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Wind-induced tall building response: a time-domain approach

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Abstract. Estimates of wind-induced wind effects on tall buildings are based largely on 1980s technology. Such estimates can vary significantly depending upon the wind engineering laboratory producing them. We describe an efficient database-assisted design (DAD) procedure allowing the realistic estimation of wind-induced internal forces with any mean recurrence interval in any individual member. The procedure makes use of (a) time series of directional aerodynamic pressures recorded simultaneously at typically hundreds of ports on the building surface, (b) directional wind climatological data, (c) micrometeorological modeling of ratios between wind speeds in open exposure and mean wind speeds at the top of the building, (d) a physically and probabilistically realistic aerodynamic/climatological interfacing model, and (e) modern computational resources for calculating internal forces and demand-to-capacity ratios for each member being designed. The procedure is applicable to tall buildings not susceptible to aeroelastic effects, and with sufficiently large dimensions to allow placement of the requisite pressure measurement tubes. The paper then addresses the issue of accounting explicitly for uncertainties in the factors that determine wind effects. Unlike for routine structures, for which simplifications inherent in standard provisions are acceptable, for tall buildings these uncertainties need to be considered with care, since over-simplified reliability estimates could defeat the purpose of ad-hoc wind tunnel tests.

Keywords: aerodynamics; building technology; database-assisted design; directionality; structural dynamics; tall buildings; time-domain methods; wind climatology; wind engineering; wind tunnels.

1. Introduction

Methods for estimating wind-induced effects on rigid buildings are fairly well established. This is true to a much lesser extent of tall, flexible buildings. For this reason estimates of wind effects obtained for the same building by different wind engineering laboratories can differ significantly from each other. For example, as reported by an official National Institute of Standards and Technology investigation (NIST NCSTAR 1-2, 2005), estimates by two wind engineering laboratories of wind-induced base moments of the same building differed significantly, owing in large measure to differences in the respective models of the extreme wind speeds and of the joint directional effects of the aerodynamics and wind velocities. These modeling areas need to be addressed in future standards of practice. In view of such differences, to avoid designing structures on the basis of

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possibly inadequate estimates of wind effects, some structural engineering offices have a policy of commissioning two independent wind engineering reports.

Full access to the reports issued by the two laboratories allowed the detailed scrutiny of current practices. Such scrutiny is unusual, since wind engineering reports are typically confidential, even though they entail safety issues of major public interest. In addition to the discrepancies mentioned earlier, the lack of transparency of the reports was noted both by the NIST investigators and the independent consultants tasked with reviewing the investigators' work; that is, the reports or parts thereof were perceived as difficult if not impossible to understand and check even by experienced structural engineers with recognized expertise in wind engineering (NIST NCSTAR 1-2, 2005, pp. 329 ff).

The procedure described in this paper requires that the following data be provided to the structural engineer by wind engineering consultants:

i) aerodynamic pressures measured simultaneously in wind tunnel tests for a sufficient number of wind directions at a large number of pressure taps on the building envelope
ii) directional wind speeds obtained from climatological data and attendant simulations, and
iii) micrometeorological data simulated in wind tunnel tests, consisting of the ratio, for each wind direction, between wind speeds at 10 m elevation in open terrain and the corresponding mean hourly wind speeds at the top of the building being designed.

Once these data are made available by the wind engineering consultant, the structural engineer, with the possible assistance of a wind engineer, can use them for routine calculations of the building response to wind, including the internal forces and demand-to-capacity ratios used for the design of individual members, just as is typically done for a building subjected to seismic loads.

We first explain the basic features of the procedure, and its use when uncertainties pertaining to the factors that determine the wind loading are not explicitly taken into account. This case is applicable to the estimation of wind effects corresponding to a basic mean recurrence interval (MRI) associated with allowable stress design (ASD), or to a larger MRI that implicitly accounts for those uncertainties and is used for strength design (SD). Next we address issues associated with accounting explicitly for uncertainties in the factors that determine the wind effects. Unlike for routine structures, for which simplifications inherent in standard provisions are acceptable, for tall buildings these uncertainties need to be considered with care, since over-simplified reliability estimates could defeat the purpose of ad-hoc wind tunnel tests. In particular, we present calculations according to which the design of tall buildings for SD may need to account for uncertainties in the dynamic parameters, which play no role in the design of rigid buildings. An application of the procedure is then presented. The paper ends with a set of conclusions.

It is assumed throughout that the building does not exhibit significant aeroelastic effects, an assumption that is commonly also made for buildings designed on the basis of the High-Frequency Force Balance (HFFB) technique. Should this assumption need to be checked, this is currently typically done by testing in the wind tunnel a separate, aeroelastic model of the building. We note that, in addition, the building model must have sufficiently large dimensions in plan so that it can accommodate the tubing required for the pressure measurements.

