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Advances in the design of high-rise structures by the wind tunnel procedure: Conceptual framework

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Abstract. This paper surveys and complements contributions by the National Institute of Standards and Technology to techniques ensuring that the wind tunnel procedure for the design of high-rise structures is based on sound methods and allows unambiguous inter-laboratory comparisons. Developments that enabled substantial advances in these techniques include: Instrumentation for simultaneously measuring pressures at multiple taps; time-domain analysis methods for estimating directional dynamic effects; creation of large simulated extreme directional wind speed data sets; non-parametric methods for estimating mean recurrence intervals (MRIs) of Demand-to-Capacity Indexes (DCIs); and member sizing based on peak DCIs with specified MRIs. To implement these advances changes are needed in the traditional division of tasks between wind and structural engineers. Wind engineers should provide large sets of directional wind speeds, pressure coefficient time series, and estimates of uncertainties in wind speeds and pressure coefficients. Structural engineers should perform the dynamic analyses, estimates of MRIs of wind effects, sensitivity studies, and iterative sizing of structural members. The procedure is transparent, eliminates guesswork inherent in frequency domain methods and due to the lack of pressure measurements, and enables structural engineers to be in full control of the structural design for wind.

Keywords: aerodynamics; design; high-rise buildings; micrometeorology; structural dynamics; structural engineering; wind climate; wind engineering; wind tunnel procedure; wind tunnel testing

1. Introduction

The *wind tunnel procedure*, specified by the ASCE 7-10 Standard (ASCE 7-10, 2010), is a multidisciplinary set of techniques for the estimation of wind effects on buildings and other structures by using, among other types of information, aerodynamic or aeroelastic information obtained by *wind tunnel testing*.

Some confusion between the terms “wind tunnel procedure” and “wind tunnel testing” persists in the literature. The ASCE 49-12 Standard *Wind Tunnel Testing of Buildings and Other Structures* (ASCE 49-12, 2012) addresses, albeit incompletely, wind tunnel procedure techniques other than aerodynamic model testing techniques, that is, other than wind tunnel testing properly so called. A recent guide on techniques associated with the use of the wind tunnel for the design of high-rise structures (Irwin *et al.* 2013) also refers to them collectively as “wind tunnel testing,” even though they include wind climate analysis unrelated to wind tunnel testing.

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That guide notes that wind tunnel procedure techniques "are not widely understood by the designers using the results," and that it is therefore necessary to improve the understanding of the wind tunnel procedure in the design community. The purpose of this paper is to complement that guide by presenting an innovative conceptual framework for the structural design of high-rise structures to wind. The framework fully exploits modern experimental and numerical capabilities created by recent major technological developments, with a view to (i) achieving a design process that can be effectively scrutinized by stakeholders, including structural engineers, owners, insurers, users, and building officials, and (ii) estimating wind effects more accurately than is the case in current practice, thus achieving structural designs superior from the point of view of risk consistency, safety, cost and embodied energy consumption. This paper surveys and complements contributions by the National Institute of Standards and Technology (NIST) to knowledge and techniques aimed at ensuring that the wind tunnel procedure is based on sound methods and yields repeatable and reproducible results, independent of the entities obtaining them. Most of papers and reports referenced herein were developed in response to a NIST recommendation contained in the report on the World Trade Center (WTC) investigation www.nist.gov/el/disasterstudies/wtc/upload/WTCRecommendationsStatusTable.pdf". The recommendation was prompted by the finding that estimates of the WTC towers' response by two independent wind engineering reports differed by over 40% (Skidmore Owings and Merrill 2004, Griffis 2006).

Helping designers and building officials understand the wind tunnel procedure is tantamount to improving its transparency. Such was the perceived lack of transparency of the wind engineering reports on the WTC towers that Skidmore Owings and Merrill (2004), a prominent designer of high-rise buildings, stated: "Because the wind tunnel reports only summarize the wind tunnel test data and wind engineering calculations, precise evaluations are not possible with the provided information". It is desirable that this state of affairs change, and that it becomes possible to readily and clearly evaluate laboratory reports, as well as identifying the causes of possible discrepancies between their respective results.

Skidmore Owings and Merrill (2004) ascribed differences in the analyses and results for the WTC towers to the fact that "wind engineering is an emerging technology". More than one decade later wind engineering has considerably evolved and has greatly benefited from recent, vastly expanded experimental and numerical capabilities. Irwin *et al.* (2003) contains no reference to research based on those capabilities published during the last decade; for this reason progress due to that research is not covered therein.

In addition to improved transparency, substantive improvements in the effectiveness of the design process have become possible. For example, in current practice wind engineers produce estimates of the structural response to wind based on dynamic properties of the structure provided to them by the structural engineers. However, the properties of the structure designed on the basis of those estimates typically differ from the properties inherent in the preliminary design provided by the structural engineer, meaning that the respective dynamic response estimates would differ as well. It is therefore necessary for the design process to be iterative. The design process must also accommodate studies on the effects of various uncertainties, including, as noted by Irwin *et al.* (2013), uncertainties in the dynamic properties that depend upon assumptions on reinforced concrete cracking or steel connection slippage.

In this connection it is noted, first, that given the current distribution of tasks between wind and structural engineers, the iterative determination of wind effects would require "back and forth" interactions with wind engineers. Such interactions can be unwieldy, indeed prohibitive.

