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The past, present and future of high-frequency balance testing

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Abstract. Less than 30 years ago a new method was introduced in wind-tunnel testing of tall buildings, known variously as the High-Frequency Base Balance or High-Frequency Force Balance, which revolutionized the determination of design wind loads using model studies. The method is reviewed in hindsight, in the perspective of the present, and with a crystal ball to speculate on future developments. These viewpoints focus on various technical issues that have been solved, are being solved, and need to be solved. The intent is to assist the uninitiated develop appreciation for the technology involved, to identify various pitfalls awaiting those who embark in the method, and to identify areas of need so that practicing design engineers—the users of such studies—can appreciate the limitations and collaborate on future advances while promoting improved communication between executor and user.

Keywords: tall buildings; wind tunnel; high-frequency balance; dynamic; aerodynamic; PSD; structural

1. Introduction

A selective overview of high-frequency balance testing to obtain structural wind loads on tall buildings is presented. The intent is to draw attention to a variety of interesting aspects that represented specialized technical achievements, may not be well understood, or need continued refinement in the future. It is not meant to address, or even identify, the many additional technical aspects of the test method, which have been documented elsewhere (Tschanz and Davenport 1983, Boggs and Peterka 1989), nor comprehensive in describing applications that may be, or need to be, developed in the future. Some of these are described in detail in the other papers of this *Special Issue*.

2. High-frequency balance testing: a family member

It is instructive to consider the method's place within the realm of tools used in the wind tunnel to define loads on structures, Fig. 1. The traditional test uses an aeroelastic model, in which the model itself, as well as the simulated environment, is considered a complete scaled analog of the prototype. It is the *aeroelastic* forces that are observed and measured, which include the effect (if any) of the model's motion on the airflow around it. The alternate technique, which may be

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designated an *aerodynamic* model, involves a rigid model which, when placed in a scaled simulated test environment, is capable of sensing the aerodynamic loads imparted to it. This can be done in a variety of ways, and at present there are two popular methods. The first to be developed—and perhaps still most popular—is the use of a balance to support the model and is instrumented to sense the aerodynamic loading. As we shall see, this requires that the balance/model system have a high bandwidth, and so it is commonly referred to as a *high-frequency* balance to distinguish it from, say, a scaled-frequency balance that might be used for an aeroelastic model, or a balance/model system that is “soft” and allows the model to resonate at frequencies within the range of interest of aerodynamic loads. The second technique, known variously as *multi-pressure* or *high-frequency pressure integration*, uses a rigid model containing a multitude of pressure taps. If there is a sufficient number of taps (usually hundreds) and they can all be dynamically recorded simultaneously, then this can also characterize the aerodynamic loading. Other techniques have been experimented with in the past, such as pneumatic averaging (Surry and Stathopoulos 1977) and pressure cross-correlation (Kareem 1981, 1982, Reinhold and Sparks 1979, Reinhold 1983) and others may be developed in the future. Regardless of the measurement technique, however, an *aerodynamic* model is defined herein as *any model/instrument system designed to record the aerodynamic loading in a manner to enable the subsequent analytical prediction of static and dynamic response of the prototype structure, neglecting any motion-induced effects.*

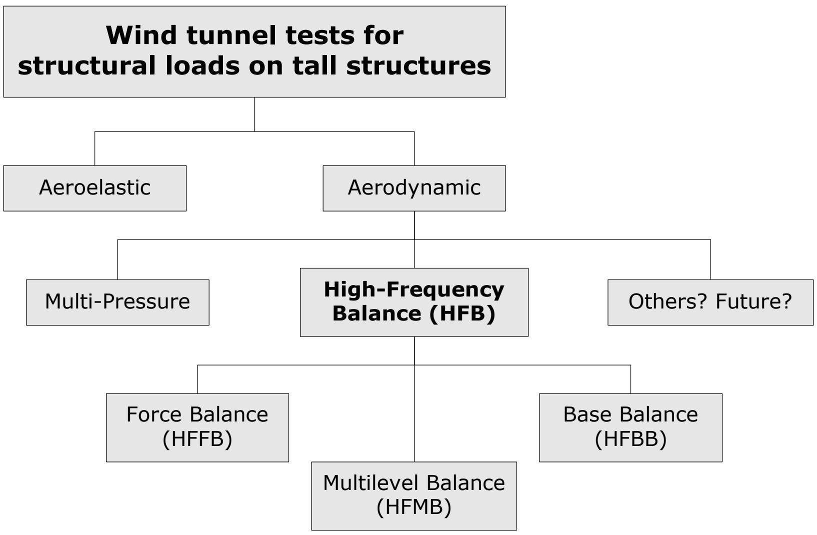


Fig. 1 High-frequency balance testing in the family of wind tunnel tools

Within the realm of high-frequency balance testing, three types may be distinguished depending on the measurand (see Fig. 2 and Section 3.1). Most significant is the base moment, because it is closely related to the generalized load in the three fundamental modes of vibration. A balance measuring only these components is called a *high-frequency base balance* (HFBB). In some cases the base shear is also measured, which can sometimes result in improved estimates of the generalized loads and first-order adjustments to the distribution of mean and background loads when a conventional pressure-model test is not performed. This is commonly known as a *high-frequency force balance* (HFFB), although this is somewhat of a misnomer, because both moments and shear forces are measured and the moments remain most important. Although not widely used, even better estimates of load distributions and generalized modal loads can be obtained by measuring the aerodynamic loading at two or more heights, using a *high-frequency multilevel balance* or HFMB.

3. Fundamentals

3.1 Theoretical requirements

The essence of the HFB method is to provide a direct method for measuring the generalized load in the fundamental modes of vibration of the tower, defined as

$$P^*(t) = \int_0^H w(z,t) \cdot \phi(z) \cdot dz \tag{1}$$

where $w(z,t)$ is the fluctuating load per unit height of the building (here we assume one-dimensional mode shapes involving a single response component). If the mode shape is scaled (normalized) to approximate the straight line z, i.e., $\phi(z) \approx z$ as in Fig. 2(a), then

$$P^*(t) \approx M(t) \tag{2}$$

An approximation to the generalized load is therefore easily sensed by the moment channel of a balance located at the zero elevation of the straight line. Deviations of the mode shape from the ideal shape are accounted for by so-called mode-shape correction factors. This fortuitous relationship breaks down for torsion-related modes, for which the ideal mode shape is constant instead of (z), resulting in a poor fit as depicted in Fig. 2(b), and a corresponding larger and more critical correction factor. Often a sway shape will fit the ideal line better by raising the zero elevation as in Fig. 2(c). Higher modes of vibration can be accommodated by fitting a piecewise linear ideal shape, as in Fig. 2(d). Although seldom utilized, the analysis of this technique is of considerable interest: If the nodes are located at elevations z_L and z_U, then the ideal mode shape is

$$\begin{aligned} \phi(z) &\approx h_1(z - z_L), & z_L < z < z_U \\ &\approx h_1(z - z_L) + h_2(z - z_U), & z_U < z < H \end{aligned} \tag{3}$$

where h_1, h_2 are the fitted incremental slopes of the lower and upper portions of the ideal shape. The generalized load becomes

$$\begin{aligned} P^* &\approx \int_{z_L}^{z_U} wh_1(z - z_L)dz + \int_{z_U}^{H} wh_1(z - z_L)dz + \int_{z_U}^{H} wh_2(z - z_U) \\ &\approx h_1 \int_{z_L}^{H} w(z - z_L)dz + h_2 \int_{z_U}^{H} w(z - z_U)dz \end{aligned} \tag{4}$$

or

$$P^*(t) \approx h_1 M_L(t) + h_2 M_U(t) \tag{5}$$

where M_L, M_U are the moments measured at the lower and upper nodes. In principle a mode shape of any complexity can be treated by measuring the moments at the nodes of a piecewise linear approximation, and forming a weighted sum according to the change in slope at each node. In this HFMB technique, the moments are usually measured by fitting strain gages to the core of the model, between gaps in the model shell. The method is ideally suited to certain structures, such as smokestacks and space launch vehicles, where the strain gages can be fixed to the model itself.