2. Description of procedure

For the sake of clarity we present the procedure in the simplest possible terms. For details see (Main and Fritz 2006) and the HR-DAD (High-Rise Database-Assisted Design) software, both available on the site www.nist.gov/wind (an updated and highly efficient version of HR-DAD, named HR-DAD.1, is scheduled for release and posting on the same site in December 2008). In this section we consider the case where uncertainties in the factors that determine the wind loading are not explicitly accounted for.

2.1. Description of the building aerodynamics

Current pressure-measurement technology allows the simultaneous measurement and recording in the wind tunnel of pressures induced at large numbers of taps on the building envelope for each of a sufficiently large number of wind directions. Associated with the pressures are forces approximately equal to the pressures at the taps times the respective tributary areas. An aerodynamic database for a building consists of simultaneous pressure time histories at the taps for each of a sufficiently large number of wind directions. Note that pressure measurements must typically be performed for cladding design purposes. Provided that the pressure measurement instrumentation is capable of simultaneity-which is currently the case for virtually all wind engineering laboratories engaged in tall building testing-the measurements performed for cladding design can be used for structural design purposes as well. A sufficient number of pressure taps needs to be provided on the building surface. In the current state of the art the order of magnitude of that number has been reported to be as high as almost 500 (Tamura, *et al*. 1999, Ueda, *et al*. 1994). The pressure measurements yield information on the distribution of the pressures over the entire building surface. This information is *not* available if the HFFB method is used. The HFFB method provides information on the effect of the pressures at the building base and, therefore, under the assumption that the sway fundamental modal shapes are linear, on the respective generalized forces. However, it cannot provide the information required for corrections needed if the modal shapes are not linear. In the absence of simultaneous pressure measurements, that information must be guessed. The uncertainty inherent in the guessing can be significant in the presence of aerodynamic interference effects due to the vicinity of other tall buildings, in which case it may defeat the purpose of seeking accurate estimates of the response by resorting to wind tunnel testing. The conclusions of the HFFB method then need to be qualified accordingly. This is even truer if higher modes of vibration need to be taken into account, which the HFFB method cannot do, or if mode coupling occurs owing to the non-coincidence of mass and elastic centers. It is emphasized, however, that HFFB is advantageous for, e.g., the rapid evaluation of alternative aerodynamic shapes or for preliminary designs, and may be required for the testing of buildings with dimensions in plan that do not allow placement of pressure tubing or with façade features that prevent useful pressure measurements. Note that the potential of simultaneous pressure measurements on tall building models was recognized by Ueda, *et al*. (1994) and Tamura, *et al.* (1999), among others.

2.2. Description of the extreme wind climate

For hurricane-prone regions the extreme wind climate at the location of concern is described by directional wind speed data sets for large numbers of simulated storm events (e.g., 1,000 or even 10,000 hurricanes, with wind speeds generated by each hurricane for, say, 16 or 36 directions), and by the annual rate of occurrence of hurricane winds (see www.nist.gov/wind). If hurricane wind speed data are available only for 16 directions, as is the case for the only publicly available U.S. hurricane wind speed database, the software included in the site www.nist.gov/wind allows the data to be used conservatively in conjunction with aerodynamic data recorded for, e.g., 10° angular intervals. For non-hurricane regions similar wind speed data can be generated from observations by numerical simulation, following the procedure developed for hurricane wind speeds in (www.nist.gov/wind Section III A, Grigoriu 2006a, 2006b). (A detailed report on the application of this procedure to synoptic and thunderstorm winds is being developed by NIST for publication in early 2009.)

2.3. Estimation of the dynamic response and of wind effects for specified MRIs

Given the data of Section 2.1 and 2.2 above, it is possible to calculate dynamic responses induced by each wind event for each directional wind speed, by solving in the time domain the respective modal differential equations of motion of the system excited by the time-dependent forces applied at the taps. These equations yield the inertial forces acting on the structure at each floor level. For example, consider 1,000 storm events, and assume that for each storm event wind speeds are available for 16 directions. Then, for each storm event, 16 peak directional responses will be calculated. For that event, the response of interest from a design viewpoint is the largest of those 16 responses. This approach allows the construction of a one-dimensional set of 1,000 peak responses, each corresponding to one of the 1,000 storm events described by the climatological database. From this set it is possible to estimate, on a distribution-free basis, the peak response with any desired MRI (Simiu and Miyata 2006, p. 139). To fix the ideas we consider the following deliberately simple illustration.

Let the wind speed time series v_{ij} (in m/s) consist of three storm events (i = 1, 2, 3) with two wind directions (j = 1,2):

Event 1: 54 (dir. 1), 47 (dir. 2),
Event 2: 41 (dir. 1), 46 (dir. 2),
Event 3: 47 (dir. 1), 39 (dir. 2).