The fact that for this reason the requisite iterations are often not performed is detrimental to the

efficiency of the structural design. The more effective design process presented in this paper includes the ready capability to perform the requisite iterations.

Second, wind effects are currently typically combined "by eye". While, in the absence of measurements, decisions based on engineering judgment are necessary, decisions based on measurements are clearly preferable. A design process of superior effectiveness can use measurements to reduce or eliminate significant estimation errors due to inadequate accounting for wind effect combinations. In the procedure presented in this paper this is achieved by the use of measured time histories of pressures in conjunction with appropriate time-domain computation techniques.

Third, an effective design process must account, in a transparent and realistic manner, for the dependence upon direction of the wind speeds, the aerodynamic response, and the dynamic response. For typical buildings designed in accordance with routine code provisions, the mean recurrence intervals (MRIs) of the wind effects are nominally the same as those of the non-directional wind speeds producing them. However, this is not the case for high-rise buildings, for which wind effects with specified MRIs must be estimated by accounting for directionality in specific detail. The estimation of wind effects with specified MRIs using methods included in the guide by Irwin *et al*. (2013) but viewed by structural engineers (e.g., Skidmore Owings and Merrill 2004) as opaque and insufficiently validated or established was deemed to contribute significantly to the large discrepancies between the estimated responses of the WTC towers.

The wind tunnel procedure presented in this paper eliminates or reduces these weaknesses in current practices. The paper is organized as follows. In the next section it discusses briefly the widely used High Frequency Force Balance (HFFB) technique. The need to update the "Wind Loading Chain" (Davenport 1982) so that it conforms to modern wind and structural engineering needs is then discussed. Subsequent sections, limited to the case of structures with no significant aeroelastic effects, are devoted to the tasks of the wind engineer and those of the structural engineers. The paper ends with a set of conclusions.

2. The high-frequency force balance (HFFB) technique

The HFFB technique is described, e.g., in Simiu (2011). It is relatively inexpensive and fast, and can be used to good effect (i) for comparing the building performance under various aerodynamic scenarios (e.g., buildings with sharp versus chamfered corners), or (ii) for measuring the aerodynamic response of buildings with features that prevent the effective use of pressure taps. HFFB was nevertheless used almost universally in the past. In the absence of more effective methods such use was entirely legitimate, as was, for example, before the development of computerized structural analysis methods, the moment distribution method. This is true even though HFFB introduces significant errors in the information needed for the sizing of structural members. These errors are due to the following facts: (i) because measurements are not available, the distribution of the wind loading with height is not known and must be estimated – guessed at, -- which is especially difficult for loading due to wind speeds skewed with respect to the building's principal axes, as well as for cross-wind loading; (ii) guesswork must also be used for the estimation of combinations of wind-induced forces in structural members; (iii) HFFB cannot account for effects of higher modes of vibration; (iv) the fundamental modal shape assumed for sway motions is assumed to be a straight line that starts from zero at the base, and any attempted correction to account for the curvature of that shape is dependent upon guessed-at wind loading

distributions with height.

The first point is easy to demonstrate. Determining the magnitude of the wind forces acting at various levels on the building is clearly not possible if measurements are available only for *two* quantities -- the base moment and shear, -- even in the simple case in which the wind loading were to consist of only *three* concentrated wind forces; a similar argument holds *a fortiori* for the case of a continuous distribution of the wind loads. The second point was clearly demonstrated by significant differences between guessed-at load combinations used in the two wind engineering reports on the WTC towers mentioned earlier. To appreciate why correct wind effect combinations are important, recall that the design wind effects that determine the sizing of structural members consist of *demand-to-capacity indexes* (DCIs), that is, of the left-hand sides of member *interaction equations*. The DCIs are weighted sums (combinations) of total internal forces in structural members (e.g., total axial forces *and* total bending moments). Each total internal force is in turn a sum (combination) of several contributing internal forces. As an illustration, for a member whose sizing is determined by an axial force and a bending moment, the following contributing wind-induced internal forces need to be combined to yield total internal forces: an axial force and a bending moment due to wind loading acting along one of the principal axes of the structure, an axial force and a bending moment due to wind loading acting along the second principal axis, and bending moments due to the building's aerodynamic torsional moment. Typically, each of these five contributing internal forces is the sum of wind-induced forces multiplied each by different influence coefficients, as will be seen subsequently, and each has typically peaks occurring at different times. Rigorously accounting for such complex combinations of wind effects exceeds in practice the capabilities of the HFFB technique. The third and fourth points are inherent in the nature of the HFFB approach; see, e.g., Simiu (2011).

3. Division of tasks between the wind and the structural engineers

A well-known framework for the determination by the wind engineers of the structural response to wind is the "Chain of Wind Loading" (Fig. 1), proposed by Davenport (1982). The chain is in effect a flow chart that contains five items: "Wind Climate", "Influence of Terrain," "Aerodynamic Effects," "Dynamic Effects," and "Criteria" (to avoid confusion the term "item" is used here, instead of the original term "link," which is now part of the internet vocabulary). The outputs of the "Chain of Wind Loading" are estimates of wind effects on the basis of which the structural engineers perform the final sizing of the structural member.