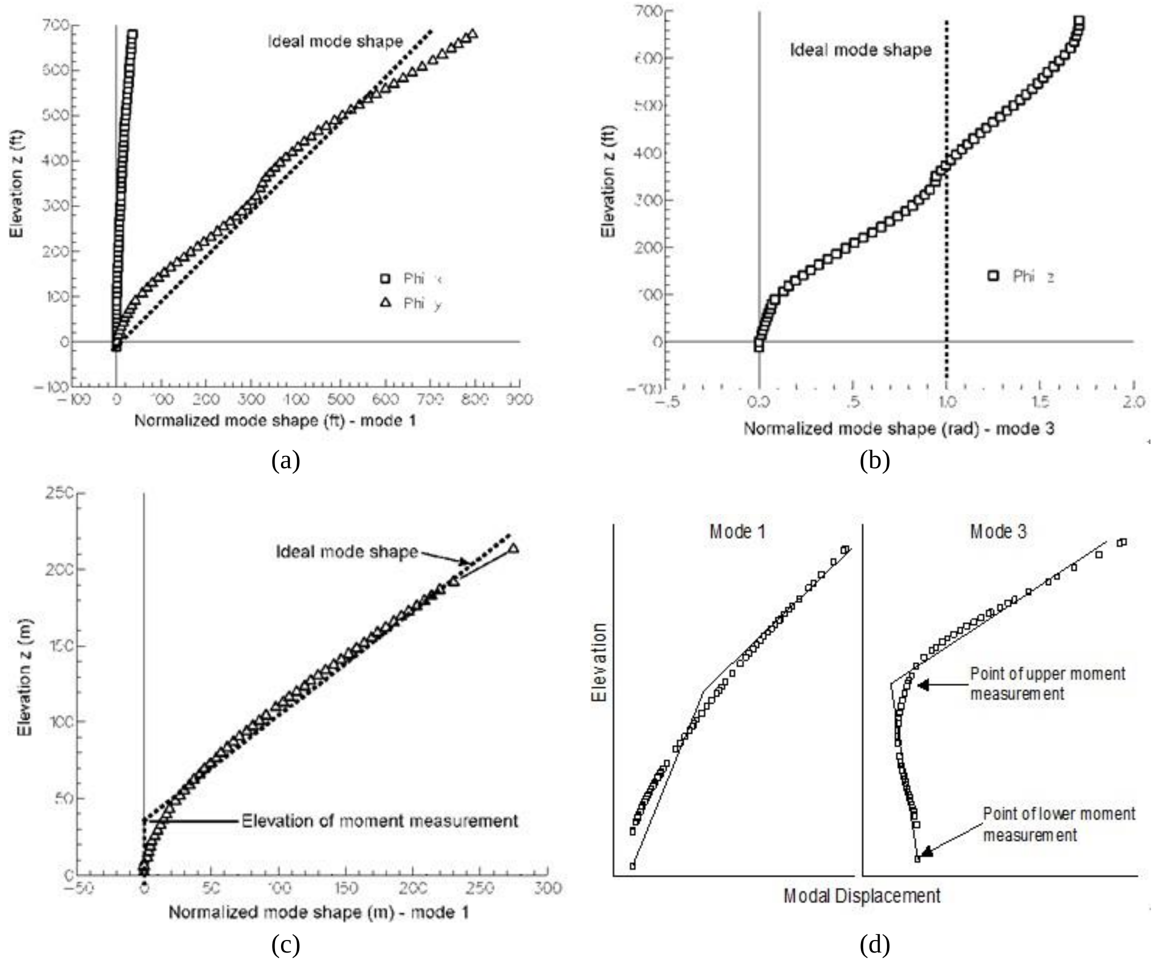
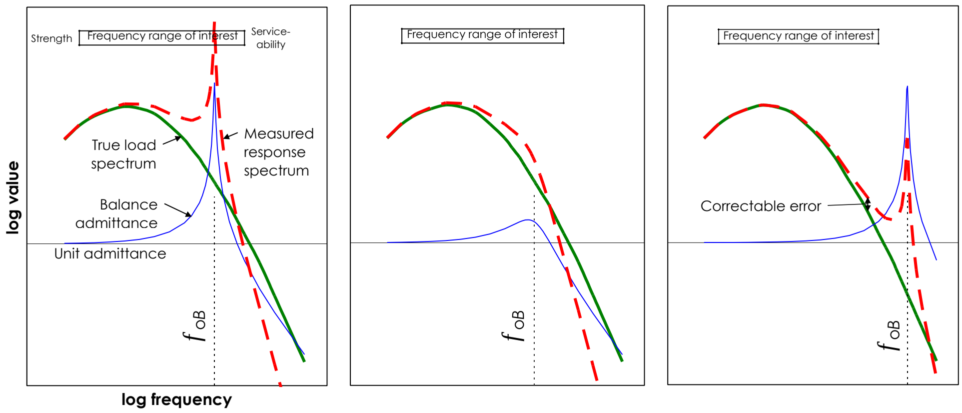


Fig. 2 Approximation of generalized modal excitation by "fitting" an ideal mode shape to the actual shape: (a) Sway mode shape fit by straight line through center of action of a moment balance; (b) Torsional mode shape has inherently poor fit; (c) Sway mode shape better approximated by moment at an elevated height, or equivalently, moment and shear at a fixed (typically below-grade) height; (d) Best approximation to sway shapes, even in higher modes, using a piecewise-linear shape

3.2 Instrumentation requirements

Exactly what is meant by "high frequency" in these designations? Although the data processing can be performed in either the time domain or the frequency domain, critical characteristics are best viewed in the frequency domain as illustrated in Fig. 3. The aerodynamic load spectrum is required to be measured over some "frequency range of interest (f_A," * of which the upper limit must exceed the structure's natural frequency, f_0 (usually in each of three fundamental modes), by a factor of at least 1.5, on the assumption that the biggest contributor (or at least a significant contributor) to the structure's response will be the resonance associated with aerodynamic excitation at the structure's lowest modal frequencies. To measure this with a balance requires consideration of the device's usable bandwidth, which is limited by its own resonant response to the very aerodynamic excitation it is trying to measure. If the balance damping and natural frequency, f_{0B}, are too low as in Fig. 3(a), the measured spectrum will be greatly distorted, rendering useless information in the upper part of the frequency range of interest. One way of combating this is to increase the balance damping, as in Fig. 3(b). However, the amount of damping required is very high and would be difficult to achieve and quantify, and corrections are still needed based on the exact value of damping present. Moreover, the transfer function of the balance is such to *diminish* the measured spectrum at frequencies higher than f_{0B}, making interpolation within the resonant region exceptionally difficult to quantify. The preferred solution, shown in Fig. 3(c), is to increase f_{0B} to some amount higher than the maximum frequency of interest. Corrections may still be necessary as indicated, but if f_{0B} is high enough (say 1.5 to 2.0 times f_A) these will be insensitive to the balance's damping and therefore easily performed. This is the essence of the "high-frequency" balance.



a) Low-frequency balance b) Overdamped balance c) High-frequency balance

Fig. 3 Frequency-domain relations between balance transfer function, measured aerodynamic excitation spectrum, and the actual excitation spectrum: (a) Actual spectrum distorted due to low damping and natural frequency; (b) Distortion reduced by increasing damping and (c) Distortion reduced by increasing natural frequency

* This discussion avoids the issue of scaling between frequencies at full scale and model scale: it should be understood that comparisons refer to both values transformed as necessary to the same scale.

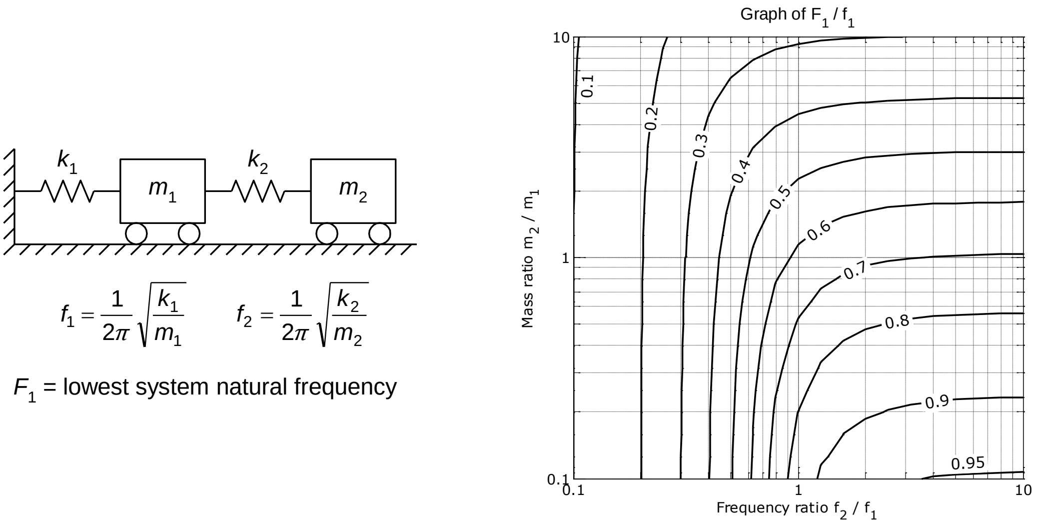


Fig. 4 Generalized 2DOF system analogous to a balance/model system, illustrating degradation of the usable system frequency depending on the frequency and mass ratios

Of course in the above we are talking about the model-scaled natural frequency of the building, and therefore the model-scaled frequency of interest, so the *x* axis in Fig. 3 is best viewed as reduced frequency, *fD*/*U*. In theory the reduced balance frequency can be made as high as desired merely by testing at a low model speed. However, this invariably leads to errors due to low Reynolds number and poor signal/noise ratio (S/N) as well as a longer required sampling time. By similar reasoning, the *required* frequency range of interest is higher when the full-scale speed is lower, as is needed for the determination of serviceability loads or acceleration response.

