conditions of the general terrain and the near-field exposure characterizes the shielding conditions caused by the surrounding buildings.

The WAD-based wind response evaluation procedure consists of the reference building selection process and the database-assisted wind response analysis. The reference building is defined as the building whose aerodynamics and exposure conditions are similar to those of the building (i.e. the target building) for which the wind responses are to be evaluated. The reference building selection process is carried out based on a fuzzy rule-based inference system and compares the aspect ratio, slenderness ratio, and far-field and near-field exposure of a candidate building with the target building. The fuzzy rule-based inference system consists of the following four steps: (1) fuzzy matching, (2) inference, (3) combination, and (4) defuzzification (Sivanadam et al. 2007). In the fuzzy matching step, the input data are converted into fuzzy values, i.e. values that range from 0 to 1.0, using membership functions. The input data include the absolute difference between the numerical values of the near-field categories corresponding to the target and candidate building, denoted as \( n \), and the absolute difference between the aspect/slenderness ratios of the target and candidate buildings divided by the aspect/slenderness ratio of the target building, denoted as \( r_A \) and \( r_S \) respectively. The far-field categories corresponding to the target and candidate buildings must be the same for the candidate building to be considered in the selection process. The inference step includes the evaluation of the following rule: “If \( n < 2 \) and \( r_A < 0.2 \) and \( r_S < 0.2 \), then a candidate building is included in the combination step.” For this fuzzy inference system the combination step is not required because only one fuzzy rule is considered. The defuzzification step involves evaluating the final matching index that characterizes the
degree of overall matching between the target building and a candidate building. The standard MatLAB® defuzzification method, smallest of maximum, is employed to determine the final matching index. A candidate building is selected as a reference building if the final matching index is greater than or equal to 0.5. Once the reference buildings have been selected, the RMS base moment coefficients and the normalized power spectrums of the base moments are collected for the database-assisted wind load analysis.

By assuming that the fundamental sway and torsional modes of a tall building are uncoupled and linear along the height of the building, a modified 3D MGLF approach was derived based on the modal analysis and random vibration theory, and implemented in WAD to calculate the peak base bending moments about the $x$- and $y$-axes and peak base torque of the building corresponding to arbitrary wind attack angles. The peak moment about a given axis consists of the mean component as well as the background and resonant dynamic components. Given the base bending moments, equations for calculating the peak accelerations at the center of mass of each floor were also derived. Finally, equations were also derived to distribute the peak base moments along the height of the building to evaluate the equivalent static wind loads (ESWL) that can be used in the preliminary design. The WAD-based wind load evaluation inherently considers the building’s along-wind, across-wind, and torsional responses acting at arbitrary wind attack angles with the consideration of interfering/shielding effects caused by surrounding buildings. In addition, the WAD program carries out the reference building selection process without having the user make judgments on selecting the appropriate building in WAD.

A MatLab®-based graphic user interface has been developed to facilitate the interaction between the user and WAD. The user is required to input parameters such as the geometric and dynamic properties, the upstream exposure conditions, and the design wind speed at the roof height of the target building corresponding to the exposure category. The upstream exposure conditions and the design wind speeds need to be defined at eight 45-degree quadrants around the building. The MatLab® script program included in the graphic user interface then carries out the reference buildings selection
process and the database-assisted wind response analysis, and outputs results. The program output consists of the results of the reference building selection process, the peak base moments, and peak accelerations at the top of the building and center of mass evaluated using the modified 3D moment gust loading factor approach. The peak base moments calculated according to NBCC 2010 are also provided for comparison.

Analyses were carried out to validate the WAD-based procedure for evaluating the wind responses of tall buildings. To this end, each of the 56 buildings in WAD was subjected to the reference building selection process to select the corresponding reference buildings from the remaining 55 buildings. This resulted in 36 buildings, each of which is associated with at least one reference building. The peak base moments about the $x$-, $y$- and torsional axes and peak accelerations at roof height evaluated based on WAD were compared with the corresponding results obtained from the wind tunnel tests for these 36 buildings. Furthermore, peak base moments and accelerations at the roof height were also evaluated using ASCE 7-10, AS/NZS 1170.2:2011, NBCC 2010 and NALD and compared with the corresponding results evaluated based on WAD.

