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The Use of Wind Tunnel Measurements in Building Design

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1. Introduction

The ASCE 7-10 Standard contains provisions on the use of the wind tunnel, but those provisions are incomplete. For this reason estimates of structural response to wind can vary significantly depending upon the laboratory that provides them. This has been the case, for example, for New York City's World Trade Center's twin towers, for which such differences have exceeded 40 % (NCSTAR 1-2, Appendix D, 2004). While the emphasis here is on the testing of buildings in wind tunnels, it is therefore necessary to review the inter-related elements involved in the estimation of the response of buildings to wind, namely, micrometeorology, aerodynamics, similitude, wind climatology, statistics, and structural reliability. The chapter also discusses the validation of wind tunnel measurements, and their application to low-rise and tall buildings, and concludes with a description of a time-domain method for designing structural members known as Database-Assisted Design. For additional materials on wind tunnel testing the reader is referred to, e.g., ASCE (1999), Reinhold (1982), and Simiu (2011).

2. Micrometeorology

Estimates of aerodynamic pressures and forces depend upon the features of the atmospheric flow being adequately simulated in the wind tunnel. This section briefly reviews the description of atmospheric flows affecting buildings and other structures in strong winds. The description includes the characterization of mean wind profiles (Sect. 2.1) and of atmospheric turbulence (Sects. 2.2-2.5).

2.1 The atmospheric boundary layer and the mean wind profile

Before the 1960's aerodynamic tests of buildings were typically conducted in wind tunnels with uniform flow. Later measurements were made in boundary-layer wind tunnels, thanks to the influence of Jensen (1954), who rediscovered Flachsbart's observation, made in 1932 (Flachsbart, 1932), that wind pressures in shear flows can differ markedly from measurements in uniform flow. In this section we discuss the effects of the wind tunnel flow features on the aerodynamic effects of interest.

The mean wind profile in horizontally homogeneous terrain can be described by the power law, characterized by its exponent α that depends on terrain roughness (Hellman, 1916, Davenport, 1965):

$$\frac{U(z_1)}{U(z_2)} = \left(\frac{z_1}{z_2}\right)^{\alpha} \quad (1)$$

where $U(z_1)$and $U(z_2)$ are the wind speeds at elevations z_1 and z_2. An alternative description of the mean wind profile in horizontally homogeneous terrain is the logarithmic law, characterized by the surface roughness length z_0:

$$U(z) = \frac{1}{k} u_* \ln \frac{z}{z_0} \quad (2)$$

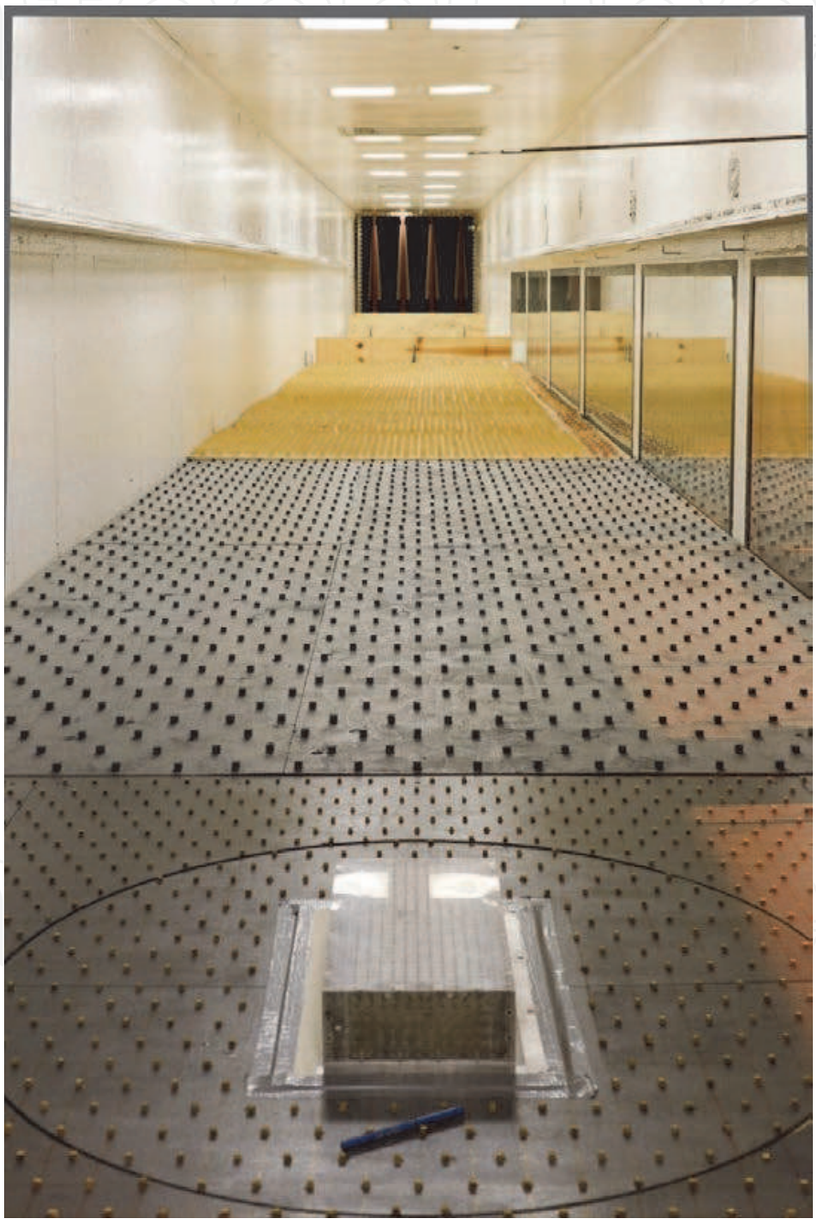


Fig. 1. Meteorological wind tunnel, Wind Engineering Laboratory, Colorado State University. Model and turntable are in the foreground, and spires are in the background (courtesy of Professor Bogusz Bienkiewicz; photo by Gregory E. Stace).

where $k \approx 0.4$, u_* is the flow shear velocity, and $U(z)$ is the wind speed at elevation z. For strong winds the applicability of the logarithmic law up to elevations of about 400 m has been established theoretically by Csanady (1967) (for additional references and details see also Simiu, 1973, Simiu & Scanlan, 1996) and by measurements in the atmosphere by Powell et al. (2003). These results supersede the earlier belief (Davenport, 1965) that the logarithmic law is valid for any wind speed up to about 50 m elevation. Typical values of z_0 range from 2 m to 0.005 m depending on the exposure category (Table C26.7-1, ASCE 7-10).
Over a rough floor, a tunnel length of 20 m to 30 m is required to develop a boundary layer of 0.5 m to 1 m (Marshall, 1984). To increase the depth of the boundary layer, wind tunnels make use of passive devices such as grids, barriers, fences and spires (Fig. 1). In general, similitude in the turbulence of air flows in the natural and the experimental settings is not achieved, even for long wind tunnels, and especially for short (≈ 5 m) ones. Cermak (1982) discusses the flow features downwind of a floor covered with different kinds of roughness elements.