It is challenging to find an off-the-shelf commercially available balance of suitable performance. One must remember that the “high-frequency” requirement refers not to the balance itself, but rather the balance with building model mounted, and the entire assembly mounted on a turntable. The balance/model system can be well represented by the generalized 2DOF system depicted in Fig. 4, where m_1 and m_2 are the generalized masses (physically, the moments of inertia) of the balance and model, and f_1 and f_2 are natural frequencies for the balance alone and the model alone. The useful natural frequency of the system, F_1, is depicted graphically as a function of the mass ratio and frequency ratio. If the balance and model are of similar stiffness and mass—resulting in $f_1 \approx f_2$—then F_1 will be approximately 40 percent lower. To maximize the useful frequency (relative to the balance frequency), the model should have a higher natural frequency than the balance, but it is even more important that the mass of the model be lower than the balance so as not to overload it.

The balance must therefore have a demanding combination of rotational stiffness with high sensitivity—two requirements that are inherently at odds. Equally important is the model construction, which must be stiff yet lightweight. In critical applications this is best accomplished by increasing stiffness in the lower portion even if sacrificing the low-weight goal, transitioning to low weight in the upper portion even if sacrificing high stiffness. Certain modern architectural features can be troublesome, especially added complexity at the top of a model which necessitates the addition of “parasitic” weight having no contribution to stiffness. Occasionally a simple finite-element (beam-type) analysis is performed to investigate and optimize a construction method. In the author’s laboratory, tall building models are typically tested at a length scale of 1:300 to 1:500 and speeds of 8 to 10 m/s. A custom designed and built balance typically achieves balance/model natural frequencies of 100 – 150 Hz with good S/N.

Thus in general, the problem of model construction, which was largely neglected in the early days of HFB testing, has achieved an increasingly demanding role and this is likely to expand in the future.

It is also highly desirable to measure the aerodynamic spectrum with a sufficient degree of “smoothness” to allow the response to be computed with accuracy. Even with an excellent balance system, the measured spectrum consists of frequency points that are statistical estimates of the true value, and may have considerable scatter even among adjacent points. The mean-square error of each point estimate can be improved by careful specification of data acquisition parameters, utilizing such tools as additional data-segment averaging, frequency averaging of longer data segments, or improved numerical methods. The first two of these are most effective but require additional test time, and it is essentially impossible to circumvent the large amount of data that must be obtained. It may please the laboratory scheduling manger to realize a side benefit of a true high-frequency balance: if the bandwidth can be made say twice as high, then the test can be conducted at a speed twice as high, and *everything* happens twice as fast—meaning the data sample rate must be twice as high, and the required test time is cut in half.

Signal processing must also be carefully implemented to avoid contamination of the upper bandwidth region by aliasing of balance resonance signals. High-quality, sharp roll-off, analog low-pass filters are the traditional solution, but these are expensive and the technique of “over-sampling” can be a more cost-effective solution today.

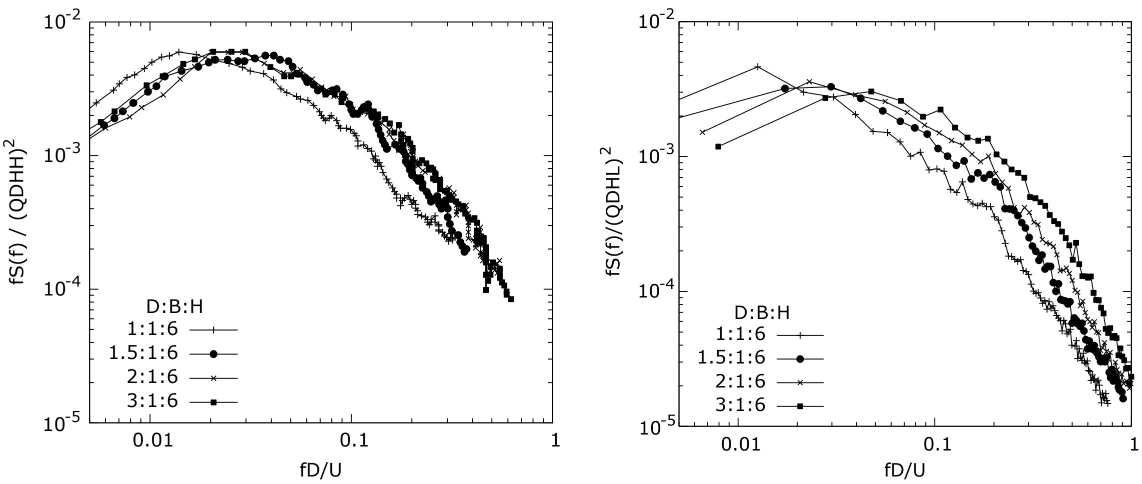


Fig. 5 Examples of measured load spectra in two different laboratories: alongwind component on prismatic shapes; D = crosswind width. (Exposure conditions are uniform but not identical.)

Samples of aerodynamic spectra measured in two different laboratories are shown in Fig. 5. Although subtle, some characteristics of the results on the left may indicate some shortcomings:

- The S/N ratio may be too low because of a low wind speed necessitated by a low natural frequency in the balance/model system (evidenced by general scatter of results).
- The S/N ratio may be low because the balance is inadequately isolated from vibrations in the turntable (evidenced by more general spectral patterns indicative of mechanical background—not aerodynamic—vibration).
- The bandwidth is limited at the high-frequency end because of limited balance/model system frequency (high-reduced-frequency excitation can become important for determining acceleration response for frequent low-speed winds).
- The signal was not properly filtered to prevent aliasing and/or resonant amplification (indicated in the 1:1:6 results where the decay slope for $fD/U > 0.2$ fails to decrease as expected).

All of the above must be carefully planned and executed to achieve a quality test.

3.3 Data processing

Once quality data is obtained in the laboratory it must be processed to obtain the structure's dynamic response—typically in the form of static-equivalent loads or top-floor acceleration. The common procedure is to associate each of the three measured orthogonal base moments to a fundamental vibration mode of the structure. This association is imperfect unless the mode shapes are "ideal" as described in Section 3.1. Of course this is not true in practice, especially in torsion. Nevertheless the procedure is capable of good results provided that corrections are developed for non-ideal mode shapes. This problem, which was one of the first major hurdles to be addressed in the history of HFB testing (Vickery *et al.* 1985, Boggs and Peterka 1987, 1989), is not reviewed here although its importance cannot be underestimated. This is especially true of today's tall buildings of structural complexity, which commonly feature three-dimensional mode shapes coupling the three degrees of freedom, and for which the generalized modal loads must be synthesized as a combination of the three measured moments. Research in this area continues, with perhaps the most interesting development being the so-called LMS (linear mode shape) method (Tse *et al.* 2009). In this technique, it is recognized that the displacement portion of any mode shape that includes significant torsion has a magnitude that depends on the reference location used for the center of twist. By selecting a suitable location the nonlinear portion can be made to vanish in terms of measurements, and mathematically restored in data processing. This method is not without some controversy and additional experience is needed to evaluate its integrity and usefulness. The method may have no benefit for the torsional component itself of the mode shapes, nor for simple symmetric buildings with no coupling.

Setting these complexities aside, the calculation of the structure's dynamic response is usually performed in the modal frequency domain

$$\tilde{\mathbf{P}}*^2 = \int_0^\infty |H(f)|^2 \cdot S_{P*}(f) \cdot df \tag{6}$$

where $S_{P*}(f)$ = spectrum of generalized load, $P*$ (including mode-shape correction), $|H(f)|^2$ = mechanical admittance of building, and $\tilde{\mathbf{P}}*^2$ = mean square generalized response including dynamic amplification.