Let the aerodynamic coefficients $C_{p,j}$ be

0.8 (dir. 1), 1.0 (dir. 2)

The corresponding nominal wind effects are assumed to be, to within a constant dimensional factor, $[C_{p,j}v_{ij}^2]^{1/2}$, that is,

Event 1: 48 (dir. 1), 47 (dir. 2),
Event 2: 37 (dir. 1), 46 (dir. 2),
Event 3: 42 (dir. 1), 39 (dir. 2),

(e.g., for event 1, dir. 1, $[0.8 \times 54^2]^{1/2} = 48$).

Since for each hurricane it is the largest nominal wind effect that matters, the time series to be considered for design purposes is (in m/s)

Event 1: 48
Event 2: 46
Event 3: 42

Assuming further that the rate of occurrence of the storm events is 1/yr, it follows from the above calculations that the wind effects with approximate MRIs of 3 and 2 years are, respectively, 48 and 46 m/s.

If the calculations were performed without accounting for the directionality of the wind speeds and the largest pressure coefficient (in this example C_p = 1) were used for all directions, then the estimated wind effects with approximate MRIs of 3 and 2 years would be, respectively, 54 and 47 m/s. A 0.85 directionality factor is used in North America to account, albeit summarily, for directionality effects on rigid buildings. Multiplication of the 54 m/s and 47 m/s values by this factor would yield wind effects with approximate MRIs of 3 and 2 years of, respectively, 46 and 40 m/s, rather than 48 and 46 m/s, as obtained by using a physically realistic model.

It is seen that, if the event with an approximately 2-yr MRI were of interest, the difference between the physics-based result (46 m/s) and the result based on the use of the directionality factor (40 m/s) would be 15% (in this case, the directionality factor approach would yield an unconservative estimate). This is why wind engineers attempt, by using a variety of methods, to take wind and aerodynamic directionality into account other than by applying the directionality factor approach. For a detailed critique of those methods see (Simiu and Miyata 2006, Sect. 4.2.2). According to Isyumov, *et al*. (2003), the so-called out-crossing method, discussed in some detail in (Simiu and Miyata 2006), tends to underestimate the response. In addition, the out-crossing method, which relies in principle on parent populations, is problematic for two reasons. First, for large-scale extratropical storms it is not clear that the extreme value population and the presumed parent population belong to the same class of meteorological phenomena and, therefore, that statistical inferences based on the putative parent population are relevant for the extreme wind speeds. Second, for thunderstorms and tropical storms parent populations cannot even be identified (Simiu 2007). For a critique of the *sector-by-sector* method for estimating directionality effects, see (Simiu and Miyata 2006, Simiu and Filliben 2005).

An efficient method for performing the requisite response calculations is described in detail in (Main and Fritz 2006). The method consists of calculating for each direction the set of responses corresponding to a limited number of wind speeds covering the whole range of wind speeds in the climatological database, and of obtaining from that set the response for each speed within that database by interpolation. This method is incorporated in the HR-DAD software listed on www.nist.gov/wind.

2.4. Structural response

The response of interest can consist of the building deflections and accelerations, and of wind-induced internal forces, or of the contribution of wind to the demand-to-capacity ratio, for any MRI and any individual structural member. A wind-induced internal force can be obtained by linear superposition of the internal forces due to (a) the resultants, acting at the center of mass of each building floor, of the tributary aerodynamic forces in the sway modes, (b) the calculated inertial forces acting at the center of mass at each floor, (c) the aerodynamic torsional moments about the center of mass of each floor, and (d) the inertial tosiona moments associated with (c). Each of these forces and moments is multiplied by the respective influence coefficient for the member of interest. The influence coefficients can be obtained efficiently by using commercial structural analysis software. The case of non-coincident elastic and mass centers is easily accommodated (Simiu and Miyata 2006, Venanzi 2005).

In the approach to the estimation of the internal forces presented here the estimates specifically account, for each member, for its influence coefficients, so that the sum of the effects on that member of the pressures and inertial forces acting on the building can be easily calculated. If, instead of internal forces, demand-to-capacity ratios are considered, it is possible to verify directly whether members designed by using preliminary estimates of the wind loads are adequate or not (such preliminary estimates can be made by using along-wind response estimates (see, e.g., the along-wind response estimation software available on www.nist.gov/wind, Section II B, which can account for nonlinear modal shapes and higher modes of vibration). If they are not adequate the members are redesigned and the procedure is iterated until the demand-to-capacity ratio is satisfactory.

Note that, while for typical rigid structures it is assumed that the MRI of the wind effects is the same as the MRI of the design wind speed, *this equivalence does not hold for tall building design* performed on the basis of wind tunnel testing. The MRI of the wind effects of interest is calculated as a function of (i) the directional aerodynamics characterizing the building, and (ii) the directional wind climate at the site, as indicated in Section 2.3 and therefore differs from and should not be confused with the nominal MRI of the design wind speed specified in standards and codes.

