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# Aspects of the dynamic wind-induced response of structures and codification

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**Abstract.** This paper describes the work of the International Association for Wind Engineering Working Group E - Dynamic Response, one of the International Codification Working Groups set up at the Tenth International Conference on Wind Engineering in Copenhagen. Comparisons of gust loading factors and wind-induced responses of major codes and standards are first reviewed, and recent new proposals on 3-D gust loading factor techniques are introduced. Then, the combined effects of along-wind, crosswind and torsional wind load components are discussed, as well as the dynamic characteristics of buildings. Finally, the mathematical forms of along-wind velocity spectra for along-wind response calculation and codification of acceleration criteria are discussed.

**Keywords:** gust loading factor; wind load combination; wind spectra; building dynamic characteristics; acceleration criteria.

## 1. Introduction

This paper discusses some important issues relevant to wind-induced "dynamic response" for use in wind loading codes and standards, on behalf of Working Group WGE, one of the working groups set up by the International Association for Wind Engineering (IAWE) to review and make recommendations for harmonization of international codification for wind loads.

The general recommendations of these working groups, their membership and meetings held are discussed in the introductory paper of this series.

Comparisons of Gust Loading Factors (GLFs) and wind-induced responses of major codes and

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standards, 3-D Gust Effect Factor (GEF) and GLF techniques, combinations of wind load effects, dynamic structural characteristics, wind spectra for along-wind response calculations, and acceleration criteria are discussed in the following sections.

This paper aims at briefly summarizing the recommendations contained in some of the world's major wind codes, and is meant neither to be a detailed review of the literature nor to fully cover the current status. Many items are left in the discussion, but only a few recommendations are given in the concluding remarks.

Each section was written by a separate author with his partial responsibility as follows: Sections 1 and 4 by Y. Tamura; Sections 2 and 3.2 by A. Kareem; Section 3.1 by G. Solari; Section 5 by K.C.S. Kwok; Section 6 by J.D. Holmes; and Section 7 by W.H. Melbourne.

## 2. Comparisons of GLFs and wind-induced responses of major codes and standards

Under the action of wind, typical tall buildings oscillate simultaneously in the along-wind, crosswind and torsional directions. It has been recognized that for many high-rise buildings the crosswind response may exceed the along-wind response in terms of both serviceability and survivability designs (e.g. Kareem 1985). Nevertheless, most codes and standards provide only procedures for the along-wind response and provide little guidance in the crosswind and torsional directions. This is partially attributed to the fact that the crosswind and torsional responses, unlike the along-wind response, result from the aerodynamic pressure fluctuations in the separated shear layers and the wake flow field, which are not amenable to quasi-steady theory. Therefore, to-date no acceptable direct analytical relationship between these pressure fluctuations and the oncoming wind velocity fluctuations has been established. Further, higher-order relationships may exist that are beyond the scope of the current discussion (Gurley, *et al*. 2003). In lieu of theoretical solutions, alternatives based or either an experimentally derived database or through calibration of simplified analytical methods have been advanced. Therefore, for the sake of completeness, a brief review the GLFs and wind induced response provisions of major codes and standards is first presented.

There have been several comparative studies on dynamic responses estimated by codes, e.g., Kijewski and Kareem (1998), Hui, *et al*. (2001), Zhou, *et al*. (2002), and Asami (2002). Different definitions of the design wind speed, e.g., hourly mean, 10-min mean, and 3s-gust, different terrain categories and different mean wind profiles and turbulence intensity/scale profiles make comparisons quite difficult. In these studies, conversions were introduced to facilitate comparisons to enable careful evaluation of the results based on different standards. The differences and commonalities in the provisions for the dynamic response in the major codes and standards are cataloged in these studies. A short summary is presented here.

A comprehensive comparison of the along-wind loads and their effects on tall buildings was conducted utilizing the major international codes and standards: the US Standard (ASCE 7-98 2000), the Australian Standard (AS1170.2 1989), the National Building Code of Canada (NBCC 1995), the Architectural Institute of Japan Recommendations (AIJ-RLB-1993 1993) and the European Standard (Eurocode ENV1991-2-4 1994). These codes and standards utilize some form of the traditional displacement-based gust loading factor for assessing the dynamic along-wind loads and their effects on tall structures. Although derived from a similar theoretical basis, considerable scatter in the predictions of codes and standards have been reported, e.g., Kijewski and Kareem (1998). Unfortunately, globalization of the construction industry and the prospect of developing unified international codes and standards make it increasingly important to better understand the underlying differences, prompting an in-depth investigation by Zhou, *et al*. (2002). It was found that the varying definitions of wind field characteristics, including mean wind velocity profile, turbulence intensity profile, wind spectrum, turbulence length scale, and wind correlation structure, were the primary contributors to the scatter in predicted response quantities. An example presented in Zhou, *et al*. (2002) highlights these differences. Since none of the preceding codes and standards included guidance on the along-wind and crosswind response, no such comparison is reported here. Nonetheless, in light of the sensitivity of the crosswind and torsional response to the approach flow characteristics, especially turbulence, one may expect large variations in the estimated crosswind and torsional response based on code provisions. It is also recommended that such code provisions and those proposed in the following section may only be used to identify potentially sensitive buildings for subsequent detailed wind tunnel study with surrounding buildings.

## 3. 3-D GEF and GLF techniques

### 3.1. 3-D GEF technique and equivalent static wind loads

Original studies on the dynamic along-wind response of structures (Davenport 1967, Simiu 1976, Solari 1982) expressed the maximum displacement as the product of the mean static displacement by a non-dimensional constant coefficient, the Gust Response Factor (GRF), calculated by taking into account only the first vibration mode. These studies also defined the Equivalent Static Wind Load (ESWL) as the force that, statically applied to the structure, produces the maximum displacement. Exploiting structural linearity, this force was assigned as the product of the mean static force by the GRF.

Further researches developed along two distinct lines. The first was aimed at evaluating maximum effects due to along-wind response by using the load response correlation method (Kasperski 1992), the influence function technique (Davenport 1995), suitable along-wind GEF (Zhou and Kareem 2001) and load combination procedures (Holmes 2002). The second generalized the original criteria from the along-wind response to the crosswind and torsional responses, by fitting the results of wind tunnel tests (Tamura, *et al*. 1996 and Zhou, *et al*. 2003) or by developing simplified analytical methods, calibrated on experimental results, based on a so-called 3-D GRF (Piccardo and Solari 2000).

The 3-D GEF technique (Piccardo and Solari 2002) creates a general framework that represents the junction of these two research lines and involves, as particular cases, most of the previous approaches.