This is easier said than done—or at least than can be done accurately. The lower and upper integration limits of zero and infinity do not exist in the measured spectrum, and suitable consideration must be made. More subtly, the damping of most structures is so low that most of the dynamic response occurs within a very small frequency range. If the damping ratio is x percent of critical, then fully one-half of the variance in response occurs due to frequencies within x percent of f_0 (Boggs 1991). If the damping ratio is say 0.01 and the measured spectrum is defined by 200 frequency points, then half of the variance—and 70 percent of the standard deviation—may be represented by as few as three consecutive points in the spectrum. If a simple integration scheme such as Simpson's rule is used, the calculated rms response can be in significant error.

One common means of circumventing the general problem of integration is to utilize the so-called white noise approximation, requiring the spectrum to be evaluated at a single point

$$\tilde{\mathbf{P}}^{*2} \approx \frac{\pi}{4\zeta} f_0 S_{P^*}(f_0) \tag{7}$$

Of course any error in that point will directly transfer to the result. Because this equation gives the exact response to a fictitious white noise spectrum having constant magnitude $S_{P^*}(f_0)$, and that value is generally less than the real spectrum at lower frequencies, most of the contribution from $S_{P^*}(f < f_0)$ is lost and the response tends to be underestimated. To compensate, the background-adjusted white noise approximation is often used:

$$\tilde{\mathbf{P}}^{*2} \approx \frac{\pi}{4\zeta} f_0 S_{P^*}(f_0) + \tilde{P}^{*2} \tag{8}$$

In most cases this produces an acceptable result. But in tall or slender buildings where f_0 may be near the vortex shedding frequency, Eq. (2) may actually overestimate the response, and Eq. (3) will overestimate it even more, as shown in Fig. 6 (Boggs 1991). For practical cases in which the reduced velocity is significantly less than critical (say < 8 in the case shown), Eq. (2) underestimates the response by 25 percent or more, and Eq. (3) underestimates it by 10 to 15 percent. Near the critical reduced velocity of 10.5, both equations overestimate the response by 10 percent. This case is of great concern in research applications, where such an error could be misinterpreted as negative aerodynamic damping.

Once $\tilde{\mathbf{P}}^*$ is found, the process of translating this to the corresponding dynamic base moment and floor forces is relatively straightforward (Boggs and Peterka 1987), and is not pursued here. Further, these must be combined with the mean response to obtain peak values. This is addressed through the concept of a peak factor, to be addressed below.

3.4 Aerodynamic damping

Perhaps the most important problem in the use of any aerodynamic method concerns the limitation due to inherent neglect of motion-induced or *aeroelastic* forces. These are recognized by experienced practitioners, perhaps not by others, and they are not well quantified by any. Well known is the effect on a square cylinder, shown in Fig. 7(a): the added aeroelastic response only comes into play as the reduced velocity U/f_0D exceeds the critical (vortex-shedding) value U/f_sD and is more serious for buildings of low density and damping (Boggs 1992). Hence it is usually neglected, often without justification. Less well known is that for other shapes, such as a circular cylinder or the triangular cylinder shown in Fig. 7(b), the aeroelastic amplification begins at much lower reduced velocity—and in fact is nearly symmetric about the critical velocity. Aeroelastic effects can sometimes be estimated utilizing the concept of aerodynamic damping, referring to documented aerodynamic damping coefficient curves (Vickery 1995), Fig. 8. Unfortunately such curves are limited to a small sample of regular prismatic shapes, response levels, and boundary layer conditions.

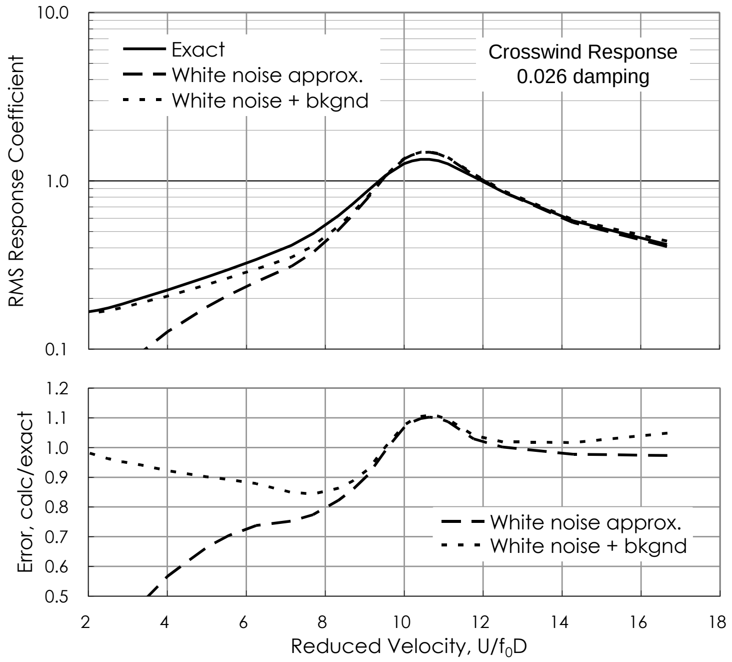
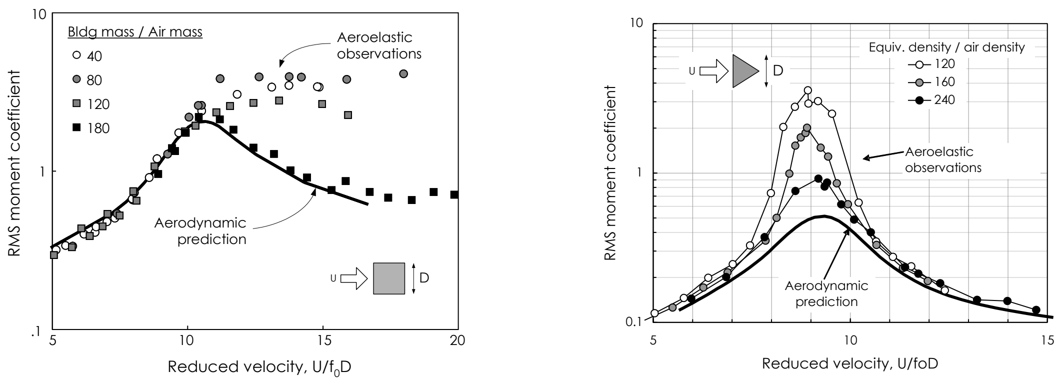


Fig. 6 Computed response as function of reduced velocity and error using approximate methods



(a) (b)

Fig. 7 Effect of motion-induced loads illustrated by aerodynamic-model prediction and aeroelastic-model observation. (a) Square cross section and (b) triangular cross section

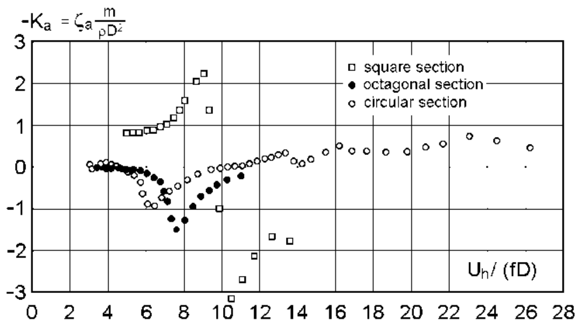


Fig. 8 Documented aerodynamic damping coefficients for simple prismatic shapes (Dyrbye and Hansen 1997)

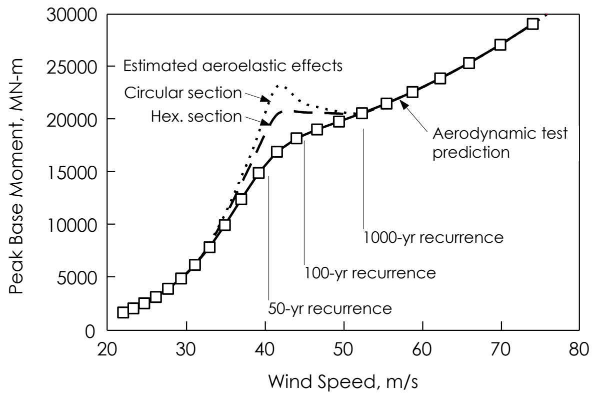


Fig. 9 Load-velocity prediction curve with “risk zone” due to aerodynamic damping

At best, these textbook curves can be used to establish approximate limitations, or “risk zones,” on the response-velocity curves provided by the wind tunnel laboratory to the structural engineer, Fig. 9. Such warnings may be sufficient to cause the engineer to increase the stiffness or density of the building, or (less likely) change the building shape. Aerodynamic damping information at present must be considered a rather primitive state, and improvement is highly desirable for the future of aerodynamic model testing. Until then, serious cases will only be resolved through an aeroelastic model test.