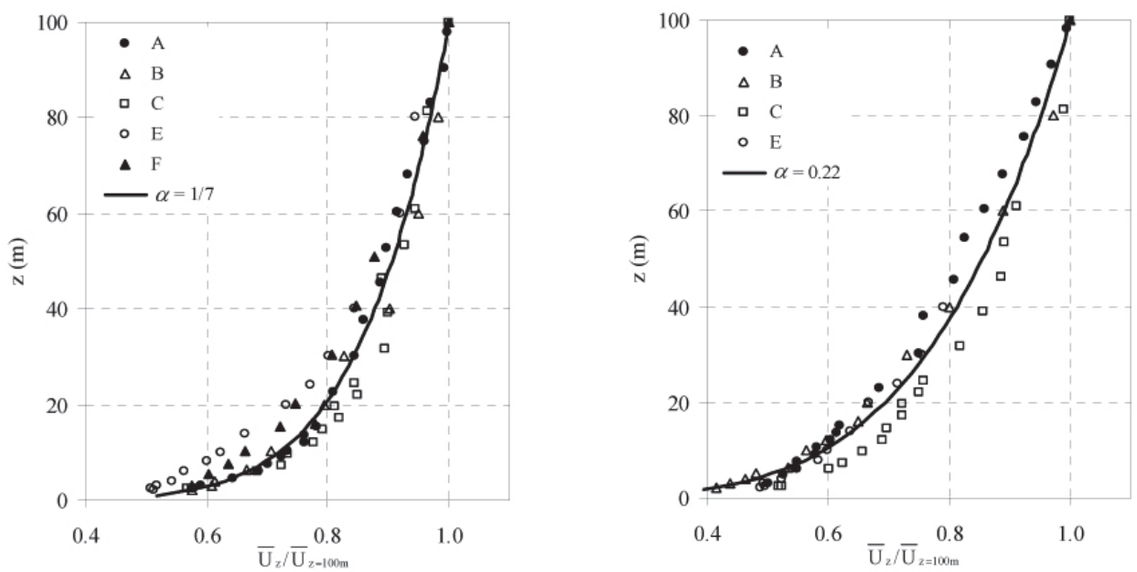


Fig. 2. Wind speed profiles in simulations by wind tunnels participating in the Fritz et al. (2008) comparison (Bienkiewicz et al., 2009). Open exposure refers to flat open country, grasslands, and water surfaces, with scattered obstructions less than 9 m high. Suburban exposure refers to wooded areas or other terrains with numerous, closely spaced single family dwellings.

The various experimental set-ups used by various laboratories to simulate atmospheric flows can result in fairly widely varying properties of the respective flows. Laboratories that participated in an international comparison of wind tunnel estimates of wind effects on low-rise buildings (Fritz et al., 2008) achieved mean wind profiles with power law exponents α that varied between 0.139 and 0.191 (typical target value 1/7 = 0.143) and between 0.165 and 0.234 (target value 0.22) for open and suburban exposures, respectively (Bienkiewicz et al., 2009), see Fig. 2. These differences contributed to the significant discrepancies among the respective values of the wind effects of interest.

2.2 The turbulence intensity

The turbulence intensity $I(z)$ at elevation z corresponding to the longitudinal flow fluctuations (i.e., the fluctuations $u(z)$ in the mean speed direction) is defined as the ratio between the fluctuations' root mean square and the mean wind speed $U(z)$:

$$I(z) = \frac{\sqrt{\overline{u^2(z)}}}{U(z)} \tag{3}$$

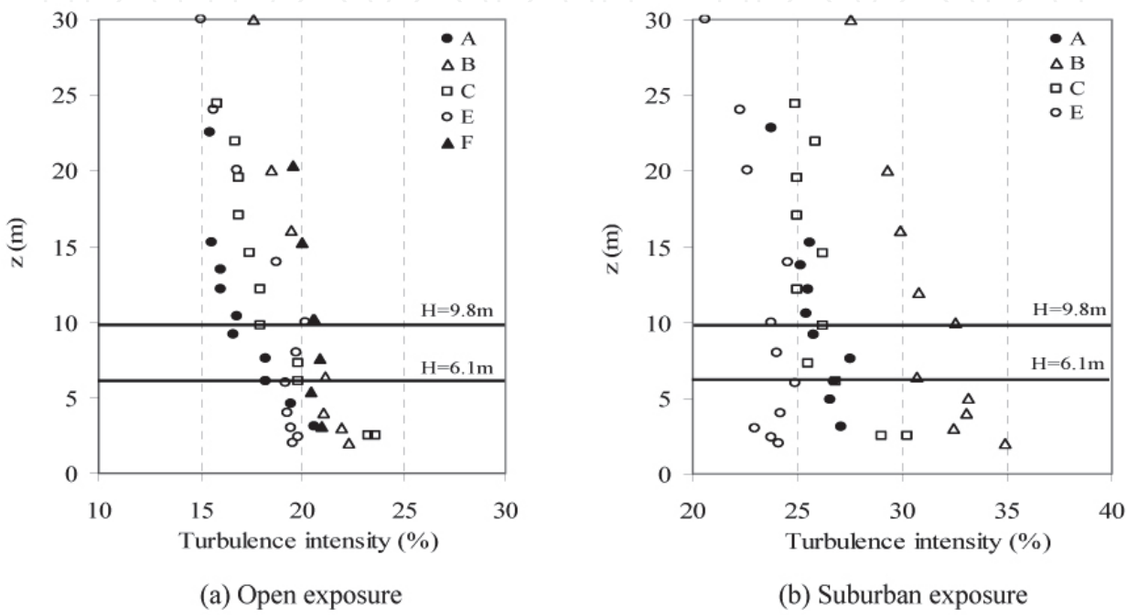


Fig. 3. Turbulence intensities in simulations by wind tunnels participating in the Fritz et al. (2008) comparison (Bienkiewicz et al., 2009).

Similar definitions hold for flow fluctuations in the lateral and vertical directions. Longitudinal turbulence intensities achieved in flows simulated by six laboratories (Fritz et al., 2008) exhibited strong variations, especially for suburban exposure (Fig. 3). For structures with equivalent height of 10 m, $I(z)$ ranges from 0.30 to 0.15 depending on the exposure (ASCE 7-10 Sect. 26.9.4).