Consider a vertical cantilever structure whose height $h$ is much greater than the reference size $b$ of its cross-section. Let $x$, $y$, $z$ be a Cartesian reference system where $z$ coincides with the structural axis and is directed upwards; $x$, $y$ are coplanar with the ground, $x$ being aligned with the mean wind direction. The structure has linear elastic behavior and three un-coupled components of motion, the along-wind and crosswind displacements, towards $x$, $y$, and the $\theta$ torsional rotation, around $z$. Let $e_\alpha$ be a generic load effect at level $r$, associated with the generalized $\alpha$ direction of the motion. Its maximum value and the related ESWL are given by the relationships:

$$\bar{e}_{\alpha,\max}(r) = \bar{e}^x_\alpha(r) G^e_\alpha(r) \tag{1}$$

$$F^e_{\alpha eq}(z, r) = \lambda_\alpha G^e_\alpha(r) \bar{F}_x(z) \tag{2}$$

where $\bar{F}_x$ is the mean static force in the along-wind direction; and $\bar{e}^x_\alpha$ is the static effect due to the application of the generalized force $\lambda_\alpha \overline{F}_x$ in the $\alpha$ direction, where $\lambda_x = \lambda_y = 1$, $\lambda_\theta = b$; $G^e_\alpha$ is a non-dimensional quantity referred to as the 3-D GEF. It is furnished by:

$$G^e_\alpha(r) = \mu^e_\alpha(r) + g^e_\alpha(r)\sqrt{B^e_\alpha(r) + R^e_\alpha(r)} \tag{3}$$

where $\mu^e_\alpha$, $B^e_\alpha$, $R^e_\alpha$ are non-dimensional quantities referred to as, respectively, the static, quasi-static and resonant terms of the effect:

$$\mu^e_\alpha(r) = \frac{\overline{e}_\alpha(r)}{\overline{e}^x_\alpha(r)} \;;\; B^e_\alpha(r) = \frac{[\sigma^e_{B\alpha}(r)]^2}{[\overline{e}^x_\alpha(r)]^2} \;;\; R^e_\alpha(r) = \frac{[\sigma^e_{R\alpha}(r)]^2}{[\overline{e}^x_\alpha(r)]^2} \tag{4}$$

$\overline{e}_\alpha$ is the mean static value of $e_\alpha$; $\sigma^e_{B\alpha}$, $\sigma^e_{R\alpha}$ are the root mean square values of the quasi-static and resonant parts of $e_\alpha$; and $g^e_\alpha$ is the peak factor.

The non-dimensional quantities $\mu^\alpha_e$, $B^e_\alpha$ and $R^e_\alpha$ may be evaluated through wind-tunnel tests or by analytical models. The former provide accurate but burdensome solutions. The latter are suitable for fast preliminary calculations and simplified schemes for code provisions. In this second case, the static and quasi-static terms $\overline{e}_\alpha$, $\overline{e}^x_\alpha$, $\sigma^e_{B\alpha}$ may be evaluated by the influence function technique. The resonant term $\sigma^e_{R\alpha}$ is easily determined assuming that the resonant response in the $\alpha$ direction depends only on the related first vibration mode. Based on such hypotheses, closed form solutions of $G^e_\alpha$ are given in Piccardo and Solari (2002). Solari and Repetto (2002) introduces a method for classifying vertical structures into homogeneous categories, and assessing structural tendencies due to gust buffeting.

It is worth noting that, when $e_\alpha$ denotes generalised displacements, Eqs. (1)-(3) identify with the 3-D GRF technique (Piccardo and Solari 2000). Moreover, focusing attention on the along-wind response, they coincide with Davenport's original formulae Davenport (1967). In other words, the 3-D GEF technique completes the chain of steps taken towards the complete generalisation of the original GRF technique including, in one simple quantity, the 3-D GEF, all the information necessary to determine the 3-D gust-excited load effects on vertical cantilever structures.

It is also worth noting that Eq. (2) defines the ESWLs through one compact formula that implies a unique load pattern, $\overline{F}_x$. In the along-wind and crosswind directions, it is scaled by the along-wind and crosswind GEFs, $G^e_x$ and $G^e_y$, respectively; in the torsional direction, it is multiplied by the equivalent eccentricity $bG^e_\theta$, $G^e_\theta$ being the torsional GEF. This definition is very useful in the engineering sector. Only three static loading conditions, the along-wind force $\overline{F}_x$, the crosswind force $\overline{F}_x$, and the torsional moment $b\overline{F}_x$, are initially applied. The related effects $\overline{e}^x_x$, $\overline{e}^x_y$ and $\overline{e}^x_\theta$ are then derived by equilibrium relationships. Maximum effects are finally obtained by scaling these patterns by the appropriate 3-D GEF (Eq. (3)).

Alternative methods for evaluating the ESWL can be framed into two classes (Repetto and Solari 2004) referred to as the Load Combination (LC) technique and the Global Loading (GL) technique: the former is the synthesis and the advance of a wide literature (Davenport 1995, Zhou, *et al.* 2001 and Holmes 2002) the latter is first derived in Repetto and Solari (2004).

The LC technique assigns the ESWL as a combination of three load distributions related, respectively, to the static, quasi-static and resonant parts of the response. Conventionally, the quasi-static and resonant parts of the ESWL are defined herein as providing rms values. This means that the three parts of the ESWL give rise to load effects such as $\overline{e}_\alpha(r)$, $\sigma^e_{B\alpha}(r)$ and $\sigma^e_{R\alpha}(r)$. The maximum load effect $\overline{e}_{\alpha,\max}(r)$ can be evaluated by Eqs. (1), (3) and (4).

The static and resonant parts of the ESWL are conveniently defined as independent of the load effect considered. This aim is fulfilled, respectively, by the mean static action $\overline{F}_\alpha(z)$ and by the inertial load that produces the rms value of the resonant response related to $\Psi_{\alpha 1}(z)$ (Repetto and Solari 2004):

$$F_{R\alpha,eq}(z) = m_\alpha(z)(2\pi n_{\alpha 1})^2 \overline{d}^x_\alpha(z)\sqrt{R^d_\alpha} \tag{5}$$

where $m_\alpha$, $n_{\alpha 1}$ and $\psi_{\alpha 1}$ are the generalised mass per unit length, the fundamental frequency and the fundamental modal shape in the $\alpha$ direction; $\overline{d}^x_\alpha(z)$ is the reference displacement due to the application of $\lambda_\alpha \overline{F}_x(z)$ in the $\alpha$ direction.

The quasi-static part of the ESWL is more complex to evaluate since no load distribution provides, simultaneously, the full scenario of the quasi-static load effects. Thus, it depends on the load effect considered and such dependence is not unique:

$$F^e_{B\alpha.eq}(r,z) = \lambda_\alpha \overline{F}_x(z)\Delta^e_\alpha(r,z)\sqrt{B^e_\alpha(r)} \tag{6}$$

Assuming $\Delta^e_\alpha = 1$, the quasi-static part of the ESWL is proportional to the unique load distribution $\overline{F}_x(z)$ and represents the quasi-static part of the ESWL given by GEF technique (Eqs. (2) and (4)). Determining $\Delta^e_\alpha$ by the Load Response Correlation (LRC) method (Kasperski 1992), makes Eq. (6) the most probable load distribution for each specified load effect $\sigma^e_{B\alpha}(r)$ (Repetto and Solari 2004). Both these choices give rise to correct values of $\sigma^e_{B\alpha}(r)$: the former is easier; the latter is physically more adequate.