3.5 Load combinations and cases

Determination of the response (or static-equivalent) base moment, herein designated **M**, for any given wind speed U and direction α, is the essence of the HFB method but it is far from the required end result. What is needed is to specify loads such that all major parts of the structure are stressed to some known level of reliability. Some number of load combinations producing a desired mean recurrence interval must be determined. This is a general problem applicable to the wider class of aerodynamic testing, although some special considerations applicable to base balance testing are noteworthy because the loads must be derived from the information on **M**.

There are at least three methods to determine a design value from the **M**(U, α) information:

1. The "sector" approach: by suitable adjustment of the directional wind speeds, the largest **M** at any direction is selected.
2. The "total probability" approach: The desired value of **M** is computed mathematically as a weighted average of the **M**(U, α) results, usually via "upcrossing analysis."
3. The "time history" approach: the **M**(U, α) functional relations are not stated explicitly; rather, a historical record of (U, α) is applied to the basic test results to synthesize a pseudo-historical record of **M** for all significant storm events. The design value of **M** is then derived from extreme-value statistical analysis of these events. In other words, statistical analysis is performed on the load effect instead of the wind speed and direction.

All of these have their advantages and disadvantages, and are not discussed further here. However, it is noted that the sector method has an appealing quality in focusing on a fixed wind direction, which aids the physical interpretation of certain load combination issues.

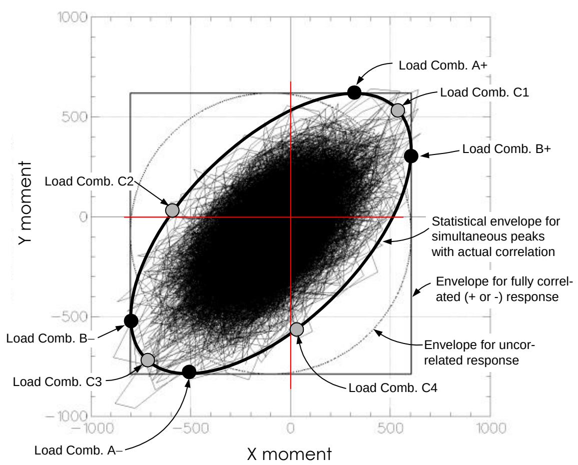


Fig. 10 The joint response and possible load combinations of two components at a fixed wind direction

At a given wind direction, it is of interest to investigate the simultaneity of actions in the three components $\mathbf{M}_x$, $\mathbf{M}_y$, $\mathbf{M}_z$. The joint time history of two of these is plotted in Fig. 10. The ellipse shown provides a reasonable envelope of probable extreme combinations. The ellipse can be based on the individual extreme values of each component to define the circumscribed rectangle, and on the correlation between components to define the skewness. This assumes that the correlation of the peak events is similar to the correlation for the general signal. Although any point on the ellipse could be interpreted as a possible design point, in practice this is limited to a few points of presumed special interest. Illustrated are four combinations (A+, A–, B+, B–) to include the extreme maximum and minimum value of each individual component, and four (C1 – C4) to include the maximum vector resultant having all possible combinations of positive and negative senses (i.e., the largest resultant in each quadrant). Among the first four, the extreme value of one component is designated the *principal* load, while the corresponding sub-extreme value of the other component is designated the *companion* load. Of course when all three components are considered, the number of combinations at each wind direction increases.

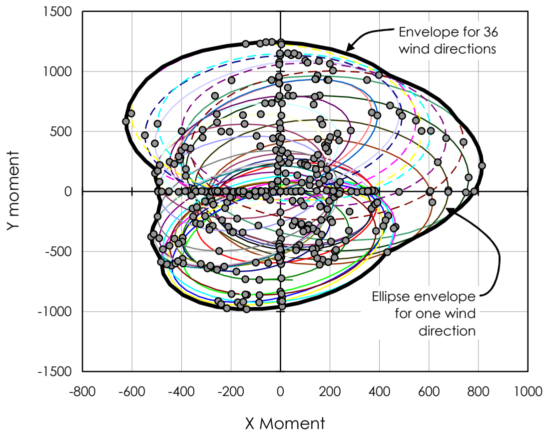


Fig. 11 The full picture of joint loads between two components at all wind directions

The "big picture" including all wind directions must now be made, as in Fig. 11. A "hyper-ellipse" enveloping all of the ellipses is easily visualized, but is it easily defined mathematically? Is it adequate to select the extreme values of the individual ellipse points, which all lie on the hyper ellipse, for design? In the example shown, it would appear that some key combination ranges may have been passed over, which may be of relevance to the structural engineer, such as (−500, 1000). It is difficult to quantify and certainly to automate these decisions. In most tall buildings the cases shown would be adequate, but it has been found otherwise, for example, in a building where the primary shear walls run diagonal to the established *x*-*y* coordinate directions. In such a case the principal-companion loads based on coordinate-system directions may underestimate the maximum resultant loads acting parallel to important structural elements.

Regardless of the method used, it is evident that care must be exercised in the definition of load combinations. It is only through collaboration with the design structural engineer that rational load cases can be specified. Improved methods of identifying and executing these specifications need to be developed.

Once the design **M** is defined, it remains to specify the distribution of corresponding forces and torques over the structure's height. Complications arise in combining the background and resonant actions at each floor level. This can be dealt with in various manners not addressed here, although the procedure described below in connection with extreme changes in shape is especially promising.

All of the above concerns load cases in terms of a single response index **M**. Carried to a logical extreme, load cases would ultimately be developed to obtain maximum actions such as axial and shear forces and bending moments in all structural members, or at least selected members deemed critical by the design engineer. In practice the added complexity of this is usually not pursued, and it is simply assumed that maximizing the overall moments will lead to a satisfactory structure.

4. Structural modeling issues

The issues discussed so far are highly analytical and mostly the responsibility of the wind tunnel. Yet, in the author’s experience, one of the most important yet time-consuming tasks is coordination with the structural engineer. It is surprisingly uncommon for design engineers of tall or medium-height buildings to have experience in structural dynamics, and if so it is often more applicable for seismic design than wind design. It is not unusual for the modal information provided by the design engineer to contain serious mistakes that are exposed by careful review of the wind-tunnel engineer. Several important and sometimes troublesome issues are discussed in this section.

4.1 Structural damping

The traditional rules of thumb—0.01 for a steel structure and 0.02 for a concrete structure—are being seriously (and rightfully) questioned, especially for modern design recurrence intervals ranging from “serviceability” (say 1 to 10 years) to “ultimate” (say 500 to 1000 years). At CPP we recommend nominal damping values, after asking the structural engineer if he wishes to specify an alternate value (and take responsibility for it). Responses range from “what is damping?” to “use 0.05”—the latter request coming from those schooled in seismic design, based on extreme ultimate conditions that generally don’t apply to wind design, and which results in our strong encouragement to consider a lower value.

4.2 Accuracy of predicted natural frequencies

Many simple formulas have been proposed over the years to estimate a tall building’s natural frequency, for preliminary design purposes, until a rigorous calculation can be made. Sometimes this rough approximation is the *only* estimation of natural frequency, e.g., when the engineer must choose which sections of a code design method are applicable depending on whether his building will be “flexible” or “rigid,” or when the wind-tunnel engineer is asked to perform a “desktop” prediction of the eventual loads. The formula most widely recognized today is

$$f_0\ (\mathrm{Hz}) \approx \frac{46}{H\ (\mathrm{m})} \approx \frac{150}{H\ (\mathrm{ft})} \tag{9}$$

which is apparently in reasonable agreement with many field measurements. However, we find it is not in good agreement with the predictions by structural engineers using generally accepted finite-element analyses. CPP is routinely given calculated frequencies that are only *half* of this prediction. Is this observed discrepancy due to field-measurement errors, to the need for a formula thought to be conservative for seismic application but in fact unconservative for wind purposes, to finite-element modeling inaccuracies and over-simplified modeling assumptions which may ignore the stiffening contribution of secondary members and other “non-structural elements,” or to a perceived need to include excessive mass and material cracking? All have merit, and standards are either nonexistent or controversial.

As consultants, it is not the position of the wind tunnel engineers to “out-guess” the design engineer or substitute our own estimate of the structure’s natural frequency; instead we are obligated to produce loads consistent with the modal properties provided, with at best a suggestion that the loads may be conservative due to calculation limitations. In addition, it would be undesirable for the design engineer, upon receipt of a final test report having loads consistent with his final and detailed calculation of modal properties, to find loads much larger than the preliminary design estimates that were based on a formula such as Eq. (9). So, this is an issue worthy of further research. Until then, it is appropriate for discussion between the wind tunnel and design engineers.