2.3 The integral turbulence length

The integral turbulence lengths of the longitudinal flow velocity fluctuations at a point are measures of the average spatial dimensions of those fluctuations. Similar definitions hold for the lateral and vertical flow fluctuations (see, e.g., Simiu & Scanlan (1996) for details). The larger the turbulence scale, the larger is the building dimension affected by the corresponding turbulent fluctuations. For example, a sufficiently large longitudinal integral turbulence scale of the longitudinal turbulent fluctuations means that, if the flow is normal to the windward face of a structure, those fluctuations can affect both its windward and leeward faces. A large lateral scale of the longitudinal turbulent fluctuations means that those fluctuations impinge almost simultaneously over a relatively large area normal to the mean wind speed, resulting in correspondingly large longitudinal fluctuating wind loads.

2.4 The spectral density (or spectrum)

The spectral density (or spectrum) of the longitudinal velocity fluctuations provides a measure of the strength of the fluctuations' frequency components. It is a plot representing the contributions of components with various frequencies to the variance of the fluctuations. Similar definitions hold for lateral and vertical fluctuations. Note that the turbulence intensity and the integral turbulence length are related to the spectral density. In practice the spectra of the turbulent fluctuations cannot be reproduced in civil engineering wind tunnels owing partly to the violation of the Reynolds number by several orders of magnitude, a fact that prevents the simulation of high-frequency velocity fluctuations, and partly to the difficulty of achieving large integral turbulence scales in the laboratory.

The Reynolds number is a measure of the ratio of inertial to viscous forces and is defined as:

$$R_e = \frac{\rho UL}{\mu} = \frac{UL}{\nu} \tag{4}$$

where U is the velocity, L a typical surface dimension, ρ the density, μ the viscosity and ν the kinematic viscosity of the fluid ($\nu = \mu/\rho$). Tennekes & Lumley (1964) suggest a Reynolds number of the order of 10^5 to ensure the existence of an inertial subrange in the turbulent flow generated in the wind tunnel. Turbulent velocity fluctuations can be represented by eddies of various wavelengths, and the inertial subrange is the portion of the spectrum in which eddy motion may be determined by the rate of energy transfer from larger eddies to smaller ones independently of viscosity (Kolmogorov's second hypothesis).

The *cross-spectral density* of longitudinal velocity fluctuations at two points is an approximate measure of the degree of coherence between the respective fluctuations. Similar definitions apply to lateral and vertical fluctuations. *For small structures*, (e.g., typical homes) for which the turbulence length scales of interest are sufficiently large in relation to the structure's dimensions, the bulk of the fluctuating longitudinal wind speed components may be assumed to be almost perfectly coherent over lengths comparable to the dimensions of the structures' exterior faces. This observation allows the use in the wind tunnel of flows from which the low-frequency fluctuations present in the atmosphere are eliminated and are replaced by an increment in the mean wind speed (Simiu et al., Fu et al., in press).

In addition to inducing resonant fluctuations in flexible structures, *high-frequency turbulent fluctuations* have an important aerodynamic effect insofar as they transport across separation layers particles with high momentum from zones outside the separation bubbles, thereby promoting flow reattachment and affecting suctions in separation zones (Simiu & Miyata, 2006). Because, as was mentioned earlier, in commercial wind tunnels, the Reynolds number is orders of magnitude smaller than at full scale, the viscous stresses within the small (high-frequency) eddies of the laboratory flow are higher. The wind tunnel counterparts of full-scale high-frequency fluctuations are therefore partly suppressed by those stresses. This can affect significantly the extent to which laboratory and full-scale suctions are similar, especially in flow separation regions where the suctions are strong. Indeed, measurements have shown that, in zones of strong suctions, absolute values of pressure coefficients are far lower in the wind tunnel than at full scale (Fig. 4).

2.5 Wind speeds as functions of averaging times

The *relation between wind speeds averaged over different time intervals* (e.g., the ratio between wind speeds averaged over 3 s and wind speeds averaged over 10 min) varies as a function of the time intervals owing to the presence of turbulence in the wind flow.

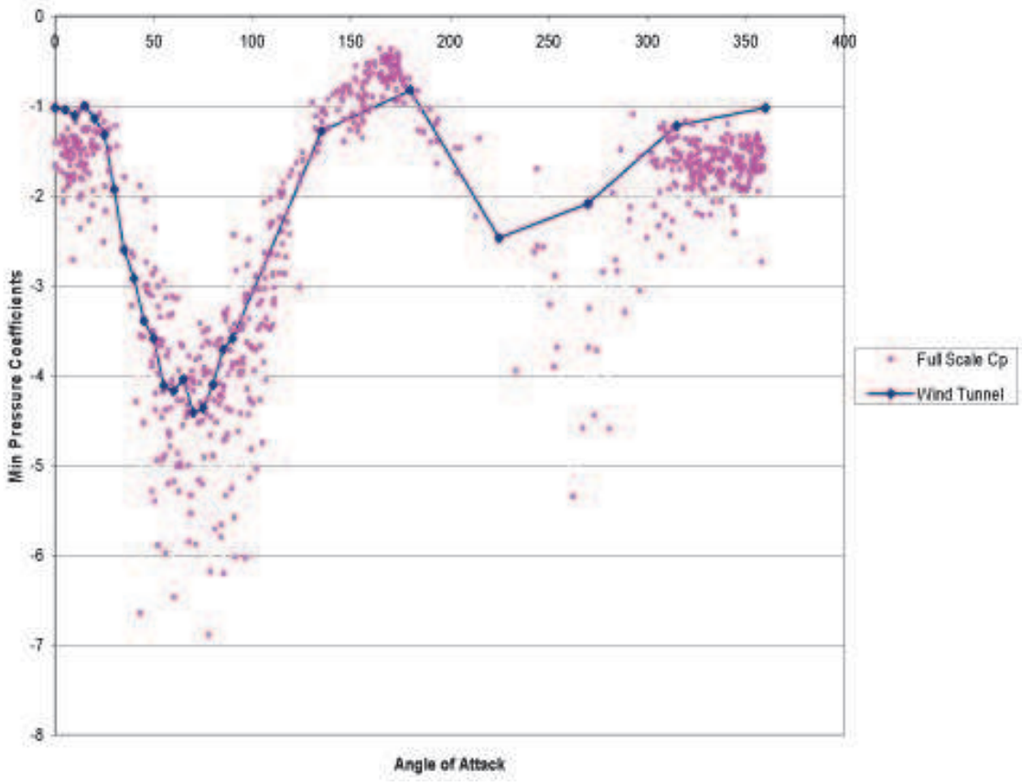


Fig. 4. Pressure coefficients measured at building corner, eave level, Texas Tech University Experimental Building (Long et al., 2006).