To fully exploit the potential of time-domain methods, a change is needed in the division of tasks between the wind and the structural engineers. The tasks that the wind engineer needs to perform are aimed at providing the requisite *wind climatological, micrometeorological, and aerodynamic* information, in formats that lend themselves to effective use by the structural engineers and satisfy the need for transparency and accountability. Also required of the wind engineers are estimates of uncertainties in that information.

The end task of the structural engineers is to finalize the design of the main wind-force resisting system by sizing the system's structural members. The latter are subjected, in addition to gravity effects, to wind effects with specified MRIs. That task requires performing: calculations of the applied aerodynamic loads, based on pressure time histories provided by the wind engineer; calculations of the dynamic effects induced by those loads, corresponding to MRIs specified by the design criteria and on the basis of correct combinations of partially coherent contributory dynamic

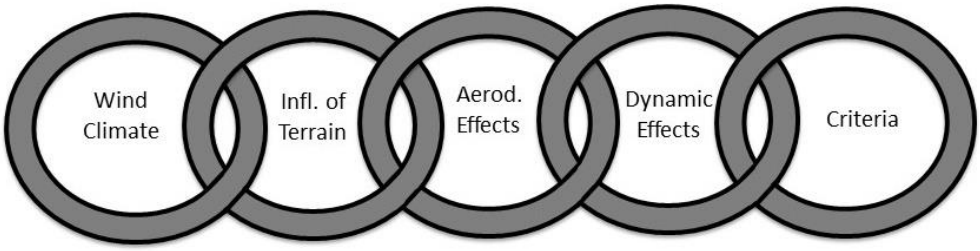

Fig. 1 The Chain of Wind Loading

effects; sensitivity studies on the effect on structural response of *uncertainties*, which include those estimated by the wind engineers as well as uncertainties associated with damping and structural stiffness; and *iterations* required to achieve risk-consistent, safe and economical member sizing. The iterations do not pertain just to the size of the member being designed, but also to the dynamic effects experienced by the structure as a result of successive changes in member sizes.

This division of tasks is effective in that it allows estimates of wind effects with specified MRIs to be performed without unnecessary guesswork on the combinations of dynamic effects and on the wind pressure distribution on the exterior surface of the building. The division of tasks is also efficient in that it eliminates the need for interactions between wind and structural engineers that sensitivity studies and iterative computations could require. The transparency inherent in this division of tasks allows rigorous accountability, as every step of the procedure is clearly and fully documented. Finally, this division of tasks enables structural engineers, ultimately responsible for the structural system's safety and cost, to be in full control of the structural design for wind loads, much as structural engineers are in full control of the structural design for seismic loads.

To the wind tunnel procedure for the design of high-rise buildings presented in this paper there corresponds the flow chart of Fig. 2, which covers the tasks of the wind engineers, and the flow charts of Figs. 3(a) and 3(b), which cover the tasks of the structural engineers. The content of those flow charts is explained in the next two sections of this work. The goal of those sections is to explain the wind tunnel procedure presented in this paper so that it is "widely understood by the designers using the results" (Irwin *et al.* 2013) in sufficient detail to allow not only its use but also its discussion and assessment by the design community.

4. Wind engineers' tasks

Wind engineers are best qualified to perform the tasks represented in Fig. 2. These tasks, listed and commented on in this section, are independent of – they generally require no contribution by or information from -- the structural engineers.

4.1 Wind climate

Obtain or develop by Monte Carlo simulation the matrix $[v_{nd}]$ of standardized directional wind speeds v_{nd} (n = 1, 2,…, n_{max}), where n_{max} is the number of storm events represented in the matrix and d (d = 1, 2, …, d_{max}) are the wind directions (e.g., 0, 10, 20,…350 degrees clockwise from the North, in which case d_{max} = 36).

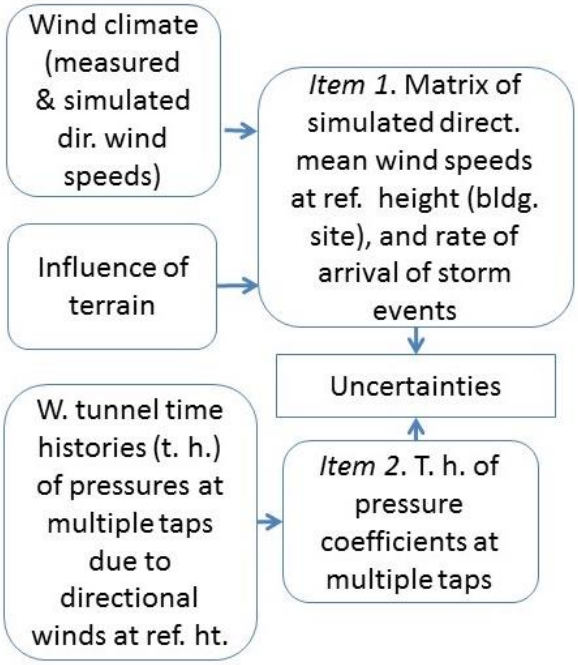

Fig. 2 Wind engineers' tasks. Items 1 and 2 are products of the three tasks shown on the left side of the flow chart