What makes possible the approach described in this section is the fact that large numbers of ordinary differential equations can now be solved routinely and efficiently on the computer. This was not the case in the 1960s, when spectral (frequency domain) methods were introduced with a view to substituting algebraic for differential equations. While the computational time for a simple frequency-domain approach is shorter, this substitution entails the loss of all pertinent phase information, meaning that demand must be estimated by using large numbers of combinations of internal forces. Moreover, the demand-to-capacity ratio required for member design entails by definition the sum of a moment demand-to-capacity ratio and an axial force demand-to-capacity ratio. Each of these two ratios entails in turn a sum of contributions by two internal forces associated with wind effects in the principal sway directions. The spectral density of the peak demand-to-capacity ratio would thus involve several cross-correlation terms. This complicates the calculations significantly. In contrast, the time-domain approach automatically results in the calculation of the peak demand or demand-to-capacity ratio by one simple algebraic addition, since no phase information is lost. Peaks of the sums can be obtained (a) by exploiting the fact that internal forces in individual members are largely due to sums of numerous independent contributions, and may therefore be assumed to have Gaussian distributions or, where this assumption is not warranted, (b) by using simple methods based on an adaptation of the Rice method to non-Gaussian processes (see www.nist.gov/wind, item III B). The application of time-domain methods renders the estimation process simpler, and more direct, transparent, and accurate than is the case for frequency domain methods, while requiring acceptable computation times (of the order of a few hours for the full analysis of buildings with thousands of members, including the calculation of demand-to-capacity ratios for all the members).

As was mentioned earlier, the wind engineering consultant's basic task is limited to supplying the design engineer with the requisite aerodynamic, wind climatological, and micro-meteorological data. The procedure presented here then allows the structural engineer to routinely perform the structural design, without the wind engineering consultant being involved other than in an auxiliary role in structural engineering matters.

Finally, we note that the time-domain approach presented in this paper allows the use of modern methods for member design optimization under drift, acceleration, and strength constraints. For details, see Spence, *et al*. (2008).

3. Accounting for uncertainties

The wind load factor is defined as

$$\gamma_w = \frac{P_{SD}}{P_{ASD}} \tag{1}$$

where P_{SD} is the wind effect for SD, and P_{ASD} is the wind effect for ASD. P_{ASD} corresponds to the

MRI of the basic wind speed specified in the standard (typically 50 years), and to an approximately 50% quantile of the uncertainty distribution of the wind effect, as obtained by Monte Carlo simulation. The SD load combination may be defined by Eq. (2a) and the ASD load combination by Eq. (2b) (see for example ASCE 7-05 Standard, Chapter 2, to which we refer in this section for specificity; however, the issues discussed in this paper are applicable to any standard or code):

$$1.2D + L + \gamma_w W_{50\text{-}yr,\,50\%} \tag{2a}$$

$$D + W_{50\text{-}yr,\,50\%} \tag{2b}$$

In Eqs. (2a) and (2b), D is the dead load, L is the live load, $W_{50\text{-yr},50\%}$ is the wind load corresponding to a 50-yr MRI and a 50% quantile. Similar expressions are specified in other codes and standards. The wind load factor γ_w can be estimated as in Ellingwood, *et al.* (1980). The estimation must account for the fact that the wind effects experienced by the structure during its lifetime could well be larger than those due to the 50-yr wind effects traditionally associated with ASD. This could be the case because: (1) owing to the inherent randomness of the wind speeds, the actual wind effect differs from the 50-yr effect, and (2) owing to knowledge uncertainties, the actual values of the factors that determine the wind effect are different from their estimated values. This is the basis for the calculations of the wind load factor adopted in the ASCE 7-05 Standard for rigid structures, see (Ellingwood, *et al.* 1980, pp. 114-118). Since, owing to the random variability of the wind speeds *and* the various knowledge uncertainties, the 50-yr wind effect is a random variable, the aim is to estimate its probability distribution. Once that distribution is available, a percentage point of that distribution (90%, say), to be determined by calibration, is used for strength design (SD) purposes. Note related contributions by Kasperski (2003) and Kasperski and Geurts (2005).

Thus, to calculate the wind load factor it is necessary to make *specific assumptions* concerning (1) the random (aleatory) variability of the 50-yr load, and (2) the knowledge (epistemic) uncertainties in the factors determining the wind effect. For example, in Ellingwood, *et al.* (1980) the assumptions concerning the random variability of the 50-yr wind effect are: (a) the wind speeds and the aerodynamics are independent of direction, (b) directionality is accounted for via a specified directionality factor, (c) the wind speeds have an Extreme Value Type I distribution with known parameters, and (d) wind loads are proportional to the second power of the wind speeds. The assumptions concerning the knowledge uncertainties are: (a) the sampling, observation, and micrometerological errors in the estimation of the wind speeds, and (b) the knowledge uncertainties in the aerodynamics and the gust factor for the quasi-static response of the rigid structure, are known. A calibration is then performed on the basis of a simplified relation between the wind load factor and the first-order second-moment safety index adjusted for conformity with past practice.