The flow characteristics discussed in Sections 2.1 through 2.5 depend to a significant extent upon whether the storm to which the structure is subjected is of the large-scale extratropical (synoptic) type, or a hurricane, a thunderstorm, a chinook wind, and so forth. In current commercial practice the type of flow being simulated is the atmospheric boundary layer typical of synoptic storms (straight line winds), and it is assumed that simulations in this type of flow are adequate even if the structure is subjected to other types of storm.
In wind engineering practice it is important to remember that the parameters of any given model of the wind flow are characterized by uncertainties in the sense that they can vary from storm to storm. Such variability should be accounted for in any uncertainty analysis of the wind effect estimates.

3. Aerodynamics and similitude

Although computational fluid dynamics has made tremendous progress in the last decade thanks to high speed computing, its use has not reached routine level in the structural design of buildings, and its predictions need to be verified by experiments. Wind tunnel testing remains the primary tool for determining wind pressures on buildings. Aerodynamic measurements can be performed simultaneously at large numbers of ports by using current pressure measurement capabilities. The quantities of interest are primarily pressure coefficients for mean pressures and for peak pressures (see, e.g., Fig. 4). This state of affairs is changing, owing in part to approximate methods that replace low-frequency fluctuations by increments in the mean speed, as discussed in Sect. 2.4.

Proper length scaling in boundary-layer wind tunnel requires:

$$\left(\frac{L_x}{D}\right)_m = \left(\frac{L_x}{D}\right)_p \tag{5}$$

where D is a characteristic length of the structure being tested, and subscripts m and p refer to the model and the prototype respectively. L_x is the longitudinal integral turbulence scale, or length of the fluctuating longitudinal component of the wind speed. It is difficult to produce large values of L_{xm} in wind tunnels, and this can be a limiting factor for the size of models testable in the laboratory.

As explained in the preceding section, it is in current practice desirable to ensure similitude between prototype and model for the variations with height of mean speed and turbulence effects, measured by turbulence intensities, integral scales, and spectral characteristics. The following numbers are important in aerodynamics and scaling.

3.1 Reynolds number

Since flow separation occurs at sharp corners regardless of the value of the Reynolds number, this phenomenon is assumed to be modeled correctly in a wind tunnel. Violation of Reynolds number scaling, however, does affect the flow reattachment in long, bluff bodies, and this affects the aerodynamic pressures. For curved bodies, the boundary layers at high Reynolds number tend to be turbulent, and it is possible to generate approximately the same behavior by roughening the surface of scaled models.

3.2 Froude number

The Froude number is the ratio of inertial to gravity forces:

$$F_r = \frac{U^2}{Lg} \tag{6}$$

where g is the acceleration of gravity. Take for example a model scaled at $L_m/L_p = 1/400$. Froude scaling requires $U_m/U_p = 1/\sqrt{400} = 1/20$. The Reynolds number for the model is:

$$R_{em} = \frac{U_m L_m}{\nu_m} = \frac{(U_p/20)(L_p/400)}{\nu_p} = \frac{R_{ep}}{8000} \tag{7}$$

Thus it is seen that Froude scaling and Reynolds scaling are incompatible if the laboratory uses the same gravity constant and the same fluid (air) as the atmosphere.

3.3 Rossby number

The atmospheric boundary layer depth δ is proportional to the Rossby number R_o:

$$\delta = cR_o \tag{8}$$

where $c \approx \frac{0.25\,h}{2.5\,\ln(h/z_0)}$, $R_o = \frac{U(h)}{hf_c}$, $U(h)$ is the mean wind speed at a height h, z_0 is the roughness length and f_c is the Coriolis parameter. As the wind speed increases, so does the boundary layer depth. For wind speeds of interest to the structural engineer, the boundary layer is several kilometers deep. Most buildings are situated entirely in the lower one-tenth of the boundary layer, where the longitudinal velocity spectrum $S(z,n)$ in the inertial subrange can be expressed as:

$$\frac{nS(z,n)}{u_*^2} = 0.26f^{-2/3} \tag{9}$$

where n is the frequency of turbulent eddies, z the height, u_* the shear velocity of the flow, and $f = nz/U(z)$ the Monin (or similitude) coordinate.
In a long wind tunnel, a boundary layer of the order of 1 m develops over a rough floor, and the above equation only applies to the bottom 0.1 m. A 200 m tall building modeled to the scale of 1/400 would only see its bottom 40 m exposed to the spectrum described by the equation above.

3.4 Strouhal number

Vortex shedding is a regular phenomenon characterized by the Strouhal number:

$$S = \frac{N_s D}{U} \tag{10}$$

where N_s is the frequency of full cycles of vortex shedding, and D is a characteristic dimension of the body normal to the mean flow velocity U.

3.5 Scaling effects of turbulent flow around bluff bodies

Davenport et al. (1977) compare wind pressure measurements for three models at scales of 1/500, 1/250 and 1/100. The prototype is a low-rise building 30.5 m x 24.4 m x 5.9 m with a gable roof of slope 4.7 degrees. The ratios between corresponding measurements, in terms of peak, mean and root-mean-square, differ significantly from unity in some instances and do not follow any discernable pattern. For example, the ratio of the peak pressure of the 1/100 model to the 1/500 model measured at a roof corner is 1.34, and the ratio of the peak pressure of the 1/250 model to the 1/500 model at the center of the long side is 0.63.
One reason for these discrepancies is the difficulty in determining the characteristic length (integral scale) of turbulence at full scale (measurements can vary by a factor of five, even ten), and in scaling this length to that of the model. In the investigation mentioned above (Davenport et al., 1977), the integral scale of turbulence in the wind tunnel was estimated to be 1/500 of its nominal value in atmospheric flow. Another possible reason for the discrepancies is that the change in actual elevation of the corresponding measurement points of the various models causes changes in turbulence intensities.
In the testing of rigid trussed frameworks, the effect of high-frequency turbulence components may be assumed to be insignificant, since no flow reattachment can be expected to occur on typical truss members, even in the presence of high-frequency turbulence fluctuations. Low-frequency turbulence components may be assumed approximately to be perfectly coherent over the width of vertical frameworks or the depth of horizontal frameworks. The tacit acceptance of these assumptions explains why, even after the advent of boundary-layer wind tunnels, aerodynamic testing of trussed frameworks has been performed in smooth flow (Whitbread, 1980), i.e., flow with constant velocity and no significant turbulence fluctuations. For bluff bodies with small dimensions (e.g., residential homes, or tributary areas of individual portal frames in industrial buildings) this extreme simplification of the testing may not be appropriate because high-frequency flow fluctuations can strongly influence test results.

3.6 Blockage

A structural model placed in the air flow provides partial blockage to the flow and causes it to accelerate. Correction factors have been worked out for a limited number of

measurements and geometries. In general, for a blockage ratio of 2 %, the corrections are of the order of 5 % and proportional to the blocking ratio (Melbourne, 1982).