The GL technique provides a global and unique load distribution able to furnish, per each motion direction, a correct scenario of all the structural load effects. This requirement is satisfied by expressing the ESWL through the polynomial expansion (Repetto and Solari 2004):

$$F_{\alpha,eq}(z) = \lambda_\alpha \overline{F}_x(h)\sum_0^n {}_j p_{\alpha j}\left(\frac{z}{h}\right)^j \tag{7}$$

where $p_{\alpha j}$ ($j = 0, 1, 2, \ldots n$) are $(n+1)$ non-dimensional coefficients used to impose that Eq. (7) gives rise to the correct values of $(n+1)$ specified load effects in the $\alpha$ direction.

Analogously, the quasi-static part of ESWL may be expressed as (Repetto and Solari 2004):

$$F_{B\alpha,eq}(z) = \lambda_\alpha \overline{F}_x(h)\sum_0^n {}_j q_{\alpha j}\left(\frac{z}{h}\right)^j \tag{8}$$

where $q_{\alpha j}$ ($j = 0, 1, 2, \ldots n$) are $(n+1)$ non-dimensional coefficients used to impose that Eq. (8) gives rise to the correct values of $(n+1)$ specified quasi-static load effects in the $\alpha$ direction.

Strategies for determining the lowest value of $n$ and the best values of the coefficients $p_{\alpha j}$ and $q_{\alpha j}$ are developed and discussed in Repetto and Solari (2004).

### 3.2. 3-D GLF based on aerodynamic loading database

This section discusses the concept of 3-D GLF for estimating dynamic load components in three directions based on an aerodynamic loading database in the "gust loading factor" format that has generally been used for the along-wind response (Zhou, *et al.* 2003). It is envisaged that the new formulation will be most appropriate for inclusion in codes and standards and also serves as a convenient format for the interpretation of wind tunnel test results.

The proposed 3-D GLF is an extension of the GLF concept based on the base bending moment or base torque response defined as:

$$G = \hat{\boldsymbol{M}} / \overline{M}' \tag{9}$$

where $G$ = GLF; and $\overline{M}'$ = reference mean base bending moment or base torque, which can be computed for the sway and torsional modes, respectively, by

$$\overline{M}'_{x,y} = \int_0^h \overline{P}(z) \cdot z \cdot dz \tag{10}$$

$$\overline{M}'_z = \int_0^h \overline{P}(z) \cdot (0.04b) dz \tag{11}$$

where $\overline{P}(z)$ = mean along-wind load at any height $z$ above the ground; and $h$ and $b$ = building height and width normal to the oncoming wind, respectively. Subscripts $x$, $y$ and $z$ represent the along-wind, crosswind and torsional directions, respectively. If not specifically indicated, the given formulation is applicable to all three directions. The reference mean base moment in Eq. (10) in the crosswind and the base torque in Eq. (11) are not the actual mean base moments that act on the building.

For convenience, the reference mean base moment in the crosswind is set equal to the along-wind mean base moment. $\hat{\boldsymbol{M}}$ = peak base bending moment or base torque response which can be expressed as:

$$\hat{\boldsymbol{M}} = \overline{M} + g \cdot \sigma_{\boldsymbol{M}} \tag{12}$$

where $\overline{M}$ = mean base bending moment or base torque; and $g$ = peak factor, which is usually around $3 \sim 4$; and $\sigma_{\boldsymbol{M}} = \sqrt{\int_0^\infty S_{\boldsymbol{M}}(n) dn}$ = rms of the base bending moment and base torque response, and $S_{\boldsymbol{M}}(n)$ = power spectral density (PSD) of the fluctuating base moment or torque response. It has been a general practice to divide the integration term of the fluctuating response into two portions:

$$\sigma_{\boldsymbol{M}} = \sqrt{\sigma_{\boldsymbol{M}B}^2 + \sigma_{\boldsymbol{M}R}^2} \tag{13}$$

in which $\sigma_{\boldsymbol{M}B}$ and $\sigma_{\boldsymbol{M}R}$ = background and resonant components of the base bending moment or base torque response, respectively. Thus, the 3-D GLF can be expressed in the form

$$G = \overline{G} + \sqrt{G_B^2 + G_R^2} \tag{14}$$

where $\overline{G}$, $G_B$ and $G_R$ = mean, background and resonant components of the GLF, respectively, which can be computed by

$$\overline{G} = \overline{M} / \overline{M}' \tag{15}$$

$$G_B = g_B \cdot \sigma_{\boldsymbol{M}B} / \overline{M}' \tag{16}$$

$$G_R = g_R \cdot \sigma_{\boldsymbol{M}R} / \overline{M}' \tag{17}$$

where $g_B = g_u$ = background peak factor or peak factor for the fluctuating wind velocity as suggested in ASCE7-98 (2000). It is important to note that when applying to the along-wind response, the preceding 3-D GLF reduces to exactly the same result as given in a new GLF model by Zhou and Kareem (2001). This new GLF model has the advantage of offering an improved GLF format that reflects more accurately the description of dynamic load effects on structures in comparison with the traditional GLF approach used in current codes and standards. For the along-wind response, the mean component of the GLF is unity; and for the crosswind and torsional response of a symmetrical building, it is usually very small or zero. The calculation for the background and resonant components of the base bending moment response are provided in the following sections.

### 3.2.1. Aerodynamic base moment database

The genesis of aerodynamic base moments involves complex fluid-structure interactions, which can only be evaluated accurately with wind tunnel tests, except in the along-wind direction, where the strip and quasi-steady theories are usually assumed. The base moment in a non-dimensional form can be obtained from the high frequency base balance (HFBB) (Tschanz and Davenport 1983) or simultaneously monitored surface pressure measurements on scaled building models (e.g. Kareem 1982, Ho, *et al*. 1999).

Although a host of HFBB data on a wide variety of structures has been collected in laboratories worldwide, it has not been assimilated and made accessible to the global community to fully realize its potential. Fortunately, the Internet now provides the opportunity to pool and archive the international stores of wind tunnel data. The first step toward an "e-database" of aerodynamic wind loads was introduced by Zhou, *et al*. (2003), based on HFBB measurements on a host of isolated tall building models, and is currently accessible to the worldwide Internet community via Microsoft Explorer at the URL address http://www.nd.edu/~nathaz. Through the use of this interactive portal, users can select the geometry and dimensions of a model building from the available choices and specify an urban or suburban condition. Upon doing so, the aerodynamic load spectra for the along-wind, crosswind or torsional response is displayed with a Java interface permitting users to specify a reduced frequency of interest and automatically obtain the corresponding spectral value. When coupled with the supporting web documentation, examples and concise analysis procedures based on the base bending moment, the database provides a comprehensive tool for computation of wind-induced responses of tall buildings, suitable for possible inclusion in codes and standards as a design guide in the preliminary stages.