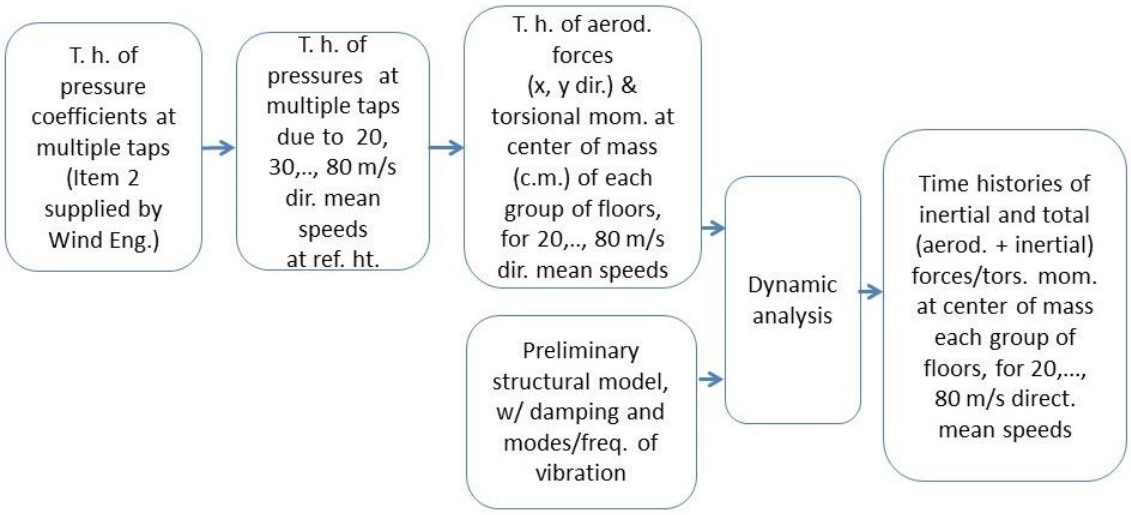

Fig. 3(a) Structural engineers' tasks (part 1)

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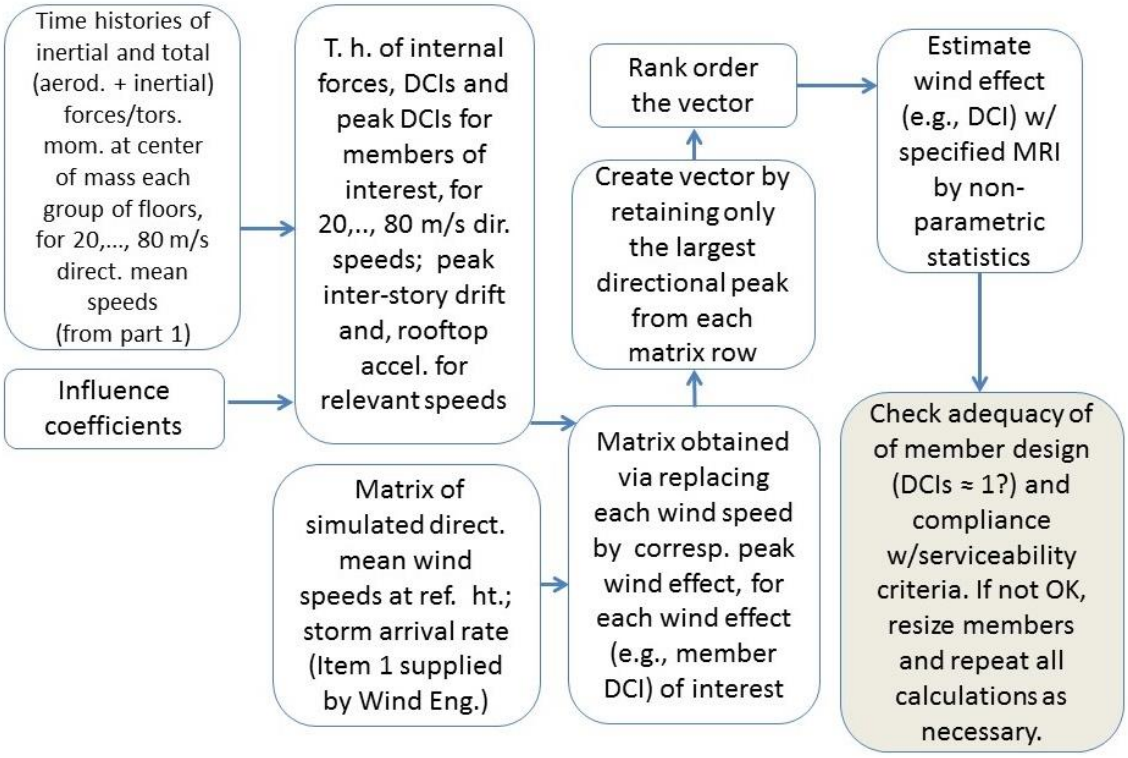

Fig. 3(b) Structural Engineers' tasks (part 2)