4. Wind climatology

The wind tunnel method typically uses directional wind speed data whenever such data are available. In some cases, directional wind speed observations are available for each of a number of equally spaced directions, but for some of the directions the number of observations is too small to allow meaningful estimates of the respective extremes. In those cases a conservative assumption is required to construct appropriate sets of data for the directions with insufficient observations by using data available for other directions. For an example of such an assumption, see Grigoriu (2009).

5. Statistics of peak effects

In this section, we are concerned with the estimation of peak effects of a one-dimensional stochastic process *induced by a given wind speed*. Such peak effects may pertain to accelerations at the top of the structure, inter-story drifts, internal forces, and sums of demand-to-capacity ratios used in interaction equations (e.g., the sum of (a) the ratio of the axial force divided by axial force capacity and (b) the ratio of the bending moment divided by the moment capacity). The estimation can take advantage of the fact that most wind effects of interest are sums of many comparable randomly distributed contributions, rendering those effects Gaussian. For this case simple, well-known techniques are available for estimating the mean values of the peaks. If the wind effects of interest are not Gaussian (this may be the case, e.g., for wind effects in low-rise buildings), techniques for estimating statistics of their peaks are also available, see e.g., Sadek & Simiu (2002) or www.nist.gov/wind, item III.

Two cases are of interest. The first case involves stochastic processes specified by their time histories (i.e., in the *time domain*). For example, such a stochastic process may consist of the internal force induced in a member by the sway responses in directions x and y. The peak internal force of interest is then simply the peak of the sum of the internal forces associated with the two sway responses. A similar simple summation yields the requisite peaks of processes consisting of sums of any number of stochastic processes.

The second case involves stochastic processes specified by their spectral densities (i.e., in the *frequency domain*). Such specification was -- and still is -- routinely used in wind engineering dynamic analyses on account of the difficulty, up to the 1980s, of dealing computationally with the solution of dynamic problems in the time domain. Obtaining the peak of a sum of several stochastic processes is no longer possible by summing up those processes, because in the frequency domain as used in typical wind engineering applications the phase information inherent in the respective time histories is lost. Therefore the estimation of peaks requires the use of sums of weighted component processes (e.g., axial forces and bending moments), with weights specified by engineering judgment. As many as dozens of such weighted combinations are prescribed to structural designers by wind engineering laboratories, in an attempt to make sure that relevant peaks are not missed. This is time-consuming from the point of view of the designer, as well as less accurate than the much simpler, more transparent, and more effective time-domain approach. Additional

drawbacks are the difficulty of accounting for wind directionality effects in a transparent and physically meaningful way (see Simiu, 2011).

If wind directionality is not taken into account explicitly (i.e., other than through the use of a blanket reduction factor, as is done in ASCE 7, 2006), then the *mean recurrence intervals (*MRI) of a peak wind effect is simply assumed to be the MRI of the wind speed inducing it. Estimation methods for extreme wind speeds with specified MRIs, regardless of their direction, are discussed, e.g., in Simiu (2011). If directionality is explicitly accounted for in the calculations, the peak wind effects with specified MRI are obtained as follows. Assume the directional wind speed data consist of m sets (e.g., m storm events) of n directional wind speeds each (corresponding to, e.g., n = 16, or n = 36 directions). For each of the m sets, calculate the peak response induced by each of the n directional wind speeds, and retain only the largest of these n responses. This yields a set of m largest peak responses. The m peak responses are then rank-ordered. If the rate of arrival of the events associated with the m sets is r/year, then the estimated MRI of *a)* the largest of the m peak responses is $(m+1)/r$ years; *b)* the p largest is $(m+1)/(pr)$ years. This estimate is non-parametric.

In hurricane-prone regions, directional wind speeds in any specified number of storms are typically obtained by Monte Carlo simulation using physical and probabilistic storm models (e.g., Batts et al. 1980; Vickery & Twisdale, 1995; Vickery et al., 2009). If the size of an existing database of hurricane wind speeds needs to be augmented (as may be necessary for the only public hurricane wind speeds database that covers the entire Gulf and Atlantic coasts of the United States, and contains 999 simulated storms for each station, listed on www.nist.gov/wind), this augmentation can be achieved by using software developed by Grigoriu (2009). For straight line (synoptic) and thunderstorm wind speed data, large sets of simulated data and associated errors in their estimation can also be obtained using the methods developed by Grigoriu (2009).

6. Structural reliability

The purpose of structural reliability is to develop design criteria assuring that the probability of inadequate strength and serviceability performance is acceptably small. The traditional approach to design, Allowable Stress Design (ASD), is no longer in use but is relevant for the understanding of the Strength Design (SD) approach, currently specified in ASCE 7-10 (2010). In its simplest form allowable stress design (ASD) achieves this purpose by requiring, for each member, that the stress induced by the sum of the basic design loads not exceed the allowable stress, typically defined as the nominal yield stress divided by a safety factor. The basic design loads and the safety factor are based on experience gained in past practice. For example, the basic design wind load is typically specified as the wind load with a nominal 50-yr MRI. If for A36 steel (nominal yield stress 36 ksi or 249 MPa), the allowable stress for non-compact sections is 22 ksi (152 MPa), then the safety factor is 36/22 = 1.64. ASD's probabilistic content consists of the specification of the MRI of the basic design wind load and of the definition of the nominal yield stress as a specified percentage point of its probability distribution.

Strength design (SD) requires in its simplest form that, for each member, the stress induced by the sum of the basic design loads, each multiplied by a load factor that depends upon the type of load, not exceed the yield stress multiplied by a resistance factor smaller than unity. Consider, for example, the case where wind is the only significant load acting on an A36 steel member with non-compact cross section. The wind load factor specified by ASCE 7 is 1.6, and if the resistance factor is 0.9, SD design requires that the basic wind load induce in the member

a stress of at most 36 x 0.9/1.6 = 20.25 ksi (139.6 MPa), rather than 22 ksi (151.7 MPa) as required by ASD. In this example SD is more conservative than ASD. This is in part due to the specification by ASCE 7 of a wind load factor equal to 1.6, even though the original intent of the standard was to specify a wind load factor equal to 1.5 (see ASCE 7-05 Commentary Sect. C6.5.4). Had the latter value of the wind load factor been specified, the SD would have required that a stress of 36 x 0.9/1.5 = 21.6 ksi (148.9 MPa) not be exceeded under the basic wind load, i.e., a stress substantially equal to the 22 ksi (151.7 MPa) stress required in ASD.

To see why the difference between ASD and SD can be significant, consider the case of a member subjected to dead load and wind load. The uncertainties inherent in the wind load being larger than those inherent in the dead load, it is appropriate that the safety margin with respect to the loading (i.e., the load factor) be larger for the wind than for the dead load. This is reflected in ASCE 7-05, Section 2.3, which specifies for the dead load factor a value smaller than 1.5 or 1.6. In contrast, for ASD, the dead loads and the wind loads are both affected by a factor equal to unity.(Reference to ASCE 7-05 is warranted in this section for two reasons. First, ASCE 7-05 is still being widely used. Secondly, ASCE 7-10 provisions on SD are based on the rationale inherent in the ASCE 7-05 SD provisions, in terms of substance if not of format.)