The reference building selection process was shown to illustrate the appropriateness of the procedures and determine areas where improvements should be made. It indicated that improvements should be made on the categorization of the near-field exposure for each building to more accurately map the configuration of the surrounding buildings. It is also important that new buildings are continually added to the database to encompass a larger range of combinations of geometric properties and exposure conditions, and so that more reference buildings can be matched with a target building. The selection of the maximum peak base moment over all wind angles and among all reference buildings is found to be sufficient for the current application of WAD.

The analysis results indicate that the predictions corresponding to WAD agree well with the corresponding wind tunnel test results: the mean values of $\gamma_{x}^{WAD}$, $\gamma_{y}^{WAD}$, $\gamma_{t}^{WAD}$ and $\omega^{WAD}$ for the 36 buildings equal 1.01, 0.94, 1.03 and 1.01, respectively; the corresponding standard deviations equal 0.24, 0.28, 0.48 and 0.44, respectively. The peak base moments and peak accelerations evaluated based on WAD are within 20% of
the corresponding wind tunnel test results for approximately 50% of the 36 buildings considered. The peak base moment ratios below 0.8 mostly results from the absence of reference buildings at angles with no shielding or low shielding conditions for a target building, while the remaining angles are categorized as moderate or high shielding. For these cases, warnings will be provided to the user. The predictions corresponding to WAD are also more accurate than those corresponding to ASCE 7-10, AS/NZ 1170.2:2011, NBCC 2010 and NALD in that the mean values of the WAD-based predictions are closer to unity, and the variability of the WAD-based predictions is in general much lower than that of the predictions corresponding to the other approaches. These results suggest that WAD is a viable alternative and is recommended to supplement existing methods for evaluating the wind responses for the preliminary design of tall buildings.

It should be noted that the application of WAD is suited for rectangular slender tall buildings with relatively low natural frequencies, but is not suited for circular buildings or buildings of unusual shapes that cannot be adequately simplified as rectangular cylinders. Caution should be exercised for situations where the target buildings are interfered by isolated individual buildings; which can potentially amplify the wind responses. Caution should also be used for buildings subjected to wind speeds that are near the Strouhal number. Due to the linear and uncoupled fundamental modes assumed in the modified 3D MGLF approach employed in WAD, the application of WAD is not recommended for buildings that have highly non-linear and/or coupled fundamental modes, or highly flexible structures that are prone to aerodynamic damping.

The recommended future work is as follows:

1. More data should be added to WAD to increase the number of candidate buildings for a wide range of combinations of the near-field and far-field exposure conditions, and the geometric properties. This will decrease the likelihood of not being able to identifying reference buildings for a given target building.
2. The fuzzy logic inference system can be improved to increase the degree of matching between the target and reference buildings.

3. The current categorization of the near-field exposure conditions is very general and can be improved to incorporate the information about the number, size, and location of the interfering/shielding buildings.

4. The terrain conditions surrounding a target building often vary with the distance away from the building. Considerations for these terrain changes should be included in the future.

5. The modified 3D moment gust loading factor approach implemented in WAD assumes uncoupled modes. The accuracy of the wind responses evaluated based on WAD can be improved by taking into account coupled modes.


Engineering Sciences Data Unit International (ESDU) (2005), *Computer program for wind speeds and turbulence properties: flat or hilly sites in terrain with roughness changes*, Item 01008, Issued December 2001 with Amendments A April 2002, ESDU International.


Soliman, B.F. (1976), “Effect of building group geometry on wind pressure and properties of flow”, Report No. BS 29, Department of Building Science, University of Sheffield, Sheffield, U.K.


Appendix A: Derivation of the Modified 3D Moment Gust Loading Factor Approach

For a tall building under the action of wind at an arbitrary attack angle, the sign conventions for the bending moments and torque at the base of the building are shown in Fig. 3.5. Following the 3D moment gust loading factor approach proposed by Kareem and Zhou (2003), we express the peak base moments about the $j$-axis ($j = x, y$ or $T$), $\bar{M}_j$, as follows:

$$\bar{M}_j = \bar{M}_j + \sqrt{g_B^2 \sigma_{MB_j}^2 + g_R^2 \sigma_{MR_j}^2}$$  \hspace{1cm} (A.1)

where $\bar{M}_j$ is the mean base moment about the $j$-axis; $\sigma_{MB_j}$ and $\sigma_{MR_j}$ are the background and resonant components of the RMS dynamic base moment about the $j$-axis respectively, and $g_B$ and $g_R$ are the background and resonant peak factors respectively. Note that $\sigma_{MB_j}$ can be set to equal $\sigma_{M_j}$, which is the RMS value of the fluctuating aerodynamic base moment about the $j$-axis. Note that the dependence of $\bar{M}_j$ on the wind angle is implicitly considered to reduce clutter in the formulation. Given the RMS values and spectrums of the aerodynamic base moments, $\sigma_{MR_j}$ can be evaluated based on the random vibration analysis described in the following.