In this database, the measured aerodynamic base moments are reduced in the following non-dimensional formats:

$$\sigma_{CM} = \sigma_M / M' \tag{18}$$

$$C_M(f) = (n \cdot S_M(n)) / \sigma_M^2 \tag{19}$$

where $M'$ = reference moment or torque in the test, which is defined by $M'_D = (1/2\rho \overline{U}_h^2 bh^2)$, $M'_L = (1/2\rho \overline{U}_h^2 dh^2)$ and $M'_T = (1/2\rho \overline{U}_h^2 bdh)$ for the along-wind, crosswind and torsional directions, respectively. The non-dimensional data can be directly used in the response analysis of buildings. It is important to note the manner in which the reference moments have been defined in this database, e.g., the crosswind moment is non-dimensionalized with respect to $d$, which is the crosswind face dimension.

### 3.2.2. Evaluation of the 3-D GLF

Given the aerodynamic base moments, the three components of 3-D GLF can be evaluated by substituting Eqs. (18) - (19) into Eqs. (15) - (17) as:

$$\overline{G} = \begin{cases} 1 & \text{for along-wind;} \\ 0 & \text{for crosswind and torsion for symmetric buildings} \end{cases} \tag{20}$$

$$G_B = g_B \cdot \sigma_{CM} \cdot M' / \overline{M}' \tag{21}$$

$$G_R = g_R \frac{\sigma_{CM} \cdot M'}{\overline{M}'} \sqrt{\frac{\pi \cdot C_M(n_1)}{4\zeta_1}} \tag{22}$$

where $\zeta_1$ = damping ratio for the first mode vibration.

### 3.2.3. Application of 3-D GLF in design

Among other advantages, the base moment response based GLF, as outlined here, exhibits a notable feature that the ESWL on a building can be obtained by distributing the base moment response to each floor. For the mean and background components, the ESWLs can be expressed by:

$$\overline{\boldsymbol{P}}_i = \overline{\boldsymbol{M}} \cdot \frac{2 + 2\alpha_V}{h^2} \cdot \left(\frac{z_i}{h}\right)^{2\alpha_V} \cdot \Delta h_i \tag{23}$$

$$\hat{\boldsymbol{P}}_{Bi_{x,y}} = \hat{\boldsymbol{M}}_{B_{x,y}} \cdot \frac{2 + 2\alpha_V}{h^2} \cdot \left(\frac{z_i}{h}\right)^{2\alpha_V} \cdot \Delta h_i \tag{24}$$

$$\hat{\boldsymbol{P}}_{Bi_z} = \hat{\boldsymbol{M}}_{Bz} \cdot \frac{1 + 2\alpha_V}{h} \cdot \left(\frac{z_i}{h}\right)^{2\alpha_V} \cdot \Delta h_i \tag{25}$$

For the resonant components, the ESWL in sway modes is given by:

$$\hat{\boldsymbol{P}}_{Ri_{x,y}} = \hat{\boldsymbol{M}}_{R_{x,y}} \cdot \frac{m_i \varphi_{1i_{x,y}}}{\sum m_i z_i \varphi_{1i_{x,y}}} \tag{26}$$

and the torsional mode by:

$$\hat{\boldsymbol{P}}_{Ri_z} = \hat{\boldsymbol{M}}_{R_z} \cdot \frac{I_i \varphi_{1i_z}}{\sum I_i \varphi_{1i_z}} \tag{27}$$

where $\overline{\boldsymbol{P}}$ = ESWL; $\overline{\boldsymbol{M}} = \overline{G} \cdot \overline{M}'$, $\hat{\boldsymbol{M}}_B = G_B \cdot \overline{M}'$ and $\hat{\boldsymbol{M}}_R = G_R \cdot \overline{M}'$ = mean, background and resonant base moment components, respectively; $\alpha_V$ = wind speed profile exponent; $z_i$ = elevation of the $i$th floor above the ground; $\Delta h_i = z_i - z_{i-1}$ = floor height of the $i$th floor; and $m_i$, $I_i$ and $\varphi_{1i}$ = mass, mass moment of inertia and first mode shape at the $i$th floor height, respectively.

Any wind load effects such as the internal forces in each member, as well as the overall deflection and acceleration, can be computed expediently through a simple analysis utilizing these ESWLs. For example, the acceleration response estimated for the serviceability checking procedure can be evaluated using only the resonant ESWL component. These response components are then combined to obtain the resultant wind load effects as

$$\hat{r} = \bar{r} + \sqrt{\hat{r}_B^2 + \hat{r}_R^2} \tag{28}$$

where $\hat{r}$, $\bar{r}$, $\hat{r}_B$, $\hat{r}_R$ = resultant, mean background and resonant components, respectively, of any wind load effects of concern.

As a result of discussions, WGE members agreed that *gust loading factor calculations in codes should focus on base bending moment rather than on deflections.*

## 4. Combinations of wind load effects

Wind load combinations must be considered in design, especially when wind directionality is taken into account. Melbourne (1975), Vickery and Basu (1984), Solari and Pagnini (1999), Tamura, *et al*. (2002a, 2002b) and Hibi, *et al*. (2003) examined the dynamic characteristics of wind force components and response components and discussed the combinations of wind load effects.

AS1170.2 (1989) gives a formula for peak resultant vector moment, where it is assumed that the peak resultant base moment is equal to the peak along-wind moment when the mean crosswind response is equal to zero and the crosswind dynamic response is less than or equal to the along-wind response. A new standard AS/NZS 1170.2 (2002) gives a formula for estimating the total scalar effect $\varepsilon_t$ such as an axial load in a column as:

$$\varepsilon_t = \varepsilon_{am} + \sqrt{(\varepsilon_{a,p} - \varepsilon_{a,m})^2 + \varepsilon_{cp}^2} \tag{29}$$

where $\varepsilon_{am}$ is the load effect due to the mean along-wind action, $\varepsilon_{ap}$ is that due to the peak along-wind action, and $\varepsilon_{cp}$ is that due to the peak crosswind action. There was an error in the above equation that has been corrected in an Amendment to AS/NZS1170.2. ASCE7-98 (2000) gives simple wind load combinations for buildings higher than 60 ft, where 75% of along-wind load and the same values are simultaneously applied in the crosswind direction, and the torsional load is also taken into account as in its previous version.