The probabilistic content is richer for SD than for ASD inasmuch as the various load factors account for estimated probability distributions of the total uncertainties, which include uncertainties in the wind speed and in the wind effect.

Uncertainties in the wind speed. The design value may well be smaller than the actual value affecting the structure during its life, since the wind speed is a random variable characterized by a probability distribution. In addition, that distribution may be affected by modeling errors (e.g., it could be a Type I Extreme Value distribution, a Type III Extreme Value distribution, a penultimate distribution, a mixed distribution of synoptic and thunderstorm wind speeds, and so forth). Finally, the assumed distribution and/or its estimated parameters are affected by sampling errors due the relatively small size of the observed data sample, by observation errors, and, in the case of hurricanes, by physical modeling errors of climatological parameters used in Monte Carlo simulations of the wind speeds, e.g., the radius of maximum wind speeds, the pressure defect at the center of the eye of the storm, and so forth.

Uncertainties in the wind effect. Uncertainties in aerodynamic pressure or force coefficients are due to measurement errors and/or to errors in the simulation of the flows that induce those pressures. For rigid structures the responses induced by aerodynamic pressures are proportional to the square of the wind speeds. For flexible structures the responses are proportional to the wind speeds raised to powers larger than two. The contribution to errors in the estimation of the wind response of estimation errors in the wind speeds is therefore greater for flexible than for rigid structures. Another contribution is due to the effect of errors in the estimation of natural frequencies of vibration, modal shapes, and damping ratios.

The wind load factor specified in ASCE 7 was estimated by accounting in an approximate manner for the uncertainties in the wind speeds and wind effects affecting *rigid* structures in *non-hurricane* wind climates. The resulting estimate was approximately 1.6 (or 1.5, depending upon the version of ASCE 7 being considered). To simplify codification, the ASCE 7 conventional methods assume, for non-hurricane regions, that the MRI of the wind effect that induces yield stresses is the MRI of the wind speed defined by the basic wind speed times the square root of the wind load factor. Calculations based on simplifying assumptions described

in the ASCE 7 Commentary show that this MRI is nominally about 500 years for a wind load factor of 1.5 and about 720 years for a wind load factor of 1.6. SD calculations can then be based on wind effects on rigid structures induced by 500-yr (or 720-yr) wind speeds. Note that in ASCE 7 the effective value of the wind load factor for hurricane-prone regions is larger than 1.6. This is achieved by maintaining the nominal 1.6 value of the load factor, while increasing the MRI of the basic speeds (see ASCE 7-05 Commentary C6.5.4).

It is assumed in current wind engineering practice that this approximate approach is applicable not only to rigid structures, but to flexible structures as well. To see why this assumption can be unwarranted, we consider the following first-order second-moment calculation (Ellingwood et al., 1980) for typical rigid structures. The approximate value of wind effect of interest may be written in the form

$$W_r = cC_pGE_zv^2 \tag{11}$$

where c, C_p, G, E_z, and v denote a proportionality factor, pressure coefficient, peak (gust response) factor, terrain exposure factor, and wind speed, respectively, and the subscript r denotes "rigid". The approximate value of the coefficient of variation of W_r is then

$$V_{Wr} = \left(V_{Cp}^2 + V_G^2 + V_{Ez}^2 + 2V_v^2\right)^{1/2} \tag{12}$$

where V followed by a subscript denotes coefficient of variation. For tall, flexible buildings the wind effect is proportional to the velocity raised to a power larger than 2; assuming for simplicity

$$W_f = cC_pGE_zv^3 \tag{13}$$

where the index f denotes "flexible", the approximate value of the coefficient of variation of W_f is

$$V_{Wf} = \left(V_{Cp}^2 + V_G^2 + V_{Ez}^2 + 3V_v^2\right)^{1/2} \tag{14}$$

Assume V_{Cp} = 0.10 for both rigid and tall buildings; V_G = 0.10 for rigid buildings and V_G = 0.14 for tall buildings, since the uncertainty in the dynamic parameters adds to the uncertainty in the ratio between peak and mean response; V_{Ez} = 0.12 for rigid buildings and V_{Ez} = 0.10 for tall buildings, since the uncertainties in the features of the wind flow are likely to be smaller if ad-hoc testing is performed; and V_v = 0.18, including aleatory variability, observation errors, and sampling errors.

Given these assumptions we have: V_{Wr} = 0.315 and V_{Wf} = 0.37. The difference between the respective estimates is about 17.4 %. Since the wind load factor is, very approximately, an increasing linear function of the coefficient of variation of the wind effect, it follows that the load factor should indeed be larger for tall buildings than for rigid buildings. Estimates based on coefficients of variation are crude, and far more accurate estimates can be obtained by numerical simulations based on a detailed model of the dynamic response that accounts for the effect of wind directionality[1] (Gabbai et al., 2008; Simiu et al, 2008). Note that the

[1] Estimates of wind load factors based on the along-wind and across-wind response due to wind normal to a building face do not reflect correctly the margins of safety applicable to a building subjected, as it is in reality, to winds blowing from any direction. Note also that the notion of along-wind and across-

value of the load factor depends upon the characteristics of the building and upon the structural member being considered. Current calculations assume that one wind load factor is appropriate for all buildings and all their structural members. A far more differentiated approach is needed, however. The development of highly efficient software now allows the use of such an approach.

As was noted earlier, the ASD and SD approaches are applied to individual members and do not consider the reliability of the building as a whole. A step forward was achieved by applying methods that allow the estimation of the MRI of incipient structural collapse, a limit state beyond which it may be assumed that the structure's strength reserves have largely been exhausted for practical purposes. Modern finite element methods and sets of directional wind tunnel pressure data obtained simultaneously at large numbers of taps have been applied recently to obtain such estimates for rigid buildings (Jang at al., 2002; Duthinh et al., 2008).

7. Validation of wind tunnel testing

Dalgliesh (1975), Dalgliesh (1982) and Dalgliesh et al. (1983) compared measurements of wind pressure at full-scale and on a 1/200 aeroelastic wind tunnel model of the Commerce Court West Tower in Toronto. The 57-story steel frame (36.5 m x 69.7 m x 239 m) was the tallest building in Canada when it was completed in 1973. Agreement between model and full-scale measurements of mean pressures is satisfactory, but there are significant differences between the two sets of measurements of fluctuating pressures caused by vortex shedding. The model, with mass lumped at seven levels, agreed reasonably well with full scale measurements at the low and high end of the acceleration spectrum, but underestimated the response in the intermediate frequency range.