Assume that the mode shapes of the three fundamental vibration modes (i.e. two sway modes and one torsional mode) of the building are uncoupled and linear over the height of the building. Let $\phi_x$ and $\phi_y$ denote the fundamental sway modes along the $x$- and $y$-axes respectively, and $\phi_T$ denote the fundamental torsional mode. It follows from the above assumptions that

$$\phi_x(z) = \phi_y(z) = \phi_T(z) = z/H$$  \hspace{1cm} (A.2)
where \( z \) denotes a given elevation along the height of the building, and \( H \) denotes the overall height of the building. Let \( p_j(z, t) \) denote the fluctuating (i.e. time-varying) aerodynamic load per unit height along the \( j \)-axis on the building, where \( t \) denotes time. Further assume \( p_j(z, t) \) to be a stationary Gaussian process.

Apply the modal analysis to evaluate the dynamic response of the building under the simultaneous action of \( p_j(z, t) \) and consider the three fundamental modes only. The generalized forces associated with the three modes, namely \( Q_x^*(t) \), \( Q_y^*(t) \) and \( Q_T^*(t) \) respectively, are given by

\[
Q_x^*(t) = \int_0^H p_x(z, t) \phi_x(z) \, dz = \frac{M_x(t)}{H} \quad (A.3)
\]

\[
Q_y^*(t) = \int_0^H p_y(z, t) \phi_y(z) \, dz = \frac{M_y(t)}{H} \quad (A.4)
\]

\[
Q_T^*(t) = \int_0^H p_T(z) \phi_T(z) \, dz \approx 0.7 M_T(t) \quad (A.5)
\]

where \( M_x(t) \) and \( M_y(t) \) are the time-varying aerodynamic base bending moments about \( x \)- and \( y \)-axes respectively; \( M_T(t) \) is the aerodynamic base torque, and the coefficient 0.7 in Eq. (A.5) is an empirical correction factor used to account for the fact that the influence function for the base torque is unity along the height of the building (Ho and Jeong 2008).

Let \( \eta_x(t) \), \( \eta_y(t) \) and \( \eta_T(t) \) denote the generalized coordinates associated with the three fundamental modes respectively. If follows from the random vibration theory (Clough and Penzien 1975) and Eqs. (A.3) through (A.5) that the power spectrum density functions (or spectrums for simplicity) of the generalized coordinates, \( S_{\eta_x}(f) \), \( S_{\eta_y}(f) \) and \( S_{\eta_T}(f) \), are given as follows:

\[
S_{\eta_x}(f) = \frac{1}{K_x^2} S_{Q_x^*}(f) \left| H_x(f) \right|^2 = \frac{1}{H^2 K_x^2} S_{M_x}(f) \left| H_x(f) \right|^2 \quad (A.6)
\]

\[
S_{\eta_y}(f) = \frac{1}{K_y^2} S_{Q_y^*}(f) \left| H_y(f) \right|^2 = \frac{1}{H^2 K_y^2} S_{M_y}(f) \left| H_y(f) \right|^2 \quad (A.7)
\]
\[ S_{\eta_T}(f) = \frac{1}{K_T^2} S_{Q_T}(f) |H_T(f)|^2 = \frac{0.7^2}{K_T^2} S_{M_T}(f) |H_T(f)|^2 \]  
(A.8)

\[ |H_x(f)|^2 = \frac{1}{\left(1 - \left(\frac{f}{f_1}\right)^2\right)^2 + 4\xi_x^2 \left(\frac{f}{f_1}\right)^2} \]  
(A.9)

\[ |H_y(f)|^2 = \frac{1}{\left(1 - \left(\frac{f}{f_1}\right)^2\right)^2 + 4\xi_y^2 \left(\frac{f}{f_1}\right)^2} \]  
(A.10)

\[ |H_T(f)|^2 = \frac{1}{\left(1 - \left(\frac{f}{f_T}\right)^2\right)^2 + 4\xi_T^2 \left(\frac{f}{f_T}\right)^2} \]  
(A.11)

where \( K_j^*, f_j \) and \( \xi_j \) (\( j = x, y \) or \( T \)) are the generalized stiffness, natural frequency and modal damping ratio associated with the fundamental mode along the \( j \)-axis, respectively; \( S_{\bullet}(f) \) denotes the spectrum for a given quantity \( \bullet \); \(|H_x(f)|, |H_y(f)| \) and \(|H_T(f)|\) are the modules of the mechanical admittance functions corresponding to the three modes respectively, and \( f \) is frequency.