The AIJ-RLB-2004 (2004) gives two methods. The first is applicable even for the case without information on crosswind or torsional responses. It proposes a wind load combination factor $\gamma$ defined as:

$$F_y = \gamma F_x \tag{30}$$

which is the crosswind force applied with the design along-wind load. This method is based on the results by Tamura, *et al*. (2002a, 2002b) and Hibi, *et al*. (2003). The combination factor is given as:

$$\gamma = 0.35\frac{d}{b} \tag{31}$$

where $d$ and $b$ are the along-wind and crosswind dimensions of a building plan. The other method gives the load combination factor defined as a function of the correlation coefficient between the crosswind response and the torsional response based on Asami (2000).

## 5. Dynamic structural characteristics

Many modern wind load standards contain procedures for the calculation of dynamic wind load and wind-induced response of wind-sensitive structures, including flexible and lightly damped tall buildings. Typically, wind-sensitive structures are those with a first-mode natural frequency less than 1 Hz and a slenderness (height to breadth or depth) ratio greater than four to five.

The natural frequencies of vibration and structural damping ratios, particularly of the fundamental mode of vibration, are the most important structural parameters in the calculation of the dynamic wind load and wind-induced response of wind-sensitive structures. The natural frequency is used in conjunction with the building dimension and the design wind speed to define the design reduced wind velocity, which in turn specifies the wind excitation energy available to cause resonant response in the building. Through the mechanical admittance function, the degree of dynamic amplification of this available energy into the resonant response depends on the structural damping ratio. Most standards that contain a dynamic calculation procedure provide relatively simple guidelines on the estimation of natural frequencies and suggested values of structural damping ratio to facilitate the design process.

Most standards provide relatively simple equations for the estimation of natural frequencies based on the building height or number of stories. The Australian and New Zealand Standard AS/NZS 1170.2 (2002), Eurocode ENV1991-2-4 (1994), Hong Kong Code of Practice on Wind Effects Draft (1996), and others have adopted Eq. (32) for all building types:

$$n_1 = \frac{46}{h} \tag{32}$$

in which $n_1$ is the natural frequency of the fundamental mode and $h$ is the building height in $m$. The UK Standard BS6399 (1997) uses a similar equation with a coefficient of 60 and a representative height. The American standard ASCE7-98 (2000) also uses a similar equation but adopts different coefficients based on building type. The Chinese Standard GB50009 (2001) uses a simple equation based on the number of stories and also adopts coefficients based on building types. GB50009 also suggests a more refined procedure based on building height and width when the structural form is known. Tamura, *et al.* (2000) has proposed to use the formulae $n_1 = 1/0.015h(=67/h)$ and $n_1 = 1/0.020h(=50/h)$ based on the database on dynamic characteristics of Japanese buildings for the evaluation of habitability to vibration of concrete buildings and steel buildings, respectively. Tamura, *et al.* (2000) has also proposed the formulae $n_1 = 1/0.018h(=56/h)$ and $n_1 = 1/0.024h(=42/h)$ for design of concrete buildings and steel buildings in the large amplitude range near the elastic limit.

A comprehensive review of damping in buildings has been reported by Tamura, *et al.* (2000). A selection of design damping ratios currently used in some countries for the full range of buildings including steel and reinforced concrete buildings is presented in Fig. 1.

Values suggested in some recently revised standards, including AS/NZS 1170.2 (2002) and GB50009 (2001), and proposed revisions such as the AIJ-RLB-2004 (2004) are included in Fig. 1 for comparison.

It is evident that the suggested damping ratios to be used in the design of tall buildings vary

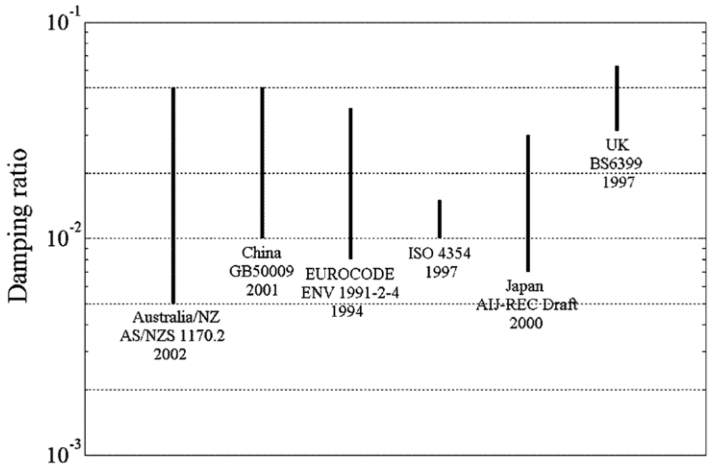


Fig. 1 Design damping ratios for tall buildings

considerably. This is largely due to the complex nature and a lack of understanding of the damping mechanism in tall buildings. The lower limits of values are representative of damping ratios for buildings in serviceability conditions in which the building and its component deformations are well within the elastic range, while the upper limits are representative of damping ratios for building deformation in ultimate limit state conditions. It is noteworthy that those suggested damping ratios are for structural damping only. For tall buildings and other tall flexible structures operating under wind conditions in which amplitude-dependent excitations such as lock-in and galloping become significant, the associated negative aerodynamic damping can lower the overall damping noticeably to cause an increase in dynamic response. These effects are normally outside the scope of wind load standards and specialist advice and/or wind tunnel tests are recommended.

Generally, steel buildings have a lower natural frequency and a lower damping ratio than reinforced concrete buildings of a similar height. However, as expected, the method of estimating natural frequency and in particular the suggested damping ratios vary quite significantly among the different standards. As a result, the calculated wind load and wind-induced response, even for identical buildings, will vary correspondingly, according to the standard used for the calculation.

It was emphasized by WGE that *reliable empirical formulae including the effects of structural type, height, and amplitude should be recommended for damping values.*

## 6. Wind spectra for along-wind response calculation

### 6.1. Mathematical forms

The following mathematical forms for along-wind velocity spectra are currently used in major current, or recent, wind code and standards:

$$\frac{nS_u(n)}{\sigma_u^2} = \frac{4\left(\frac{nl_u}{\overline{U}}\right)}{\left[1 + 70.8\left(\frac{nl_u}{\overline{U}}\right)^2\right]^{\frac{5}{6}}} \quad \text{(von Karman)} \tag{33}$$

$$\frac{nS_u(n)}{\sigma_u^2} = \frac{a_1\left(\frac{nl_u}{\overline{U}}\right)}{\left[1 + a_2\left(\frac{nl_u}{\overline{U}}\right)\right]^{\frac{5}{3}}} \quad \text{(modified Kaimal)} \tag{34}$$

$$\frac{nS_u(n)}{\sigma_u^2} = \frac{0.67\left(\frac{nL}{\overline{U}}\right)^2}{\left[1 + \left(\frac{nL}{\overline{U}}\right)^2\right]^{\frac{4}{3}}} \quad \text{(Davenport)} \tag{35}$$

$$\frac{nS_u(n)}{\sigma_u^2} = \frac{0.6\left(\frac{nL}{\overline{U}}\right)^2}{\left[2 + \left(\frac{nL}{\overline{U}}\right)^2\right]^{\frac{5}{6}}} \quad \text{(Harris)} \tag{36}$$

These forms differ primarily in the exponent used in the denominator. The length scale, $l_u$, is the integral length scale. $L$ is a different length scale used specifically in the Davenport and Harris spectra. The Harris form can be shown to be nearly identical to the von Karman form if $L$ is taken as 11.9 $l_u$. Table 1 summarizes which standards and codes have used the various forms of along-wind spectra.