4.3 The units of mode shape

It is widely believed that mode shapes have no units. This belief stems from the notion that mode shapes are of indeterminate magnitude and can be scaled to any desired size. The scaling factor can include a dimension as well as a magnitude, so if one version of the shape definition included the dimension “meters” another could just as well have no dimension at all.

The fault in this argument occurs when multi-dimensional mode shapes include both translational and rotational (twist) components, because the same factor—including dimension—must be applied to all components. The dimension of either the translation (meters) or rotation (radians) can be eliminated, but not both: if the translation is rendered to be x, then the rotation would necessarily be of the form y rad/m. The significance is illustrated in Fig. 12. The two shapes shown, derived from the same numerical data but with different units, are obviously different: the twist angle is the same, but the accompanying displacements are 10 times larger when the units of cm are assumed. This affects the physical behavior of the building dynamics—the left depiction can be described as dominated by translation whereas the right is apparently dominated by twist.

It is unfortunate that mode shape data produced by most finite-element programs is devoid of unit labels, and that design engineers may not be well versed to the units issue. Sometimes this is the fault of the programs, as we know of one which performs all internal calculations in mm or inches, including the reporting of mode shapes, even if the user requested input and output in m or ft. Apparently the program authors are not familiar with this units issue either, or they assume that users are not concerned about it. Whose responsibility is it to identify errors and correctly interpret the meaning of numerical results—the design engineer or the wind tunnel engineer? Unfortunately, in practice it often becomes the wind tunnel engineer.

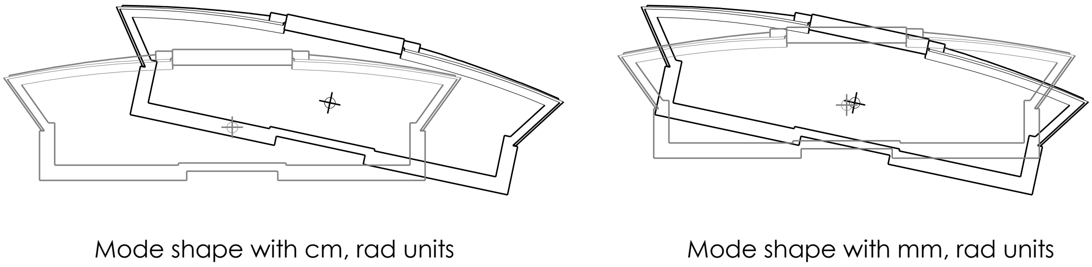


Fig. 12 The importance of units in mode shape

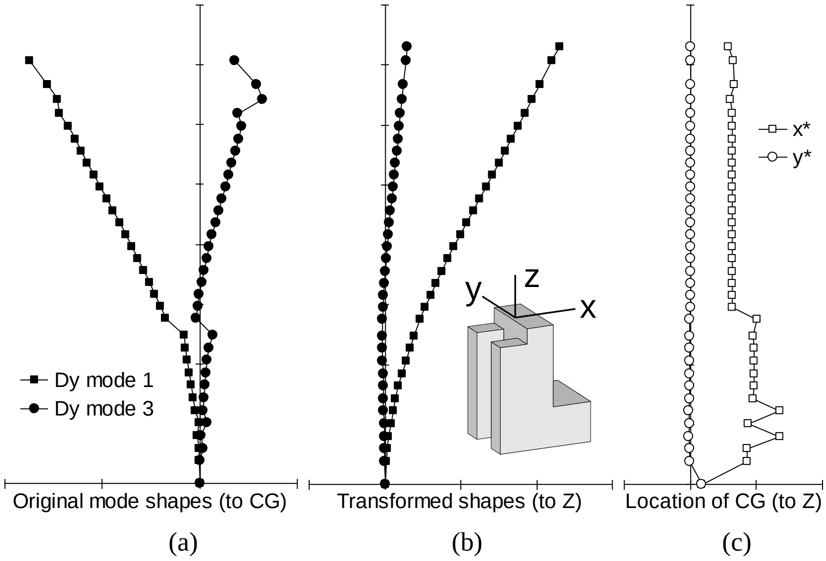


Fig. 13 Mode shape referenced to different axis systems

4.4 The mode shape reference system

Another potentially confusing aspect of mode shapes concerns the reference system used in their specification. Most commercial finite-element programs specify the components with respect to the center of mass at each floor. If the shape consists of coupled twist and displacement, then the displacement magnitude is dependent on the location of the reference origin. If the centers of mass do not align on a straight vertical axis—as in setbacks or shear wall drop-offs—then the displacements will contain offsets or “kinks” of the type illustrated in Fig. 13(a).

For use in HFB testing, it is imperative that the mode shapes be translated to the vertical *z* axis about which the twist component is measured. If done correctly, the kinks will vanish as in Fig. 13 (b)—unless, of course, the structure actually does contain an effective discontinuity.

It is essential, therefore, that the wind tunnel engineer knows the reference system used in the modal data received from the design engineer. As in the units problem described above, the confusion usually arises from an unfamiliar numerical analysis package, or when the design engineer extracts the data and provides it only in a generic table form. If the wind tunnel engineer *assumes* the shape data to be with respect to center of mass when if fact it was not, and with good intentions translates it to the balance axis, then kinks will be introduced where none existed originally. By examining the data carefully, the author has realized cases in which the design engineer even upon using a familiar program such as ETABS, noticed that the mode data had kinks and transformed them to a central axis, but without telling the wind-tunnel engineer this had been done or identifying the location of the translated axis. If mode shapes can be “accidentally” rendered nonlinear by the use of variable reference locations at different floors, then there must be *some* choice of locations for which nonlinearity disappears, rendering the *x*, *y* components of mode shape linear or ideally suited to the base balance technique. It was in fact this realization that led to development of the LMS method referred to above (Tse *et al.* 2009).

5. Architectural/structural issues

5.1 Lowrise and midrise buildings

The high-frequency balance method was originally developed for tall buildings, where wind response is dominated by resonance. Results were reasonably accurate, because mean, background, and resonant base moments could be obtained directly, and resonant floor forces calculated. The distribution of mean and background forces was uncertain, but the contribution of this to the complete load definition was relatively small. As test costs decreased and demand increased, the HFB method was increasingly used for medium height, and even lowrise buildings. The difficulty of obtaining accurate load definitions increased accordingly. Resonant loads, for which the aerodynamic model method excels, diminish relative to mean and background loads. A reasonable method of combining background and resonant floor forces became more critical. Mode shapes increasingly deviated from the ideal, and shape corrections became more important.

Even the peak factor, alluded to above, becomes more critical. When response is dominated by resonance at a natural frequency f_1, the peak load is easily and accurately estimated from the rms response using a relationship of the form (ignoring mode shape corrections)

$$\hat{\mathbf{M}} = \overline{M} + g_1 \tilde{\mathbf{M}}_1 \tag{10}$$

where $\tilde{\mathbf{M}}_1^2$ is the mean square response moment, derived from one of Eqs. (61)-(8), and

$$g_1 = \sqrt{2\ln(3600 f_1)} + \frac{.577}{\sqrt{2\ln(3600 f_1)}} \tag{11}$$

is a peak factor representing a 1-hour duration wind period. If the background response is significant, then (11) will overestimate the effective peak factor, yielding g_1 = 4 or more, because f_1 is higher than the effective cycling rate. Loads obtained from (10) are likely to be overly conservative. It is best to implement a more complex formulation of the effective peak factor, or an alternative to Eq. (10) such as

$$\hat{\mathbf{M}} = \overline{M} + \sqrt{(g_0 \tilde{M})^2 + (g_1 \tilde{\mathbf{M}}_{R,1})^2} \tag{12}$$

The peak factor for background loads, g_0, is usually taken as 3.4 to 3.5. The use of Eq. (12) is attractive because the contribution of additional modes is easily accommodated; however care must be exercised to distinguish $\tilde{\mathbf{M}}_{R,n}$, the resonant portion of the rms response due to mode *n*.

Another subtle complication with medium-rise buildings concerns the effect of roof pressure. The pressure distribution on a roof is nonuniform, and produces some contribution to the overturning moment. This is of course real, although generally ignored in design, and it does not contribute to a generalized modal load. There is no practical way to design a balance-model system that responds to the aerodynamic forces on walls but not on roofs. If a building is broad compared to its height, the loads derived from balance testing may be significantly overestimated. This can only be determined by comparison with loads derived from a pressure-tapped model test, in which the contribution of roof pressure to the overturning base moment can be easily distinguished.