The equivalent generalized forces, \( \tilde{Q}_x^*(t), \tilde{Q}_y^*(t) \) and \( \tilde{Q}_T^*(t) \) (i.e. the generalized forces that incorporate the dynamic responses of the building), are given by (Kareem and Zhou 2003)

\[ \tilde{Q}_x^*(t) = K_{x^*}\eta_x(t) = \int_0^H \tilde{p}_x(z,t)\phi_x(z)dz = \frac{\tilde{M}_x(t)}{H} \]  
(A.12)

\[ \tilde{Q}_y^*(t) = K_{y^*}\eta_y(t) = \int_0^H \tilde{p}_y(z,t)\phi_y(z)dz = \frac{\tilde{M}_y(t)}{H} \]  
(A.13)

\[ \tilde{Q}_T^*(t) = K_{T^*}\eta_T(t) = \int_0^H \tilde{p}_T(z,t)\phi_T(z)dz \approx 0.7\tilde{M}_T(t) \]  
(A.14)

where \( \tilde{p}_x, \tilde{p}_y, \) and \( \tilde{p}_T \) are the equivalent distributed forces corresponding to \( \tilde{Q}_x^*(t), \tilde{Q}_y^*(t) \) and \( \tilde{Q}_T^*(t) \) respectively, and \( \tilde{M}_x(t), \tilde{M}_y(t) \) and \( \tilde{M}_T(t) \) are the dynamic base bending moments and base torque, i.e. the base bending moments and torque that incorporate the dynamic responses of the building. The following equations can then be derived from Eqs. (A.6)-(A.8) and (A.12)-(A.14):
By applying the random vibration analysis, the resonant component of the RMS dynamic base bending moments and torque can be approximately evaluated as (Kareem and Zhou 2003)

\[
S_{M_x}(f) = S_{M_x}(f) |H_y(f)|^2
\]  \hspace{1cm} (A.15)

\[
S_{M_y}(f) = S_{M_y}(f) |H_x(f)|^2
\]  \hspace{1cm} (A.16)

\[
S_{M_T}(f) = S_{M_T}(f) |H_T(f)|^2
\]  \hspace{1cm} (A.17)

\[
\sigma_{MR_x} = \frac{\pi f_y}{4\xi_y} S_{M_x}(f_y)
\]  \hspace{1cm} (A.18)

\[
\sigma_{MR_y} = \frac{\pi f_x}{4\xi_x} S_{M_y}(f_x)
\]  \hspace{1cm} (A.19)

\[
\sigma_{MR_T} = \frac{\pi f_T}{4\xi_T} S_{M_T}(f_T)
\]  \hspace{1cm} (A.20)
Appendix B: WAD Graphical User Interface Screenshots

Figure B.1: Input Guide User Interface

Figure B.2: Output Guide User Interface
## Appendix C: Building Input Criteria in Validation Study

Table C.1: Building Geometric Properties

<table>
<thead>
<tr>
<th>Building Number</th>
<th>$H$ (m)</th>
<th>Aspect Ratio</th>
<th>Slenderness Ratio</th>
<th>Building Number</th>
<th>$H$ (m)</th>
<th>Aspect Ratio</th>
<th>Slenderness Ratio</th>
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</thead>
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<td>4.62</td>
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<td>169</td>
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<td>6.09</td>
</tr>
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<td>1.87</td>
<td>10</td>
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<td>3.21</td>
<td>13</td>
<td>167</td>
<td>1.32</td>
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Appendix D: Peak Base Moment Ratios

Figure D.1: Peak Base Moment Ratio x-Axis
Figure D.2: Peak Base Moment Ratio y-Axis
Figure D.3: Peak Base Torque Ratio
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Appendix E: Peak Acceleration Ratio

Figure E.1: Peak Acceleration Ratio

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Peak Acceleration Ratio

Figure E.1: Peak Acceleration Ratio
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</table>
Name: Bernard Kim

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- University of Western Ontario, London, Ontario, Canada
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