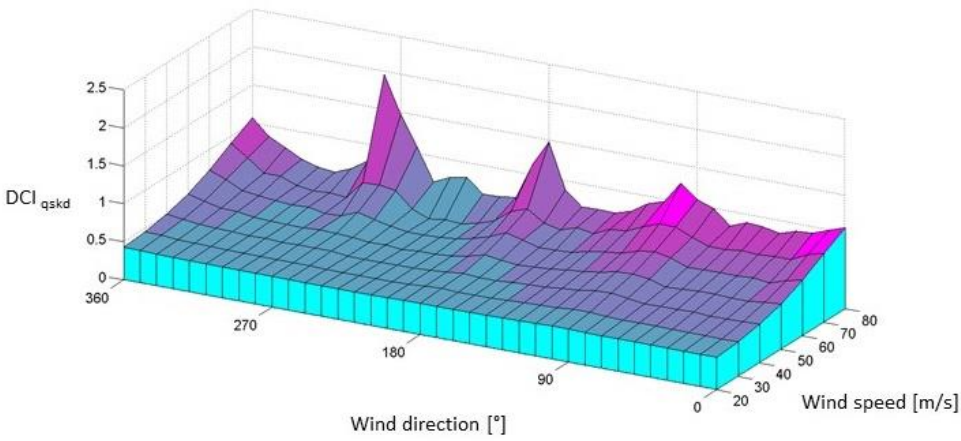

Fig. 4 DCI at cross section s of member q as a function of directional mean wind speeds V_{kd} at reference height above building site

Standardized wind speeds are defined in the United States as directional peak 3-s wind speeds at 10 m above terrain with open exposure; in some countries they are defined as directional peak 10-min wind speeds at 10 m above terrain with open exposure. The requisite number of storm events depends upon the *rate of arrival* of storm events, μ, which also needs to be provided by the wind engineer.

As will be shown subsequently, estimates of design wind effects with MRIs specified in the ASCE/SEI 7 Standard (2010), which for strength design are as large as 1700 years, require the use of *non-parametric statistics*, discussed later in the paper (for details see, e.g., Simiu *et al*. 2008, Spence 2009, Yeo 2010, Simiu 2011, Yeo and Simiu 2011, Yeo 2014). For this reason, to obtain adequate estimates of the design wind effects it is necessary that $n_{max}/\mu \geq c$ times the specified MRI, where c is sufficiently larger than unity to allow for reasonably precise estimates of wind effects with that MRI. Because measured non-hurricane wind speed datasets are relatively small (say, 15 to 50 years), large data sets must be obtained by Monte Carlo simulation; see Yeo (2014) for details. If measured data are available at several stations within the area surrounding the building, spatial statistics can be used to improve probabilistic models of the extreme wind speeds for each of the directions of interest (Pintar and Lombardo 2013). It is important that the spatial statistics used are supported by statistical theory, otherwise they can contribute to significant estimation errors, as has been the case for the estimated design wind speeds specified for non-hurricane areas in the ASCE/SEI 7-10 Standard and in earlier versions thereof (Simiu *et al*. 2003, Peterka and Esterday 2005, Simiu *et al*. 2005).

The large directional wind speed data sets required for strength design in mixed non-hurricane wind climates are generated separately from measured directional synoptic wind speeds and measured directional thunderstorm wind speeds, since these two types of wind are best fitted by different probability distributions (Lombardo *et al.* 2009) and have different rates of arrival μ_S and μ_T. Separate directional wind speed matrices, $[v_{Snd}]$ and $[v_{Tmd}]$, are therefore constructed for the synoptic and thunderstorm events, respectively. The number of rows of each matrix is, respectively, $n_{Smax}/\mu_S \geq c_S$ times the specified MRI, and $n_{Tmax}/\mu_T \geq c_T$ times the specified MRI, where c_S and c_T are larger than unity. While the probabilistic descriptions of synoptic and thunderstorm wind speeds differ, it is assumed conservatively in current practice that there is no phenomenological difference between those two types of storms. To improve upon this assumption future research on thunderstorm wind speeds would be required.

Wind climatological data sets are available for the conterminous United States on www.nist.gov/wind for directional non-hurricane wind speeds, separately listed as thunderstorms and synoptic; and for directional hurricane wind speeds. More recent directional hurricane wind speed data are available commercially. For hurricane wind speeds information see also Yeo *et al.* (2014).

4.2 *Influence of terrain*

Use relevant micrometeorological atmospheric flow models and flow management devices (e.g., roughness elements, spires) to (a) simulate in the wind tunnel the effect of the surface roughness within a sufficiently large area around the building, and (b) convert standardized directional speeds used in the description of the wind climate (e.g., peak 3-s speeds at 10 m over terrain with open exposure) into hourly (or, in some countries, 10-min) mean wind speeds at the site of the building, at the reference height considered in the definition of aerodynamic pressure coefficients.

The conversion is effected via multiplication of the standardized wind speeds by terrain-exposure-dependent micrometeorological factors r_d based on standard procedures available in the literature, for example the ASCE 7 Standard. This operation transforms the matrix $[v_{nd}]$ of the standardized wind speeds into a matrix $[V_{nd}]$ of the mean speeds at the reference height over the building site.

4.3 Aerodynamic effects

Acquire through wind tunnel measurements records of simultaneous time series of pressure coefficients of sufficient length (e.g., 10 min to 1 hr full scale), and with sufficiently high resolution (e.g., 500 Hz), measured at a sufficiently large numbers of taps.