Richardson & Surry (1991) compare full scale and wind tunnel measurements for low-rise, gable roof buildings and conclude that flow separation on the windward roof is not modeled correctly, causing significant errors in roof pressures derived from the model. Tieleman (1992) confirms that wind tunnel measurements are inadequate for roof corners, but acceptable for wall pressures. Tieleman (1993) suggests the use of small spires upstream of the model to improve the turbulence characteristics of the flow and the fidelity of wind pressures at roof corners.

Comparison of measurements by six wind tunnels showed significant discrepancies for low-rise buildings (Fritz et al. 2008). For example, the largest estimate of the bending moment at the knee of portal frames with 6 m and 9.75 m eave height is more than twice the smallest. Comparison between ASCE 7 Standard and calculations of wind loads based on wind tunnel tests at the University of Western Ontario shows differences exceeding in some cases 50 %, attributable in part to errors inherent in wind tunnel testing of low-rise buildings (Surry et al. 2003, Ho et al., 2005, St. Pierre et al., 2005, Coffman et al., 2010).

For taller buildings, results between various laboratories appear more consistent, although discrepancies exceeding 40 % have been reported (NCSTAR 1-2, Appendix D, 2004).

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wind response, assumed in the past to apply only to the case of wind normal to a building face, in fact also applies to the case of winds skewed with respect to a building face.

8. Wind tunnel testing of low-rise buildings

Low-rise buildings typically have natural frequencies much greater than their response frequencies to fluctuating wind forces, and thus their aeroelastic behavior can be neglected, i.e., they can be considered rigid. Current procedures in the US for the wind design of rigid buildings (in practice buildings with fundamental natural frequencies greater than 1 Hz - ASCE 7 Section 26.9) rely on ASCE 7 Standards that are based on wind tunnel measurements on a fairly modest number of geometries (about ten) that do not encompass all the building configurations covered by the provisions. In addition, the reduction of aerodynamic data to a few tables and curves amenable to hand calculations can lead to significant errors (Ho et al., 2005, St. Pierre et al., 2005, Coffman et al., 2010). Here we present an alternative method, called the Database-Assisted Design (DAD) method, allowed by ASCE 7-10 Standards (2010), that is more accurate and relies on modern computer technology.

8.1 Database-assisted Design (DAD) method for low-rise buildings

DAD takes the wind and structural engineers from wind tunnel pressure data to the sizing of structural members to resist wind forces, through the following steps:

Step 1. Interpolation of pressure data to the building being designed: Wind tunnel models are equipped with hundreds of pressure taps installed in a grid pattern on the walls and roof. These taps are scanned nearly simultaneously at high frequencies to provide time histories of wind pressure. The measured pressures are multiplied by tributary areas centered at the taps and defined by lines that bisect the grid defined by the taps, and in this way, forces and moments on the main wind resisting frames can be calculated. The peak responses corresponding to a unit wind speed for a particular wind direction, called Directional Influence Coefficients (DIF), have proven to be convenient and useful for structural design. Since the building is rigid, its linear response is not affected by dynamic effects, and the peak wind response is the product of the DIF and the square of the wind speed.

Most of the time, the building to be designed does not match the geometry of any model in the database, and thus interpolation is necessary. This works best when there are two models in the database of similar geometry but slightly different dimensions that bracket the building. First the coordinates of the pressure taps on the models are scaled to the dimensions of the building. DIFs are calculated from one model, or from several by interpolation, with more weight given to the models that more closely match the dimensions of the building. Main (2005) shows the interpolation agrees well with data obtained directly for the building of interest, although interpolation between roofs of different slopes needs further research.

Step 2. Influence coefficients required to calculate internal forces can be obtained from standard structural analysis software, based on a preliminary building design. Likewise statistics of peak values can be obtained from standard software, provided for example in the DAD software package www.nist.gov/wind, which also includes an interpolation scheme and methods to calculate demand-to-capacity indices (DCI) for structural members, inter-story drifts and peak accelerations with their respective mean recurrence intervals (MRI).

DAD accounts for wind directionality by explicitly using wind tunnel measurements for different wind directions (currently up to 36) and calculating statistics on the peaks of these effects. This is a clear improvement on the current practice of applying a

directionality factor of 0.85 to wind effects calculated with no consideration of wind directionality. DAD is an expansion of the method of simultaneous pressure integration leading to wind loading pioneered by the University of Western Ontario. DAD provides wind loading as well as wind effects on buildings, and checks building performance for strength and serviceability.

9. Wind tunnel testing of tall buildings

9.1 The High-Frequency Force Balance (HFFB) method

In this method, which is relatively inexpensive and fast, the building model is rigid and a high frequency balance at its base measures shear forces, bending and torsional moments. The method works best for buildings with linear mode shapes along the principal axes, although correction factors that assume a wind pressure distribution have been derived for other buildings. The bending moment measured at the base of a building of height H due to wind forces per unit height in the x direction $w_x(z,t)$ is:

$$M_x(t) = \int_0^H w_x(z,t) z dz \tag{15}$$

where z is the vertical coordinate. If the first fundamental mode shape is linear, $\varphi_{x1} = z/H$, then the generalized force

$$Q_{x1}(t) = \int_0^H w_x(z,t) \varphi_{x1} dz \tag{16}$$

is proportional to the measured moment. The same conclusion obtains for moment and generalized force in the y direction, but not for the generalized aerodynamic torsional moment (since the fundamental torsional mode shape is not uniform, $\varphi_{T1} \neq 1$).

9.2 Aeroelastic method

In this method, the model reproduces the mass distribution and stiffness of the building, and thus renders faithfully its modal shapes and dynamics. This kind of model is rarely used because of the cost and time involved in its fabrication. A more affordable and simpler type of model is the so-called "stick" model that is similar to the HFFB model, but in addition approximates the aeroelastic response of the building by accounting for its flexibility and damping characteristics. Aeroelastic modeling is deemed unnecessary if the across-wind response is insignificant (roughly speaking and for a rectangular building, when the height is less than seven times the width).

Fortunately, aeroelastic effects on tall buildings tend to be small for the following reasons:

1. The frequency range of vortex-induced forces is rather broad, and only a small fraction produces lock-in oscillations.
2. Three-dimensional flow in buildings with small or no taper impedes the formation of vortices near the building top, where they have the greatest effect.
3. Non-uniform wind profiles reduce the coherence of the vorticity in the vertical direction. This effect can be enhanced by a strong taper in the building, or varying the cross section to provide discontinuities in the vertical dimension. Introducing slits, chamfering corners can also impede vortex formation.