Several cases will now be considered: (1) rigid structures in non-hurricane regions; (2) rigid structures in hurricane-prone regions; (3) flexible structures.

(1) For typical *rigid structures in non-hurricane regions* the wind load factor has been estimated, and specified in the ASCE 7-05 Standard, to be 1.6, corresponding to a wind speed with a mean recurrence interval (MRI) of approximately 500 years. Strength design is thus performed, in effect, for wind speeds with an approximately 500-yr MRI. This *implicitly* assures that uncertainties with respect to the factors that determine the response are accounted for.

(2) For typical *rigid structures in hurricane-prone regions*, owing to distributions of the extreme wind speeds different from those assumed to hold for non-hurricane regions, wind load factors corresponding to a 500-yr MRI of the wind speed are larger than 1.6. To avoid confusing the user,

the Standard specifies the same nominal wind load factor (i.e., 1.6) for rigid structures in both non-hurricane and hurricane regions. However, instead of specifying for ASD a 50-yr basic wind speed, it defines the basic wind speed as the 500-yr wind speed divided by the square root of 1.6. Owing to the longer upper tail of the hurricane wind speed distribution, the MRI of that basic wind speed is larger than 50 years.

(3) For *flexible buildings* tested in the wind tunnel it is typically not appropriate-and it is not current practice-to use speeds estimated without regard to direction in conjunction with a constant directionality factor. In addition, because total uncertainties in the wind effect are augmented by the presence of uncertainties in the dynamic response parameters, especially for super-tall buildings, the probability distribution of the total uncertainties will have a longer upper tail than for rigid structures. This is true for both non-hurricane and hurricane regions. The assumption used in the ASCE 7-05 Standard's simplified and analytical methods that the value of the wind load factor is 1.6 should therefore not automatically be adopted for use with the wind tunnel method. That value reflects a specific set of assumptions applicable, approximately, to typical *rigid buildings in non-hurricane regions*. If that set of assumptions changes, so will the value of the wind load factor. Since additional uncertainties come into play in the case of tall buildings, a wind load factor of 1.6, or a corresponding MRI of 500 years, estimated for rigid buildings, does *not* provide the same reliability level for rigid and flexible structures. Neither the 1.6 wind load factor nor the 500-yr MRI-or any other fixed MRI used with a load factor larger than or equal to unity-are universally valid "magic numbers." Using them for different physical conditions than those for which they were derived would be inappropriate and defeat the purpose of the wind tunnel method, which is to provide as accurate a measure of wind effects as the state of the art permits. We note that Holmes and Pham (1993) discussed the requirement that for dynamically responsive structures the load factor must be higher if the design wind loads are based on a low MRI. Note that the Australian Standards (Standards Australia 1989, 2002) have used wind speeds with a high MRI (1000 years for tall buildings) to calculate design wind loads for ultimate limit states for nearly twenty years.

We now calculate load factors for cases (1), (2), and (3) above by using a slightly modified version of the Ellingwood, *et al.* (1980) approach, enhanced by the current ability to perform much more powerful Monte Carlo simulations. The random wind effects reflecting knowledge uncertainties are obtained via multiplication of the wind effects, calculated without accounting for those uncertainties, by random uncertainty factors a, b, c, and d.

The wind speeds are affected by the product bcd, where b is a measure of modeling, sampling, and observation errors in the estimation of extremes, c is a measure of uncertainties in the wind speed conversion from 10 m above open terrain to the top of the building, and d accounts for the random (aleatory) variability of the wind speeds inducing the 50-yr wind effect. For tall buildings the probability distribution of the uncertainties in the wind effect must also account for uncertainties in the modes of vibration and the modal damping ratios via random uncertainty factors T and D. Given all the individual uncertainties, that probability distribution can be estimated by using Monte Carlo simulations, by which large numbers of realizations of the set of uncertain design parameters are obtained, the wind effects corresponding to each realization being calculated by using the procedure presented in the previous section.

Instead of accounting for the aleatory variability of the 50-yr wind effect by using the factor d defined earlier, it is possible, with no significant loss of accuracy, to replace in Eq. (2a) the term $\gamma_w W_{50\text{-yr},50\%}$ by $W_{N,p\%}$, where, for the application presented in the following section, preliminary calculations showed that $N \approx 250$ years. The quantile $p\%$ was assumed to be 90%. In this paper we are interested in the *relative* magnitudes of the wind load factors for the building assumed to be on the one hand rigid, and on the other flexible, with knowledge uncertainties in the dynamic response parameters taken into account or disregarded. In general the uncertainty models should be determined by professional consensus. However, for illustrative purposes that should be helpful to code writers and practicing structural engineers interested in performing future studies and calibrations aimed at improving standard provisions, useful approximate indications on the *relative* magnitudes of the wind load factors for the cases just listed can be obtained if reasonable, though not necessarily "exact," values of a, b, c, N and p are used in the calculations.