For these and other reasons, the present trend is to use the multi-pressure type of aerodynamic model on such buildings instead of a balance. This trend will likely increase in the future, even though that method is not without other particular limitations.

5.2 Podiums and attached lowrises

Large podiums that extend well beyond the footprint of a supported tower, as in Fig. 14, may introduce special concerns. Lateral-resisting frames or shearwalls at the outer end of such appendages may be largely influenced by local wind gusts, sufficiently large to engulf the out-standing structure, while those under the tower are loaded almost entirely by the integrated gusts acting over the tower, or dynamic motions of the tower itself. If the connecting diaphragms have significant "shear lag" then the same load cases that were designed to fully stress all parts of the tower structure may not produce the same effect in all parts of the podium structure. In fact, it is possible that even the governing wind directions in remote portions of the podium are quite different from the governing directions on the tower. For example, the east-west loads in the tower of Fig. 14 could be due to the crosswind effect of north-south winds, while lowrise loads in the east-west direction are certainly more sensitive to east-west winds. The podium, to some degree, acts similarly to cladding elements, which are influenced by local flow effects, with their own gust or peak factors, and which the balance test method is incapable of measuring—at least when only one sting balance is used, located in the tower.

Ideally (from the wind tunnel viewpoint) the lowrise would be structurally isolated from the tower by movement joints, allowing wind loads on each to be considered independently. In fact the structural engineer may desire to design the lowrise using standard code procedures, and an instrumented model test may not be necessary. Complications arise when the lowrise is structurally attached. One approach is to "pretend" that they are isolated and obtain separate design load information on each portion—measuring or estimating the correlation between fluctuating load components, and establishing load combinations consisting of maximum principal loads in each portion in combination with point-in-time companion loads on the other portion. This of course results in a greater number of load cases.

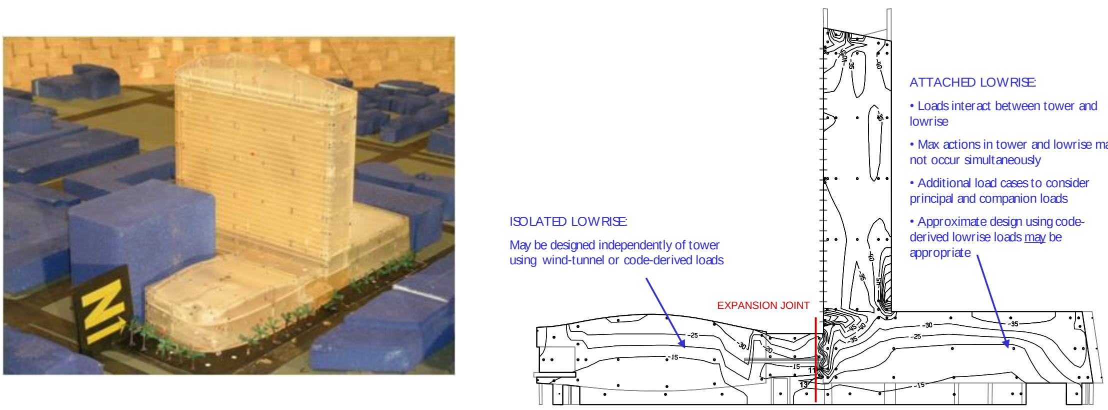


Fig. 14 Building featuring a tower and large podium. One lowrise is attached, one is isolated

If the lateral extent of an attached lowrise is minor, or is connected via extremely rigid diaphragms, then this sophistication may not be necessary. Unfortunately there are no guidelines to assist this determination. Since the implications are high in terms of test time, test cost, and analysis effort by the design engineer, further research on this subject is desirable—hopefully based on close cooperation between wind and structural engineers.

5.3 Extreme shape changes with height

In the formative years of HFB testing, tall buildings were prismatic and load case definition was relatively simple. However, a balance test could not provide design loads for *all* portions of the structure. For example, many large (broad if not tall) buildings feature small rooftop penthouses or appurtenances. Although loads on such "secondary structures" can be estimated, these may be inaccurate due to the influence of local flow phenomena—including gusts at directions independent of those affecting the overall building design—that cannot be specifically measured by a base balance, much like the attached lowrise discussed above.

Today's complex towers often feature similar extreme changes in shape at multiple elevations (Fig. 15) (Boggs and Hosoya 2001), that can no longer be viewed as simply a minor perturbation on the overall structural load. It is risky to specify design loads over the full height of the building based only on the maximum overturning moment. The critical loading on say the upper third of a tall building may be due to different aerodynamic phenomena than on the lower portions, as illustrated in Fig. 16. This depicts an illustrative building shape wherein the maximum drag coefficient varies with wind direction in three different height ranges. The alongwind quasi-static forces in floors 1 – 10 are largest when the wind direction is about 45 degrees; the critical wind direction for F11 – F20 is about 90 degrees, and the critical wind direction for F21 – F30 is about 315 (−45) degrees. The maximum accumulated shear force at any given floor is dependent on wind direction in an unclear manner, but an enveloping shear diagram can be constructed. Here for example the shear for levels 8 – 16 is controlled by a 90-degree wind. The corresponding design load diagram is then obtained by differentiating the shear diagram, i.e., the floor-to-floor differences in shear. A specification of this nature may be called a "pseudo load" because it will never exist in reality; it is a virtual condition designed to stress various portions of the structure to their maximum individual values that will not occur simultaneously.

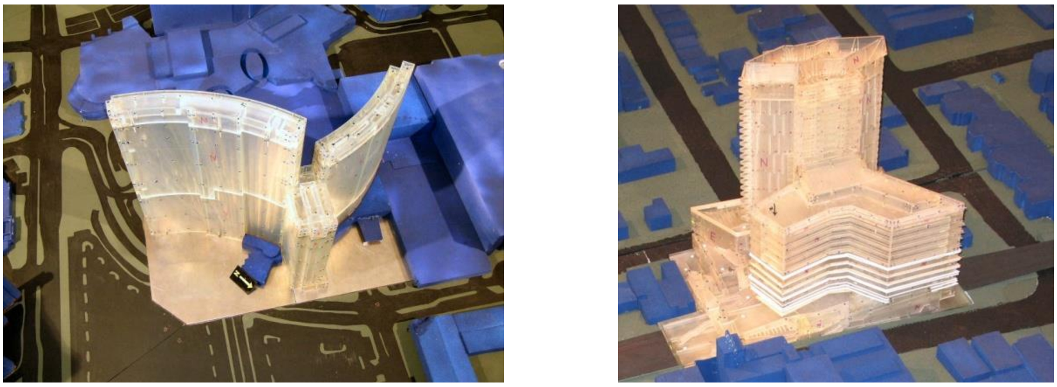


Fig. 15 Buildings with extreme shape changes causing critical wind directions to vary with height

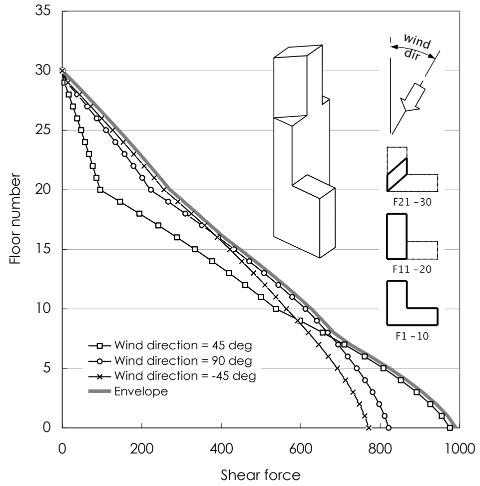


Fig. 16 Complications in load case definition when building features extreme shape changes with height

Fig. 17 Complexes with multiple structural linkages

When dynamic response is important, this effect is mitigated somewhat because a large part of the response load will be in the direction of a mode of vibration, which is relatively constant with height. Traditionally in fact, the wind response of tall buildings studied by the HFB method were so dominated by resonance that the effect was completely ignored. In the present, with the method being applied to lower buildings (perhaps not dominated by resonance) and with increasingly complex architecture, the effect merits renewed concern.

A procedure of this type is under study at CPP—and is routinely used for lowrise buildings—but is found to have practical difficulties:

- The method works well for multi-pressure aerodynamic studies. HFB studies are well equipped to obtain the maximum shear force at each floor due to resonance, but must be supplemented with additional information to obtain the mean and background shear distribution.
- The method works well for principal loads in primary components, but meaningful companion loads in secondary components can be cumbersome.
- The computation time is *much* greater (perhaps two or more orders of magnitude).
- The resulting pseudo loads are appropriate for strength design, but conservative for serviceability applications.