The wind engineering report should specify the wind tunnel reference mean speed, its averaging time, and the height above ground (converted to full scale) at which it was measured and on the basis of which the pressure coefficients were obtained. In addition, the report should provide details on the wind tunnel (e.g., dimensions, length of test section, instrumentation) and its flow management devices (e.g., spires, roughness elements), and of the extent to which the expressions and parameters that describe the wind tunnel flow match their target counterparts in the atmosphere.

4.4 Uncertainties in the mean speeds and the aerodynamic effects

Provide measures of uncertainty for the aerodynamic data and the mean directional wind speeds at the reference height used in the definition of the pressure coefficients. Such measures contribute to establishing the value of the wind load factor or, alternatively, of the mean recurrence interval (MRI) of the design wind effects.

Uncertainties depend upon: (a) the size of the sample of *measured* data; (b) the estimated errors in the measurement of the data (for hurricane wind speeds the measured data consist of the pressure defect, the radius of maximum wind speeds, and other parameters that determine the strength and direction of wind speeds in tropical cyclones); (c) the degree to which (i) the micrometeorological models used to transform the measured data into standardized data (e.g., into 3-s peak gusts at 10 m above terrain with open exposure), and the standardized data into mean speeds at the reference height over the building site, are appropriate, and (ii) the probabilistic models used to obtain by simulation large data sets on the basis of relatively small sets of measured data are appropriate as well. Note that the uncertainties associated with sample size are not commensurate with the size of the large *simulated* data sets, but rather with the size of the smaller, *measured* data sets. If only fewer than, say, 15 years' worth of measurements are available, load factors larger than those specified in building codes and standards should be determined for design purposes by using standard structural reliability approaches for the estimation of total errors in the determination of wind effects.

The formal involvement of wind engineers in design is thus limited essentially to (i) a preliminary investigation of the building's overall aerodynamic performance, typically by using an HFFB approach, if such an investigation is deemed necessary (in which case the wind engineers interact primarily with architects, rather than structural engineers), and (ii) the tasks outlined above concerning the wind climate, the influence of terrain, aerodynamic effects, and uncertainties in the products of these three items. All the data produced in these tasks need to be provided for the record in electronic form to allow their ready use by the structural engineers.

5. Structural engineers' tasks

The structural engineers' tasks are aimed at producing final structural designs consistent with specified criteria. These should be based on physically realistic modeling of the wind loads and effects and thus achieve structural designs that are more risk-consistent, safe, serviceable, and economical in both cost and CO_2 footprint terms than is the case in conventional practice. The tasks are part of a "Chain of Structural Design for Wind" and fall under the rubrics "Criteria," "Dynamic Effects," and "Member Sizing." For member sizing and serviceability design purposes the dynamic effects of interest are those that determine the Demand-to-Capacity Indexes (DCIs) (discussed later in this section), the inter-story drift values, and the rooftop accelerations, with their respective specified MRIs. Examples of software for performing the structural engineers' tasks are available in Spence (2009) and Yeo (2010). Those tasks are listed below:

5.1 Criteria

The strength design criteria require that the DCIs with specified MRIs and load and resistance factors at critical cross sections of all the building's structural members be equal to or differ from unity by small amounts accepted in structural engineering practice; and that inter-story drift and top floor accelerations with the respective specified MRIs not exceed specified values.

The MRIs considered in design are typically specified in building codes and standards, and may be relatively short (e.g., 100 years), in which case the internal forces that appear in the expression for the DCIs are multiplied by wind load factors larger than unity, or are long (e.g., 700 or 1700 years), in which case the wind load factors are equal to unity, as is the case in the ASCE 7-10 Standard. Wind load factors, or MRIs that in effect incorporate them, depend upon the magnitude of the uncertainties affecting estimates of wind effects. It is therefore necessary to estimate the increase in the code wind load factors by accounting for unusually large uncertainties, especially in the wind speeds. For very tall buildings an investigation is also in order into the effect of additional uncertainties associated with structural stiffness and damping (Gabbai and Simiu 2014).

5.2 Dynamic effects

5.2.1 Produce a preliminary structural design.

This task can be accomplished, for example, by using response calculations based on methods incorporated in standards or codes or otherwise available in the literature (see, e.g., ASCE 7 Standard 2010, Chapt. 26), and sizing structural members on the basis of those calculated responses. Since the requisite wind engineering data are available, a more accurate approach to producing a preliminary structural design may be used.

Calculate, for each wind direction d, the time series of (i) the *total applied aerodynamic forces* acting along the building's two principal axes x and y at the center of mass of each group g of floors, and (ii) the *total applied aerodynamic torsional moments* about the center of mass.

Both the forces and the moments being calculated act at the level of each group $g = 1, 2, \ldots, g_{max}$ of f_g floors, where f_g is any number of floors (say, five or six) deemed appropriate by the analyst. The total applied forces and moments are resultants along the axes x and y, denoted by F_{agxkd} (t), F_{agykd} (t) and M_{agkd} (t), of all the elemental applied wind forces F_{aigkd} (t) and torsional moments M_{aigkd} (t) associated with pressure taps i located within the group of floors g. The

subscript "*a*" stands for "applied", *i* is the identifier of the pressure tap, *d* identifies the wind direction, and *k* identifies the magnitude of the mean wind speed $V_{kd}(z_{ref})$ at the reference height z_{ref} (e.g., the height of the top of the building) used in the definition of the pressure coefficients $C_{pig}(z_{ref}, t)$. We have