4. Application

In this section we explore the wind load factor issue with specific reference to a building for which the requisite aerodynamic and wind climatological data are available. We consider a 60-story building with height $H = 183$ m, rectangular plan dimensions $B = 47$ m and $H = 30$ m, wind speed direction defined by the counterclockwise angle θ, with the positive x axis normal to the long sides of the rectangle, and axis y normal to the short sides of the rectangle. A building with these characteristics, known as the CAARC (Commonwealth Advisory Aeronautical Research Council) building, was studied by various researchers (e.g. Wardlaw and Moss 1971, Melbourne 1980, Venanzi 2005). The building is assumed to be located near New York in horizontally uniform suburban terrain. Estimated hurricane wind speeds for that area are taken from the site www.nist.gov/wind.

The simultaneous pressure time series at the taps placed on the model surface were measured at the Prato (Italy) Inter-University Research Centre on Building Aerodynamics and Wind Engineering (CRIACIV-DIC) Boundary Layer Wind Tunnel (Venanzi 2005). A 1:500 model of the prototype building in homogeneous suburban terrain was constructed and fitted with 120 pressure taps (30 taps per face, see Fig. 1). While for commercial applications a larger set of pressure taps would be used, our purpose was limited to achieving a successful application of the HR-DAD software. The sampling frequency for the wind tunnel measurements was 250 Hz, and the duration of each pressure record was 30 s (i.e. the number of data points in each time series was 7,500). Additional details are available on www.nist.gov/wind.

The response being considered consists of the demand-to-capacity ratio for the steel members of the structural system represented schematically in Figs. 2 and 3. The fundamental modal shape corresponding to each mode of vibration was assumed to be linear. The mean values for the fundamental periods of vibration in directions x and y are 5.84 s and 5.66 s, respectively. The probability distribution of the damping was assumed to be lognormal with mean 1.64%, median 1.5%, and c.o.v. 0.44 (Fritz 2003). The increase in the damping ratio with building deflections was not taken into consideration. Our results must be qualified in light of this simplification, and further research should be performed into the use of a more elaborate damping model. The lateral force resisting system of the building of Figs. 2 and 3 consists of steel frames and a bracing system. The columns are evenly spaced at 7.62 m in both the x- and y-directions, thus forming a mesh of 4 bays in the x-direction and 6 bays in the y-direction (see Fig. 2). The core, which has 4 internal bays, is stiffened by steel bracings on all floors. Cross bracings are also provided on the perimeter of the building between floors 19 and 21, 39 and 41, and 59 and 60. The concrete floor slabs are 0.1633 m thick. The live load was assumed to be 980 Pa on each floor. The building was designed in accordance with the EUROCODE 3-1993 code.

Monte Carlo simulations were performed for each realization of the process determined by the random variables a, b, c, T, and D. As mentioned earlier, we assumed $N = 250$ years and $p = 90\%$.

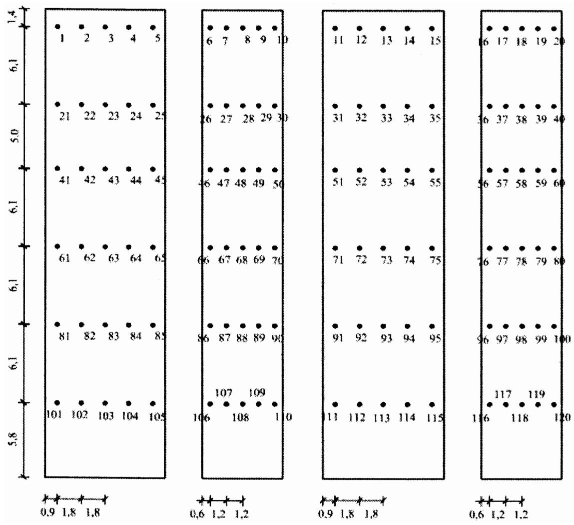

Fig. 1 Location and numbering scheme of pressure taps. Shown from left to right are the west, south, east, and north face, respectively (see Fig. 3). Dimensions in meters × 10^{-2}