Table 1 Spectral densities used in various major wind codes or standards

| Code/Standard | Form of along-wind spectral density used |
|---|---|
| AIJ-RLB-1993 (1993), AIJ-RLB-2004, (2004), Japan | von Karman |
| AS1170.2 (1989), Australia | Harris |
| AS/NZS1170.2 (2002), Australia/New Zealand | von Karman |
| ASCE7-98 (2000), United States | modified Kaimal |
| Eurocode prEN1991-2-4 (2004), Europe | modified Kaimal |
| NBCC (1995), Canada | Davenport |

### 6.2. Properties of spectra

The following properties of any empirical mathematical form of spectral density are desirable, if not essential:

$$\text{i)}\ \int_0^\infty \frac{S_u(n)}{\sigma_u^2}\,dn = 1 \tag{37}$$

– this requirement ensures that the area under the $S_u(n)$ versus $n$ graph equals the total variance of fluctuating wind speed, and is then consistent with the turbulence intensity:

$$\text{ii)}\ S_u(0) = 4\sigma_u^2\frac{l_u}{\overline{U}} \tag{38}$$

– this can be derived from the Wiener-Khintchine relations between autocorrelation and spectral density, where the integral scale, $l_u$, is defined by the area under the auto correlation function:

$$l_u = \overline{U}\cdot\int_0^\infty R(\tau)d\tau \tag{39}$$

$$\text{iii)}\ \frac{nS_u(n)}{\sigma_u^2} = A\left(\frac{nl_u}{\overline{U}}\right)^{-\frac{2}{3}}\ \text{at high frequencies, where } A \text{ is in the range 0.10 to 0.15} \tag{40}$$

– this is derived from dimensional analysis for the inertial sub-range, and measurements of atmospheric turbulence to determine the factor, $A$. This frequency range includes the natural frequency range of most tall buildings.

Traditionally, property (ii) has been regarded as less important than the other two. The von Karman form (Eq. (33)) satisfies all three properties, (i), (ii) and (iii). In the case of (iii), the constant $A$ is 0.12.

The modified Kaimal form, as given in Eq. (34), satisfies property (i) only if the ratio $a_1/a_2$ is 2/3 (0.67). Property (ii) is also satisfied if $a_1$ is 4, and $a_2$ is 6. However, for those values, the constant $A$ in Eq. (40) is 0.20, i.e., outside the desirable range. In the Eurocode prEN1991-2-4 (2004), the values of $a_1$ and $a_2$ selected are 6.8 and 10.2, respectively - then (i) is satisfied but (ii) is not; and (iii) is satisfied with a value of $A$ of 0.14.

In the case of the Davenport spectrum, (i) is satisfied. (ii) is not satisfied since $S_u(0)$ is 0. Assuming, that the length scale, $L$, used in this spectrum is 11.9 $l_u$, then property (iii) is satisfied with $A$ equal to 0.13.

Although all three spectral forms in current use satisfy the more important properties (i) and (iii), *only the von Karman form satisfies all three properties*, and is the form preferred by WGE. This form is currently used in the AIJ-RLB-2004 (2004) and the AS/NZS1170.2 (2002).

## 7. Codification of acceleration criteria in Australia

A general review of comfort criteria has been given by Melbourne (1998), and a much broader review has been under preparation by the Council for Tall Buildings and the Urban Habitat, but as yet this has not been published. A number of full-scale observations and experiments from the

1970s and 1980s to help define unacceptable acceleration levels have been summarized by Irwin (1979 and 1986). Conclusions from this work have been used to form Irwin's E2 Curve which subsequently was adopted and defined as acceleration criteria for occupancy comfort in ISO 6897 (1984). In the late 1990s a very comprehensive study of human response and acceptance of acceleration levels was carried out by Denoon (2000).

### *7.1. Australian recommendations*

Early recommendations for acceleration criteria were given by Melbourne (1983) and Melbourne and Cheung (1988). The later recommendations showed good agreement with Irwin's E2 Curve and in the Commentary to the Australian Wind Code, AS1170.2. (1989) the ISO/Irwin E2 Curve was recommended for the 5 year return period standard deviation form of the criteria along with the recommendations by Melbourne and Cheung (1988) for peak accelerations and other return periods. In summary this is given in Fig. 2 and defined by equations as follows.

The ISO/Irwin E2 Curve as standard deviation acceleration criteria for the 5-year return period is given by

$$\sigma_{\ddot{x}} = \exp(-3.65 - 0.41 \ln n) \quad (\mathrm{m/s^2}) \tag{41}$$

and the Melbourne and Cheung (1988) extension to cover peak acceleration criteria as a function of return period is given by

$$\hat{\ddot{x}} = \sqrt{2 \ln nT}\left(0.68 + \frac{\ln R}{5}\right)\exp(-3.65 - 0.41 \ln n) \quad (\mathrm{m/s^2}) \tag{42}$$

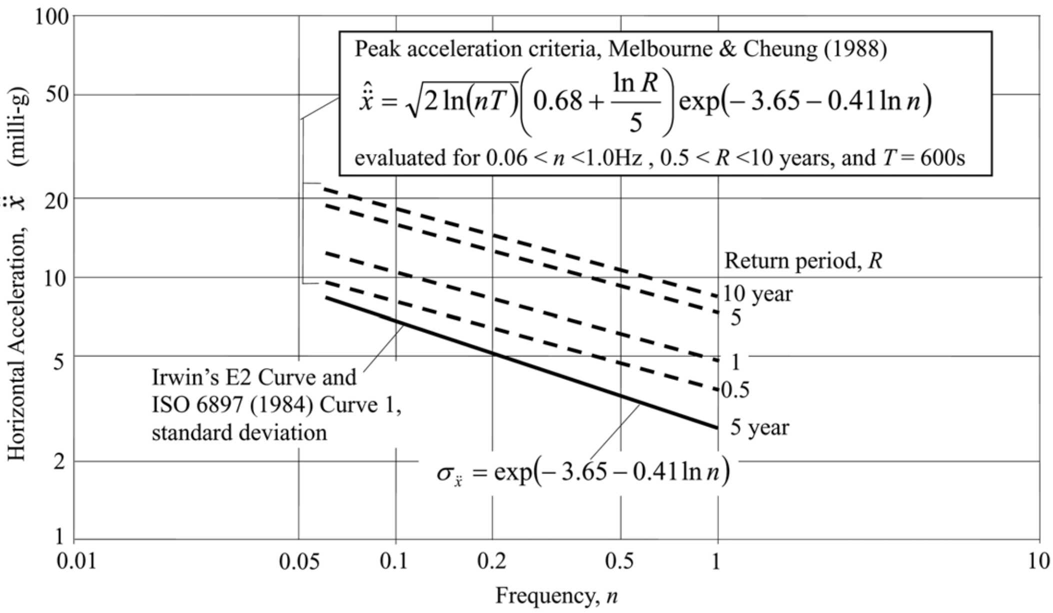


Fig. 2 Horizontal acceleration criteria for occupancy comfort in buildings

where $\sigma_{\ddot{x}}$ is standard deviation of acceleration in the horizontal plane, $n$ is frequency of oscillation with an approximately normal distribution of amplitude, $T$ is duration in seconds, and $R$ is return period in years.