$$F_{aigkd}(t) = (1/2) \quad C_{pig}(z_{ref}, t)\, V_{kd}^2(z_{ref})\, A_{ig} \tag{1}$$

$$M_{aigkd}(t) = (1/2) \quad \rho\, C_{pig}(z_{ref}, t)\, V_{kd}^2(z_{ref})\, A_{ig}\, b_{ij} \tag{2}$$

where A_{ig} is the area attached to the pressure tap *i* within group *g*, and b_{ig} is the moment arm of the force $F_{aigkd}(t)$ with respect to the center of mass. Typical mean wind velocities $V_{jk}(z_{ref})$ for which calculations should be performed correspond to, say, 20 m/s, 30 m/s, …, 80 m/s for k =1, 2, k_{max} = 7, respectively, in increments of 10 m/s (although the analyst may choose different increments and a different value of k_{max}), and to directions d = 1, 2, …, d_{max} (e.g., 0, 10, 20, …, 350 degrees).

Perform multi-modal analyses yielding the time series of the *inertial forces* $F_{pgxkd}(t)$, $F_{pgykd}(t)$ and *inertial torsional moments* $M_{pgkd}(t)$ induced in the structure by the forces $F_{agxkd}(t)$, $F_{agxkd}(t)$ and torsional moments $M_{agkd}(t)$ (in this notation the subscript *p* denotes "inertial"). For details on the analysis of structures with non-coincident mass and elastic centers, see Simiu (2011).

Create the time series of the *total (applied aerodynamic + inertial)* forces $F_{gxkd}(t) = F_{agxkd}(t) + F_{pxkd}(t)$, $F_{gykd}(t) = F_{agykd}(t) + F_{pgykd}(t)$, and torsional moments $M_{gkd}(t) = M_{agkd}(t) + M_{pgkd}(t)$.

These time series form a database of the total forces and moments at a total of g_{max} levels, due to winds with a total of k_{max} speeds blowing from a total of d_{max} directions. The total number of time series in the database is 3 g_{max}* k_{max} * d_{max}.

Compute the *influence coefficients* consisting of the effects (e.g., internal forces, displacements, accelerations) induced by *unit forces* in the directions of the principal axes of the building and of *unit torsional moments*, applied at and about the building's centers of mass, at each of the g_{max} levels.

Calculate, for each wind speed k (k = 1, 2,…, k_{max}) and wind direction d (d = 1, 2,…,d_{max}) the time series of the responses of interest (internal forces, inter-floor drift, and top floor accelerations). For example, consider the bending moment induced at cross section *s* of member *q* by a unit force f_{qsgx} acting at level *g* in the direction of the building's *x* principal axis, and its counterparts induced by a unit force at level *g* in direction *y* and by a unit torsional moment at level *g*, f_{qsgy}, and m_{qsh}, respectively.

The total internal force M_{qskd} induced at cross section s of member *q* by wind with speed V_{kd} is

$$M_{qskd}(t) = \Sigma_g\, [F_{gxkd}(t)\, f_{qsgx} + F_{gykd}(t)\, f_{qsgy} + M_{gkd}(t)\, m_{qsg}] \tag{3}$$

Similar expressions hold for any other wind effect of interest. Fig. 4 shows a plot of the DCI at a cross section *s* of a member *q* for each of k_{max} mean wind speeds at the reference height, blowing from each of d_{max} directions. Wind effects that, in addition, reflect P-delta effects, rather than just linear summations similar to Eq. (3), are calculated as necessary in accordance with standard structural design practice. Eq. (3) can be applied to as many members as deemed appropriate by the structural designer. Since the fabrication process is being increasingly automated, traditional restrictions, typically associated with constructability considerations, on the numbers of members with different sizes are becoming less severe. This allows significant economies of material to be achieved. While computer intensive, the use of equations similar to Eq. (3) for large numbers of

members is consistent with modern computational capabilities available to major design offices, as is suggested by an example of an actual tall building to which the proposed methodology has been applied (Yeo and Simiu 2011).

Calculate the capacities of structural members used in the expressions for the DCIs.

For example, the expression for the DCI induced by wind V_{kd} at cross section s of member q is

$$DCI_{qskd}(t) = [\gamma_W N_{qkd}(t) + \gamma_D N_{Dq}]/(\phi_N N_{q\,cap}) + [\gamma_W M_{qskd}(t) + \gamma_D M_{Dqs}]/(\phi_M M_{qs\,cap}) \quad (4)$$

(It is assumed in Eq. (4) that the axial force is constant throughout member q, that live loads are negligible, and that bending moments occur at cross section s of member q about one axis of the cross section only). The numerators in Eq. (4) are typically called demands. γ_W and γ_D are load factors for the wind and dead loads, ϕ_N and ϕ_M are resistance factors. The load and resistance factors typically differ from standard to standard. This task consists of calculating the member capacities for all the structure's members in accordance with standard structural engineering practice. In the example of Eq. (4) the capacities of interest are $N_{q\,cap}$ and $M_{qs\,cap.}$

Calculate the demand-to-capacity indexes $DCI_{qskj}(t)$ for all members q and cross sections s being designed.

Estimate the time series and their peaks for the DCIs of all members q and cross sections s of interest, as well as for the requisite inter-story drift and top floor accelerations, by using the appropriate demands and capacities, and the member properties relevant to calculations for serviceability.