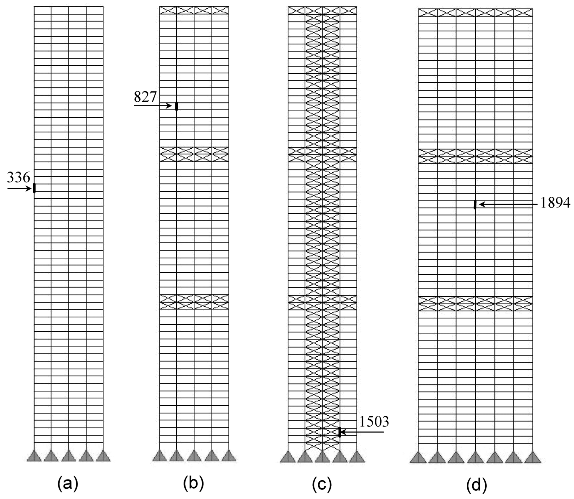

Fig. 2 Building cross-sections (see Fig. 3) a-a: Column 336 (106.68 m $\le z \le$ 109.73 m); b-b: Column 827 (140.21 m $\le z \le$ 143.26 m); c-c: Column 1503 (6.10 m $\le z \le$ 9.14 m); d-d: Column 1894: (100.58 m $\le z \le$ 103.63 m)

The calculations, using the procedure for estimating wind effects described earlier in the paper, were repeated for 350 realizations It was determined that the number of realizations $m = 350$ was adequate for the purposes of this paper (recall that wind speed estimates are typically based on less than 100 years of data; this precludes the attainment of great precision in the calculations, and using very much larger values of m would not improve the overall precision of the estimates being sought). Calculations were performed for four columns depicted in Figs. 2 and 3. The following four cases were considered:

1. The building is rigid and there are no knowledge uncertainties (Case I).

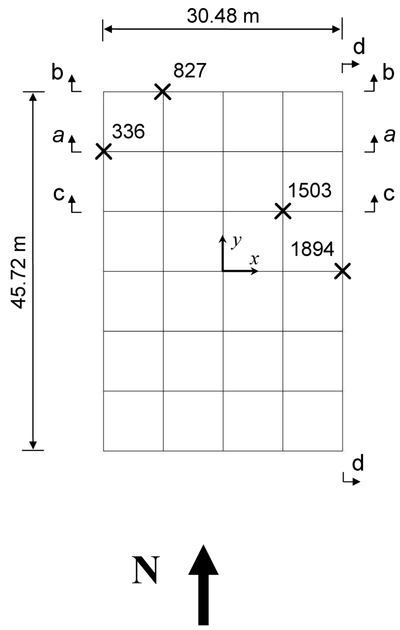

Fig. 3 Plan view of building with locations of members and cross sections of Fig. 2

2. The building is flexible and there are no knowledge uncertainties (Case II).
3. The building is flexible. The pressures are affected by an uncertainty factor *a* that reflects experimental errors in wind tunnel measurements, and the wind speeds are affected by the product *bc*, where *b* is a rough measure of sampling errors in the estimation of extremes, and *c* is a measure of uncertainties in wind speed conversion for wind speeds from 10 m above open terrain to the top of the building; *a*, *b*, and *c* have truncated normal distributions with unit mean and 5%, 7.5%, and 5% coefficient of variation (c.o.v.), respectively. The fundamental periods of vibration $\{T\}$ and the damping ratios $\{D\}$ are deterministic (Case III).
4. The building is flexible, with the uncertainties of Case III, except that $\{T\}$ and $\{D\}$ are random (Case IV). For the building being considered (which, as was noted earlier, is a steel building), $\{T\}$ is assumed to have a truncated normal distribution with 5% c.o.v. and specified mean values. $\{D\}$ is assumed to be lognormally distributed, as indicated earlier (Fritz 2003).

The results are shown in Table 1. Note that for Case I (i.e., the case corresponding to perfectly rigid buildings in the absence of knowledge uncertainties) the calculated wind load factor can be as low as 1.74 (column 1503). This is roughly consistent with a wind load factor applicable to hurricane-prone regions which, owing to the relative tail lengths of hurricane vs. non-hurricane wind speed distributions is larger than 1.6 when referenced to the 50-year event, as was noted earlier.

A comparison of the values for Case II with those for Case I shows that the flexibility of the building results in an increase of the load factor. Since no uncertainties are involved in Case II, the increase can be attributed to the fact that for flexible buildings the response increases with wind speed faster than it does for rigid buildings. For Case IV the increase of the load factor with respect to Case III reflects the effect of the uncertainties with respect to the vibration periods and damping.

The results of Table 1 suggest that, if the design provisions are adequate for the rigid building, they may be inadequate from a safety point of view for the building experiencing dynamic effects under wind loads. As noted earlier, the results depend upon the assumptions on the uncertainties

Table 1 Calculated load factors

Member	Case I[b]	Case II[c]	Case III[d]	Case IV[e]
336[a]	2.03	2.16	2.57	2.93
827[a]	2.01	2.03	2.25	2.66
1503[a]	1.74	2.07	2.44	2.68
1894[a]	1.85	2.24	2.50	2.83