The criteria recommended in the Commentary to the Australian Wind Code AS1170.2 (1989) do not differentiate between different types of building occupancy.

### 7.2. Denoon's full scale experiments

Denoon (2000) undertook a 4 year term of full-scale measurements of the wind induced dynamic responses of the Sydney and Brisbane Airport Control Towers. Surveys were conducted of the occupants of both towers. Very wide inter- and intra-subject differences to wind-induced motion were measured and were found to mask other effects resulting from a number of other factors such as thermal comfort, busyness, and posture. An exit survey found that large sections of the population, who had previously complained about the motion environment, had habituated to the motion, particularly when reassured as to the structural integrity of the tower. It was also found that those interviewees who believed that perceptible accelerations should not be allowed to occur in their workplace were not directly related to those who actually complained about the motion. Experiments were also conducted in a motion simulator. No significant effects of motion on cognitive performance were found. Conclusions of the investigations were that a satisfactory wind-induced motion environment in a building can be achieved by paying attention to both magnitudes of motion causing fear and alarm among the occupants, and to the frequency of perceptible motion. A design methodology for assessing the acceptability of wind-induced accelerations in buildings was presented and this was based on the values in ISO 6897 (1984).

### 7.3. Variation with other acceleration criteria

The main variations of the Australian recommendations with other acceleration criteria can be summarized as follows:

In North America general use is made of the acceleration criteria recommended by Isyumov (1995). These give peak acceleration criteria for 1 and 10 year events and differentiate between occupancy with stricter criteria for residential occupancy then getting less strict for hotel and office occupancy, but these criteria do not differentiate as a function of frequency. In Japan recommended acceleration criteria also differentiate between various occupancy, but do differentiate as a function of frequency (AIJ-GBV 1991).

There has been continuous evidence from the late 1960s as to the variation of perception of acceleration with frequency below 1 Hz (and above for that matter) and there appears to be little support for ignoring the effects of frequency when defining acceleration criteria for occupancy comfort in buildings.

The debate concerning differentiation between various types of occupancy seems set to go on for some time. The only observation made from the Australian perspective is that there does not appear to be adequate evidence to justify differentiating between different types of occupancy, in fact there is an argument for the criteria for office occupancy to be more stringent than residential occupancy because of absenteeism in the case of office occupancy versus the acclimatization effect for residential occupancy. One other observation is that in Australia lower levels of damping, between 0.5% and 1%, are recommended when calculating acceleration response from 1 to 5 year return period (serviceability) levels, whereas in North America there has been general use of damping of 1% for steel buildings and 1.5−2% for concrete buildings for these serviceability accelerations. The level of damping used in North America is clearly too high relative to measurements of damping in tall buildings, but use of this does tend to offset the effect of the more stringent acceleration criteria being used in North America. This practice makes it difficult to evaluate the effect of the various levels of acceleration criteria in full scale experience.

## 8. Conclusions

This paper summarized the matters related to wind-induced dynamic responses discussed in WGE, such as comparisons of gust loading factors and wind-induced responses of major codes and standards, recent new proposals on 3-D gust loading factor techniques, the combined effects of along-wind, crosswind and torsional wind load components, the dynamic characteristics of buildings and the mathematical forms of along-wind velocity spectra for along-wind response calculation. A general review of human comfort considerations in tall buildings is also presented with details of Australian practice and a comparison with practices in other regions of the world.

The recommendations agreed by WGE members are as follows:

(1) Gust loading factor calculations in codes should focus on base bending moment rather than on deflections,

(2) Reliable empirical formulae including the effects of structural type, height, and amplitude should be recommended for damping values, and

(3) The von Karman form is generally preferable as the spectrum for along-wind response calculations.

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## References

AIJ-GBV (1991), "Guidelines for the evaluation of habitability to building vibration", Architectural Institute of Japan (English summary: Goto, T., Ohkuma, T., Tamura, Y. and Nakamura, O., Guidelines for the evaluation of habitability to building, Compact Papers, ASCE Structures Congress '92, San Antonio, April 13-15, 1992).

AIJ-RLB-1993 (1993), Recommendations for Loads on Buildings, Architectural Institute of Japan (English version, 1996).

AIJ-RLB-2004 (2004), Recommendations for Loads on Buildings, Architectural Institute of Japan (in Japanese).

AS/NZ1170.2 (2002), Australian/New Zealand Standard, Structural design actions, Part 2: wind actions, Standards Australia & Standards New Zealand.

AS1170.2 (1989), SAA Loading Code, Part 2 - Wind forces, AS1170.2-89, Standards Australia.

Asami, Y. (2000), "Combination method for wind loads on high-rise buildings", *Proceedings of the 16th National Symposium on Wind Engineering*, Tokyo, Japan, 531-534 (in Japanese).

Asami, Y. (2002), "Comparisons of gust loading factors in codes and standards", Japanese Domestic Committee on TC98/SC3/WG2 for ISO4354 Revision, 5.

ASCE7-98 (2000), "Minimum design loads for buildings and other structures", *American Society of Civil Engineers*, Reston, VA.