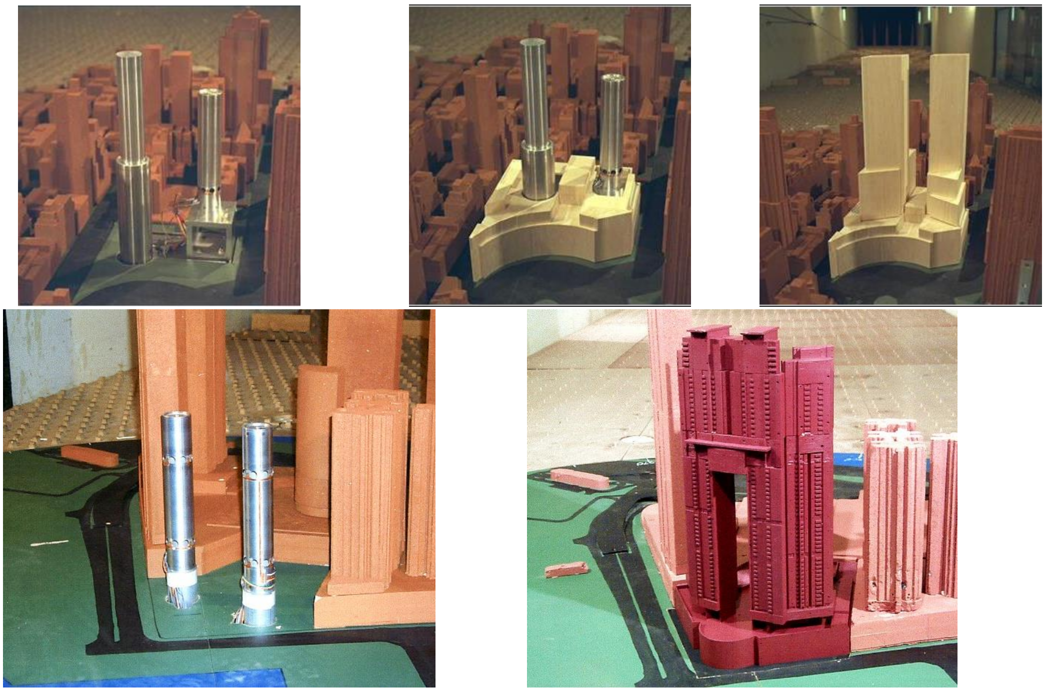


Fig. 18 Use of dual sting balances

5.4 Multiple linked towers

An extreme yet increasingly common complication is the existence of two or more towers that are structurally linked, Fig. 17. The linkage may range from minor skybridges to major structural continuity in upper sections or, more commonly, because they share a common podium without isolation joints. In general, the dynamic motions are coupled such that any motion in one tower induces some structural response in the other tower(s). Analysis of these phenomena require simultaneous measurement of the aerodynamic excitation on all towers, usually followed by the virtual synthesis of six or so generalized modal loads. A number of such studies have been performed (Xie and Irwin 1998, Boggs and Hosoya 2001, Xie and Irwin 2001, Lim and Bienkiewicz 2007, Rofail and Holmes 2007), using either multiple balances or synchronous pressure measurements, and are becoming increasingly common. The projects shown in Fig. 18 were performed with two “sting” tubes, instrumented to measure the moment at various heights to better fit the mode shapes. The second project shown was tested initially using balances at an early stage in the design process; once the tower shapes were finalized the testing was repeated using the simultaneous multi-pressure technique to obtain better mode shape correction factors and load combination definitions. Sophisticated testing of this type may or may not be necessary, depending on the stiffness of the structural linking, and improved guidelines should be developed.

6. Conclusions

The introduction of high-frequency balance testing, approximately 30 years ago, was designed to provide a relatively easy, inexpensive, and rapid assessment of wind loads on tall buildings of simple shape. The technique has since exploded in popularity, accompanied with the need for sophistication to address a variety of issues not originally conceived.

In the past, these issues included instrumentation to acquire quality data and data processing to deal with background and resonant response, non-ideal and three-dimensional coupled modes, and definition of basic load cases.

These issues were basically solved, yet remain current: new laboratories, to expand industry capability to meet demand, are implementing a new generation of equipment and expertise; developments in computer power, data acquisition, and digital signal processing can lead to better data quality; and pressure transducers and tubing systems are leading the multi-pressure technique to compete with balance systems—all for the advancement of the industry.

The high-frequency balance method is challenged by tall buildings of complex architecture and structural systems. In the past, we struggled to refine the technique to address these. At present it is often applied to tall buildings of almost any complexity. But this likely requires extensive manual effort, can be inefficient and costly, and the circumstances under which simplified approaches are acceptable are uncertain. In the future we must develop more general and automated methods to treat tall buildings of ever-increasing complexity, and better guidelines to steer us down the most appropriate and cost-effective methodology.

Most importantly, in the past, present *and* future, is the need for close communication between the wind tunnel and design engineers to work efficiently, understand each others’ needs, trade information in informative and useful forms, and (most of all) prevent errors.

References

Boggs, D.W. (1991), *Wind loading and response of tall structures using aerodynamic models*, Ph.D. Dissertation, Colorado State University.

Boggs, D.W. (1992), “Validation of the aerodynamic model method”, *J. Wind Eng. Ind. Aerod.*, **41-44**, 1011-1022.

Boggs, D.W. and Hosoya, N. (2001), “Wind-tunnel techniques to address structures with multiple coupled interactions”, *Proceedings of the Structures 2001 Structures Congress & Exposition*, (Ed. P.C. Chang), ASCE, Washington, D.C.

Boggs, D.W. and Peterka J.A. (1987), “The application of aerodynamic model tests to wind loading of tall buildings”, *Proceedings of the Dynamics of Structures, ASCE Structures Congress*, (Ed. J. Roesset).

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

115(3), 618-635.
Dyrbye, C.D. and Hansen, S.O. (1997), *Wind loads on structures*, John Wiley & Sons, Chichester, U.K.
Kareem, A. (1981), "Wind-induced torsional loads on structures", *Eng. Struct.*, **3**(2), 85 -86.
Kareem, A. (1982), "Acrosswind response of buildings", *J. Struct. Division - ASCE*, **108**(4), 869 - 887.
Lim, J. and Bienkiewicz, B. (2007), "Wind-induced response of structurally coupled twin tall buildings", *Wind Struct.*, **10**(4), 383-398.
Reinhold, T.A. (1983), "Distribution and correlation of dynamic wind loads", *J. Eng. Mech.- ASCE*, **109(**6), 1419 -1436.
Reinhold, T.A. and Sparks, P.R. (1979), "The influence of wind direction on the response of a square-section tall building", *Proceedings of the Wind Engineering 5th International Conference*, Ft. Collins, CO, USA, (Ed. J.E. Cermak), Pergamon Press.
Rofail, A. and Holmes, J. (2007), "High frequency base balance methodology for linked tall buildings", *Proceedings of the 12th International Conference on Wind Engineering*, Cairns, Australia.
Surry, D. and Stathopoulos, T. (1977), "An experimental approach to the economical measurement of spatially-averaged wind loads", *J. Wind Eng. Ind. Aerod.*, **2**(4), 385 -397.
Tschanz, T. and Davenport, A.G. (1983), "The base balance technique for the determination of dynamic wind loads", *J. Wind Eng. Ind. Aerod.*, **115**(3), 429-439.
Tse, K.T., Hitchcock, P.A. and Kwok, K.C.S. (2009), "Mode shape linearization for HFBB analysis of wind-excited complex tall buildings", *Eng. Struct.*, **31**(3), 675 -685.
Vickery, B.J. (1995), "The response of chimneys and tower-like structures to wind loading", *Proceedings of the 9th Int. Conf. Wind Engrg.*, New Delhi.
Vickery P.J., Steckley, A., Isyumov, N. and Vickery, B.J. (1985), "The effect of mode shape on the wind-induced response of tall buildings", *Proceedings of the 5th U.S. Nat. Conf. Wind Engrg.*, Texas Tech U.
Xie, J. and Irwin, P.A. (1998), "Application of the force balance technique to a building complex", *J. Wind Eng. Ind. Aerod.*, **77-78**, 579-590.
Xie, J. and Irwin, P.A. (2001), "Wind-induced response of a twin-tower structure", *Wind Struct.*, **4**(6), 495-504.