In the matrix $[V_{nd}]$ substitute for each mean velocity V_{nd} the peak of the corresponding wind effect of interest, that is, the peak wind effect of interest induced by the wind speed V_{nd}.

For example, for each critical cross section s of each member q of interest, a matrix $[(DCI_{qsnd\,pk}(t))]$, where "pk" denotes "peak," is obtained.

Not all the wind effects contained in matrices such as $[(DCI_{qsnd\,pk}(t))]$ are of interest.

Rather, for each storm event n (i.e., for each n-th row of the matrix), only the largest wind effect occurring during the event is of interest and is retained. For example, the matrix

$[(DCI_{qsnd\,pk}(t))]$ becomes a vector $\{max_d(DCI_{qsnd\,pk}(t))\}$.

Rank-order the vector $\{max_d(DCI_{qsnd\,pk}(t))\}$, which has n_{max} components. Assume that the rate of arrival of storm events is $\mu = 1$. The component of the vector with the lowest value has a 1 + 1 = 2-year MRI; the second lowest component has a 2 +1 = 3-year MRI; the 49th lowest component has a 50-year MRI, and so forth. However, if $\mu \neq 1$, then the first lowest component has a (1/μ)-year MRI, the 49th lowest has a (50/μ)-yr MRI; for example, if $\mu = 0.5$, to the 49th lowest vector component there corresponds, approximately, an MRI of 100 years. This explains why to obtain peak wind effects with an N-year MRI, where N is large, it is necessary to have simulated wind speed sample sizes that are commensurately large. As was pointed out earlier, software is available for the simulation of large data sets from smaller measured data sets. For mixed wind climates (e.g., thunderstorms and synoptic storms) the estimation of wind effects with specified MRIs can be performed in a straightforward manner as shown, e.g., in Simiu (2011). Recall that the matrix (or, in the case of mixed wind climates, matrices) $[V_{nd}]$ are supplied by the wind engineers.

5.3 Member sizing

If, as is typically the case, the design wind effects based on the original member sizes do not adequately satisfy the design criteria, re-size the members and repeat the tasks described above

until the structural design is satisfactory.

If necessary, a device to reduce dynamic effects associated with serviceability performance, typically a Tuned Mass Damper, may be added to the structure.

6. Conclusions

The following innovations that occurred primarily in the last two decades have led to the development of a wind tunnel procedure that results in improved structural designs of high-rise structures subjected to wind loads:

- Scanning devices capable of measuring simultaneously time histories of pressures at hundreds of taps on the exterior surfaces of buildings.
- Computer-intensive time domain analysis methods that allow the accurate, measurement-based calculation of dynamic effects and of complex combinations of wind effects. While the analysis methods are more elaborate than those typical of conventional practice, their use is entirely feasible given current computational capabilities available to structural engineers, and is fully justified given that the computation costs may be expected to be small compared to the cost benefits inherent in designs based on more accurate estimates of wind effects.
- The simulation of directional wind speed data sets covering periods of thousands of years from relatively small sets of measured data.
- The choice of the member Demand-to-Capacity Indexes (DCIs) with specified MRIs as the wind effects to be determined by the wind tunnel procedure for strength design purposes. The motivation for this choice is that DCIs embody with superior accuracy all relevant wind loading combinations, and that they are directly used by structural engineers for the sizing of structural members.
- The application for wind/structural engineering purposes of non-parametric statistical methods for the effective estimation of directionality-dependent peak wind effects with specified mean recurrence intervals (MRIs) of up to thousands of years.

The implementation of the procedure requires a change in the division of tasks between wind engineers and structural engineers. Wind engineers should be responsible for providing in transparent form, susceptible of being readily scrutinized, the requisite data on: (i) large sets of directional extreme wind speeds that incorporate information on the wind climate and the influence of terrain; (ii) the requisite pressure coefficient time series measured in wind tunnel testing; and (iii) estimates of uncertainties in the wind speeds and the aerodynamic pressure coefficients, for use in the development of wind load factors (or in the selection of mean recurrence intervals of the design wind effects) commensurate with those uncertainties. Structural engineers should be responsible for performing the response calculations and the iterations required to size the structural members in accordance with design criteria for strength and serviceability.

The structural engineers' tasks are automated. The wind tunnel procedure described in the paper eliminates guesswork in combining the various wind effects that contribute to the DCIs, the inter-story drifts, and the rooftop accelerations. The procedure supports clear accountability of the participants in the design process, and enables meaningful and reliable inter-laboratory comparisons, the credibility of which must be ensured by the independence of the entities

performing them and the public availability of the supporting documentation. Finally, the procedure enables structural engineers ultimately responsible for the structure's safety and cost to be in full control of the structural design for wind loads, much as they are in full control of the structural design for seismic loads.

This work is offered as a contribution to the future development of a standard on the wind tunnel procedure, and is intended to stimulate open discussion and further contributions by wind and structural engineers. Future work should include accounting for P-delta effects in the calculations, and the development of protocols on accounting for uncertainties in the response estimates, which are not covered in this paper. Finally, it is noted that the proposed Database-Assisted Design approach presented in this paper is also highly relevant to, among other structures, large roofs, such as roofs on sports facilities.

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