Boggs, D.W. and Peterka, J.A. (1989), "Aerodynamic model tests of tall buildings", *J. Eng. Mech.*, ASCE,

**115**(3), 618-635.
BS 6399 (1997), Part 2, Loading for building Structures. Part 2. Code of practice for wind loads, British Standards Institution.
Davenport, A.G. (1967), "Gust loading factors", *J. Struct. Div.*, ASCE, **93**, 11-34.
Davenport, A.G. (1995), "How can we simplify and generalize wind loads", *J. Wind Eng. Ind. Aerodyn.*, **54-55**, 657-669.
Denoon, R.O. (2000), "Designing for wind-induced serviceability accelerations in buildings", PhD Thesis, University of Queensland.
Eurocode ENV1991-2-4 (1994), EUROCODE 1: Basis of Design and Actions on Structures, Part 2.4: Wind Actions, CEN/TC 250/Sc1, 1994.
Eurocode prEN1991-2-4 (2004), EUROCODE 1: Design and Actions on Structures, Part 1.4: General Actions - Wind Actions, CEN TC 250.
GB50009 (2001), Chinese Standard for wind loads.
Gurley, G., Kijewski, T. and Kareem, A. (2003), "First- and higher-order correlations determination using wavelet transform", *J. Eng. Mech.*, ASCE, **129**(2), 188-200.
Hibi, K., Tamura, Y. and Kikuchi, H. (2003), "Peak normal stresses and wind load combinations of middle-rise buildings", Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan, 20057, September, 113-114 (in Japanese).
Ho, T.C.E., Lythe, G.R. and Isyumov, N. (1999), "Structural loads and responses from the integration of instantaneous pressure", *Wind Engineering into the 21st Century*, **3** (editors: Larsen, Larose and Livesey).
Holmes, J.D. (2002), "Effective static load distributions in wind engineering", *J. Wind Eng. Ind. Aerodyn.*, **90**, 91-109.
Hong Kong Code of practice on wind effects (1996), Buildings Department, Draft.
Hui, Z., Bachmann, A. and Graubner, C.A. (2001), "Wind loads on high-rise structures - Comparative study of code provisions according to EUROCODE1 and Chinese Code", *Proc. APCWE V*, Kyoto, Japan, 613-616.
Irwin, A.W. (1979), "Human response to dynamic motion of structures", *The Structural Engineer*, **56A**(9), 237-244.
Irwin, A.W. (1986), "Motion in tall buildings", *Proceedings of Conf. Tall Buildings and Urban Habitat "Second Century of the Skyscraper"*. Van Nostrand, Chicago, 759-778.
ISO 6897 (1984), Guidelines for the Evaluation of the Response of Occupants of Fixed Structures, Especially Buildings and Off-Shore Structures, to Low Frequency Horizontal Motion (0.063 to 1 Hz), Geneva.
Isyumov, N. (1995), "Motion perception tolerance and mitigation", *Proc. 5th World Congress of Council on Tall Buildings and Urban Habitat*, Amsterdam.
Kareem, A. (1982), "Measurements of total dynamic loads from surface pressures", *Proceedings of the International Workshop on Wind Tunnel Modeling Criteria and Techniques*. Ed. T. A. Reinhold; National Bureau of Standards, Gaithersburg, Maryland, Cambridge University Press.
Kareem, A. (1985), "Lateral-torsional motion of tall buildings to wind loads", *J. Struct. Eng.*, ASCE, **111**(11), 2479-2496.
Kasperski, M. (1992), "Extreme wind load distributions for linear and nonlinear design", *Eng. Struct.*, **14**, 27-34.
Kijewski, T. and Kareem, A. (1998), "Dynamic wind effects: a comparative study of provisions in codes and standards with wind tunnel data", *Wind and Struct., An Int. J.*, **1**(1), 77-109.
Melbourne, W.H. (1975), "Probability distributions of response of BHP house to wind action and model comparisons", *J. Indus. Aerody.*, **1**(2), 167-175.
Melbourne, W.H. (1983), "Notes on recommendations on acceleration criteria for occupancy comfort in tall structures", Commercial Wind Tunnel Investigation Reports, MEL Consultants.
Melbourne, W.H. (1998), "Comfort criteria for wind-induced motion in structures", *Struct. Eng. Int.*, 40-44.
Melbourne, W.H. and Cheung, J.C.K.C. (1988), "Designing for serviceable accelerations in tall buildings", *4th Intl Conf. on Tall Buildings*, Hong Kong and Shanghai, 148-155.
NBCC (1995), User's Guide-NBC1995 Structural Commentaries (Part 4).
Piccardo, G. and Solari, G. (2000), "3-D wind-excited response of slender structures: Closed form solution", *J. Struct. Eng.*, ASCE, **126**, 936-943.
Piccardo, G. and Solari, G. (2002), "3-D gust effect factor for slender vertical structures", *Probabilistic Eng.*

*Mech.*, **17**, 143-155.
Repetto, M.P. and Solari, G. (2004), “Equivalent static wind actions on vertical structures”, *J. Wind Eng. Ind. Aerodyn.*, **92**, 335-357.
Simiu, E. (1976), “Equivalent static wind loads for tall buildings design”, *J. Struct. Div.*, ASCE, **102**, 719-737.
Solari, G. (1982), “Alongwind response estimation: closed form solution”, *J. Struct. Div.*, ASCE, **108**, 225-244.
Solari, G. and Pagnini, L.C. (1999), “Gust buffeting and aeroelastic behaviour of poles and monotubular towers”, *J. Fluid and Struct.*, **13**, 877-905.
Solari, G. and Repetto, M.P. (2002), “General tendencies and classification of vertical structures under wind loads”, *J. Wind Eng. Ind. Aerodyn.*, **90**, 1299-1319.
Tamura, Y., Kawai, H., Uematsu, Y., Marukawa, H., Fujii, K. and Taniike, Y. (1996), “Wind loads and wind-induced response estimations in the Recommendations for loads on buildings”, *AIJ 1993, Engineering Structures*, **18**, 399-411.
Tamura, Y., Kikuchi, H. and Hibi, K. (2002a), “Quasi-static wind load combinations for low- and middle-rise buildings AGD2002 symposium preprints”, *Engineering Symposium to Honor Alan G. Davenport for His 40 Years of Contributions*, The University of Western Ontario, Canada, C3-1 - C3-12.
Tamura, Y., Kikuchi, H. and Hibi, K. (2002b), “Quasi-static wind load combinations for buildings”, *The Second International Symposium on Wind and Structures*, Busan, Korea, August 21-23, 61-68.
Tamura, Y., Suda, K. and Sasaki, A. (2000), “Damping in buildings for wind resistant design”, *International Symposium on Wind and Structures for the 21st Century*, Cheju, Korea, 115-130.
Tschanz, T. and Davenport, A.G. (1983), “The base balance technique for the determination of dynamic wind loads”, *J. Wind Eng. Ind. Aerodyn.*, **13**, 429-439.
Vickery, B.J. and Basu, R.I. (1984), “The response of reinforced concrete chimneys to vortex shedding”, *Eng. Struct.*, **6**, 324-333.
Zhou, Y. and Kareem, A. (2001), “Gust loading factor: new model”, *J. Struct. Eng.*, ASCE, **127**(2), 168-175.
Zhou, Y., Kareem, A. and Gu, M. (2002), “On the mode shape corrections for wind load effects on tall buildings”, *J. Eng. Mech.*, ASCE, **128**(1), 15-23.
Zhou, Y., Kijewski, T. and Kareem, A. (2002), “Along-wind load effects on tall buildings: a comparative study of major international codes and standards”, *J. Struct. Eng.*, ASCE, **128**(6), 788-796.
Zhou, Y., Kijewski, T. and Kareem, A. (2003), “Aerodynamic loads on tall buildings: an interactive database”, *J. Struct. Eng.*, ASCE, **129**(3), 394-404.