[a]Columns are pinned-pinned.
[b]Rigid building, no uncertainties.
[c]Flexible building, no uncertainties.
[d]a, b, c random, $\{T\}$ deterministic, $\{D\}$ deterministic;
MRI = 250 years, quantile = 90%.
[e]a, b, c random, $\{T\}$ random, $\{D\}$ random; MRI = 250 years; quantile = 90%.

affecting the wind effects, in particular the uncertainties regarding the structure's dynamic characteristics, and on the choice of MRI and quantile in Eq. (1). At least some if not all the actual uncertainties are likely to be larger than those assumed here. In that case the calculated individual wind load factors would increase. In addition, our assumption with respect to damping does not account for the likely increase of the damping ratio with the MRI of the wind effects. Finally, estimates of wind load factors or corresponding MRIs of wind effects for SD (e.g., 500 years, or 1000 years, say) would be different for buildings with other dimensions and dynamic characteristics. For these reasons the results of Table 1 provide only a qualitative picture of the relative magnitudes of the estimated wind load factors for flexible vs. rigid buildings, and for flexible buildings for which uncertainties on natural frequency and damping are disregarded vs. flexible buildings for which those uncertainties are taken into account. Our objective was to draw attention to a potential structural reliability estimation problem, and to outline a procedure for investigating the extent to which such a problem may be significant for any given tall building.

5. Conclusions

We presented a procedure for estimating wind effects on tall, flexible buildings that takes advantage of two important technological developments of the last three decades. First, pressure transducers are now available that allow the simultaneous measurement of pressures at taps distributed over the entire exterior surface of a building model. The use of such pressure taps results in a complete definition of the pressure field on the building surface. This eliminates possible inaccuracies associated with the unknown pressure field that affect internal force estimates obtained by using the High-Frequency Force Balance (HFFB) method for: buildings with non-linear fundamental translational modes of vibration, especially in the presence of aerodynamic interference effects due to neighboring buildings; buildings with significant effects of higher modes of vibration; and buildings with coupled modes of vibration due to non-coincident mass and elastic centers.

Second, time-domain calculation methods can now be performed routinely for dynamical systems such as tall buildings. The loss of phase information inherent in the spectral approach creates problems when estimating internal forces and demand-to-capacity ratios, which require superpositions of random effects such as moments and axial forces or internal forces due to different modes of vibration. These problems are dealt with in the spectral approach by considering various combinations of wind effects, based in part on guesswork. In contrast, in the time domain, which preserves all phase relationships, all superpositions are obtained by simple algebraic addition of known effects. In addition, some engineers view the transformations inherent in the frequency domain approach as unintuitive and lacking transparency. The time domain approach can account, if necessary, for effects of higher modes of vibration, and renders practical the use of member-specific influence coefficients for the accurate estimation of internal forces and demand-to-capacity ratios for each individual structural member. Limitations of the time-domain procedure are: (1) the building must not be sensitive to motion-induced effects-as is also the case for the HFFB method; (2) the building model must have sufficiently large dimensions in plan so that it can accommodate the tubing required for the pressure measurements; (3) the building facade must not have features that render pressure measurements impractical.

The advantages of the proposed method notwithstanding, it noted that the HFFB methods is warranted for: the analysis of buildings with simple shapes, linear or almost linear fundamental sway modes of vibration, weak mode coupling, and weak higher mode effects; rapid evaluations of alternative aerodynamic shapes; and testing buildings with façade features that preclude effective pressure measurements, or are too slender to accommodate tubing required for such measurements.

Depending upon building type, post-elastic strength reserves may be available that could compensate for the lower reliability of tall buildings designed without accounting for dynamic uncertainty effects. Short-duration load effects have been invoked in this connection, as well as estimates of natural frequencies that may be favorably biased from the point of view of the dynamic effects experienced by certain types of building. The dependence of damping on deflections may also contribute to reducing the response. However, whether these effects can be relied upon for any particular building needs to be evaluated with care. For example, short-duration load effects may not be counted upon in the absence of ductile behavior under dynamic loads; relying on increased damping without the benefit of reliable relevant information may be imprudent; and relying on biased estimates of natural frequencies would be unwarranted for many types of steel buildings.

According to our results, disregarding the effect of errors inherent in the dynamic modeling of tall buildings subjected to wind loads can lead to reliability levels that would be lower for tall buildings than for typical rigid buildings. The differences between reliability levels would be even larger for buildings taller and more flexible than the building considered in this paper.

Our results suggest that additional research is in order. Analyses based on the methodology presented in this paper should in our opinion be performed for tall buildings with significant dynamic response. Calibrations should be performed based on the results of the analyses and on consensus among code-writers and practitioners. It is pointed out that the wind load factor issues considered in this paper pertain primarily to buildings for which wind tunnel tests need to be performed, as opposed to buildings whose design does not require ad-hoc wind tunnel testing.

Finally, we note that the physically more realistic estimation of demands inherent in the time domain approach described in this paper allows modern design optimization procedures to be applied to tall buildings subjected to wind loads.

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