9.3 Database-assisted design of tall buildings

DAD can be used for tall as well as low-rise buildings. Unlike current design methods that rely on spatially averaged wind pressures, the Database-Assisted Design (DAD) is a time-domain method that uses directional pressure time histories recorded by a sufficiently large number of pressure taps in wind tunnel tests. In addition, the method accounts for directional wind data recorded at weather stations and supplemented by Monte Carlo simulations. The wind engineer relates the pressures on the prototype building to the wind tunnel pressures by calculating the ratio of the wind speed measured at the weather station to the reference wind speed at the top of the building model, accounting for the building's exposure. With this information and the use of standard software to calculate influence coefficients, the structural engineer can obtain the demand-to-capacity index (DCI) of various members, as well as other quantities of interest, such as the inter-story drift and the top floor acceleration, corresponding to a given mean return interval (MRI). The DCI calculations are based on current American building practice embodied in ASCE 7, AISC (American Institute of Steel Construction) and ACI (American Concrete Institute international).

The DAD method takes advantage of modern computer technology in providing designers direct access to wind tunnel as well as weather station and simulation data. Increases in computer memory and speed make possible the simultaneous scanning of hundreds of pressure taps via calibrated plastic tubes. Furthermore, DAD accounts for wind directionality and any building modal shape in an explicit and transparent manner. As a time domain approach, it also allows superposition of wind effects. The main disadvantage of the method is its cost and duration, since the wind tunnel tests require multiple pressure taps as opposed to the simpler HFFB. In some instances, architectural details may make it impractical to install pressure taps.

10. Wind effects estimation

To assure transparency and accountability in the estimation of wind effects, *wind engineering laboratories should provide*:

1. Records of the requisite wind climatological data. These consist of relevant measured and/or synthetic directional wind speeds obtained from legitimate sources, e.g., meteorological stations or reliable hurricane wind speed databases, augmented if necessary by methods such as those developed by Grigoriu (2009).
2. Information, typically based on wind tunnel tests, on the ratio between directional wind speeds at the reference site (e.g., 3-s peak gusts or 1-min speeds at 10 m above ground over terrain with open exposure) and the corresponding nominal mean hourly wind speeds at the top of the building being considered. This information is required because wind tunnel data are referenced with respect to those mean speeds. In addition, the wind tunnel laboratory needs to provide estimates of veering angles. Veering in atmospheric flows, which can be aerodynamically significant for tall buildings, is an effect of the Coriolis acceleration, which is not reproduced in the wind tunnel.
3. Wind tunnel measurement records of primary wind effects on structures.

- For *rigid structures* the measurement records of interest typically consist of time histories of simultaneously measured pressures at large numbers of taps on the structure's exterior surface.
- For *dynamically active structure*s with no significant aeroelastic effects under winds that may be expected to occur during their lifetime, the measurement records consist, for

each of the wind directions for which wind speed data are considered, either of time histories of simultaneously measured pressures at large numbers of taps on the structure's exterior surface, or of time histories of strains measured on a high-frequency force balance at the base of rigid building models.

- For *structures that exhibit aeroelastic effects,* we refer the reader to Zhou & Kareem (2003) and Diana et al. (2009).

Once the structural designer has these data, he/she can use them as input to available procedures that produce the requisite design information. This information is far more comprehensive and differentiated than what wind engineering laboratories currently provide and includes, for any given set of the building's structural member sizes, demand-to-capacity ratios (which typically account for axial force and bending moment in interaction formulas), and inter-story drift, corresponding to the respective specified MRIs. The calculations involve structural analyses to obtain the requisite sets of influence coefficients. In addition, for flexible structures, analyses are needed to estimate natural frequencies of vibration and the corresponding modal shapes. For details, see Main (2006), Simiu & Miyata (2006), Simiu et al. (2008), and Spence (2009). Initial member sizes are based on preliminary calculations. Final member sizes are obtained by repeating the calculations if necessary. Research is ongoing on an iterative optimization process for obtaining member sizes consistent with strength and serviceability constraints (Spence & Gioffrè, 2008).

The innovation that makes possible the clear separation between tasks incumbent on the wind engineering laboratory on the one hand and the structural designer on the other is the estimation of wind effects by using time domain instead of frequency domain methods. Frequency domain methods, in which differential equations are converted into algebraic equations at the cost of losing phase information, were introduced by Liepmann (1952) for aeronautical engineering applications and were adapted to structural engineering applications by Davenport (1961). As was noted earlier, their use was justified by the fact that, for computational reasons, modal equations of motion could not in practice be solved in the time domain. This constraint no longer exists. Nevertheless the use of frequency domain methods persists in spite of their significant drawbacks.

11. Conclusion

A fundamental difficulty in achieving reproducible wind tunnel measurements of wind effects, especially on low-rise buildings, is the simulation of atmospheric flows. In particular, the correct simulation of suctions in separation bubbles requires the reproduction of high-frequency atmospheric turbulence, which is prevented in large part by the action of viscous dissipation at model scales typically used in commercial wind tunnels. Low-frequency fluctuating components, which are the major contributors to the flow's turbulence intensities and integral turbulence scales, may be expected to affect aerodynamic response insignificantly provided (i) that the characteristic dimensions of the structure of interest are sufficiently small, as is the case for, e.g., residential homes, and (ii) that the mean speeds in the simulated flow are commensurate with the peak speeds in the atmospheric flow. For structures with dimensions for which testing proves that this proposition holds, valid aerodynamic simulations can be obtained in flows that simulate only mean speeds and high-frequency flow fluctuations, provided that the mean velocity profile is correctly modeled (Huang et al., 2009). Using rapid scanning equipment, fast computers with vast storage,

modern wind tunnels can provide simultaneous measurements of numerous pressure taps, thus allowing the development of time domain methods of calculating the response of buildings to wind.

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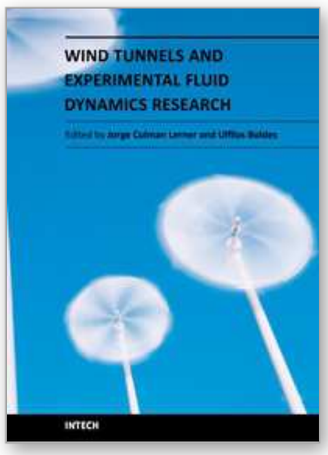
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The book â€œWind Tunnels and Experimental Fluid Dynamics Researchâ€ is comprised of 33 chapters divided in five sections. The first 12 chapters discuss wind tunnel facilities and experiments in incompressible flow, while the next seven chapters deal with building dynamics, flow control and fluid mechanics. Third section of the book is dedicated to chapters discussing aerodynamic field measurements and real full scale analysis (chapters 20-22). Chapters in the last two sections deal with turbulent structure analysis (chapters 23-25) and wind tunnels in compressible flow (chapters 26-33). Contributions from a large number of international experts make this publication a highly valuable resource in wind tunnels and fluid dynamics field of research.

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