

A reliability approach for the wind-induced response assessment of tall buildings using the high frequency force balance

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ABSTRACT: The high frequency force balance is a versatile and widely used method for estimating the wind-induced response of tall buildings. Nevertheless, the estimates of the response parameters are inevitably affected by uncertainties essentially due to the lack of knowledge on the forces acting over the building height. Also, the usage the method is often hindered by the impossibility of considering more than three modes in the response assessment. In this paper a novel methodology is presented that allows to overcome the aforementioned shortcomings through the definition of a reliability framework in which the uncertainties arising from the use of the high frequency force balance are taken into account. The application to a case study building with coupled and non-linear modes shows the effectiveness of the method in providing safe estimates of the response parameters considering exclusively the base measurements.

KEYWORDS: Wind loading; Reliability models, Tall buildings, HFFB technique.

1 INTRODUCTION

The high frequency force balance (HFFB) is one of the most widely adopted techniques for the assessment of the wind-induced response of tall buildings, especially because the measurement of the base moments constitutes a relatively easy way to assess the overall action of the wind on the structure. Over the years, a number of methodologies have been proposed in the literature that are aimed at providing estimates of the structural responses commencing from the measured base moments. In most cases, this scope is attained through the identification of expressions that yield a set of correction coefficients that, applied to the three measured base moments, allow the estimation of the first three generalized forces [1, 2, 3, 4, 5]. These coefficients depend on the mode shapes of the building – therefore termed mode shape correction (MSC) coefficients – and on the pressure field acting over the building surface that, being unknown, has to be hypothesized. In the case of buildings with uncoupled fundamental modes it has been seen that these methodologies are able to provide good results in the estimation of structural response parameters [6, 7]. When the modes are coupled, in general, things become more involved and the estimates of the first three generalized forces and then of the response parameters are strongly affected by the assumptions made on the pressure field when estimating the MSC coefficients [6, 7]. It is worth underlining that, in any case, only the contribution of the fundamental modes is included in the response assessment: in general, while this is not a problem for the resonant response component, the background response can be affected by significant errors [8]. Indeed, it is common practice to use the HFFB technique only with the purpose of estimating the resonant response and not in order to obtain estimates of the background response. Different techniques that are not based on MSC coefficients, and are in general more complex from an applicative

point of view, have also been proposed [9, 10, 11]. In these cases, any number of modes can be included in the response calculation of the pressure field, but the necessity of hypothesizing the pressure field remains. Indeed, any method adopted for the assessment of the wind induced response starting from a set of base measurements is inevitably affected by what can be termed aerodynamic uncertainties [12], that is by the uncertainties generated by the lack of knowledge on the pressure field. These uncertainties propagate to the response parameter estimates, producing errors that can be significant [7], especially for tall buildings characterized by complex geometric shapes. As for the estimation of the mean response, when pressure measurements are unavailable the traditional approach consists in hypothesizing the distribution of the mean floor loads over the height of the building (linked to conventionally defined mean pressure profiles) and to calibrate it with the information at the base of the building [11]. In alternative, the method proposed in [13] exploits the definition of a power law model for the mean wind forces in order to define correction coefficients used to estimate mean values of the first three generalized forces. In any case, aerodynamic uncertainties result to be introduced also in the estimates of the mean responses.

A first step towards overcoming some of the aforementioned limitations of the HFFB technique has been proposed in [12], where the effect of the aerodynamic uncertainties on the resonant response was taken into consideration. In particular, a reliability framework was identified in which the aerodynamic uncertainties, and so their influence on the estimated response acceleration parameters, could be included.

The objective of this work is to further develop the approach proposed in [12]. This is done by firstly introducing a novel model for the probabilistic description of the mean. Secondly, a description of the correlation structure of the pressure field is carried out, which is more complete than the model presented in [12]. This allows the inclusion in the modal analysis of any number of modes, and therefore the probabilistic modeling of the total background response. To show the effectiveness of the proposed approach, a case study building characterized by a complex geometric form and non-linear coupled mode shapes is considered.

2 THE RELIABILITY FRAMEWORK

The aim of this section is to present the reliability framework in which the proposed approach for taking into account the aerodynamic uncertainties is set.

The fragility F for a generic response component R can be defined as the following conditional probability of exceedance of the response level $\tilde{R}$ [14]:

$$F = P(R_{max} > \tilde{R} | \mathbf{v}) \quad (1)$$

where $\mathbf{v} = \{U_H, \alpha\}$ is the vector of the hazard intensity with U_H the mean wind speed at the top of the building and α the wind direction while R_{max} is the peak value of the response process $R(t)$ for $t \in [0, T]$ where T is the observation period. The probability of exceedance $P_{f,R}$ can then be expressed in terms of the fragility as

$$P_{f,R} = \int_{D'} F(\mathbf{v}) \cdot p_{\mathbf{v}}(\mathbf{v}) d\mathbf{v} \quad (2)$$

where D' is the domain of all the possible hazard intensities while $p_{\mathbf{v}}$ is the probability density function of the hazard. Because $R(t)$ may be considered a stationary process, R_{max} with specified exceedance probability is given by:

$$R_{max} = \mu_R + g_R \sigma_R = \mu_R + \sqrt{g_{Rb}^2 \sigma_{Rb}^2 + g_{Rr}^2 \sigma_{Rr}^2} \quad (3)$$

where g_R is the peak value of the normalized (zero-mean and unit-variance) response process for $t \in [0, T]$, μ_R is the mean value, σ_R is the standard deviation while the subscript b and r indicate the background and resonant response component respectively. The estimates of μ_R and σ_R are traditionally considered unaffected by loading model uncertainties, resulting in a fragility only depending on g_R . Indeed, this is commonly done when choosing a deterministic model for the unknown pressure field over the building surface. On the other hand, if aerodynamic uncertainties are taken into account, the fragility can be re-written as:

$$F = 1 - \int_D p_{\mu g \sigma | \mathbf{v}}(\mu_R, g_R, \sigma_R) \cdot dD \quad (4)$$

where $p_{\mu g \sigma | \mathbf{v}}$ is the joint probability density function of μ_R , σ_R and g_R , conditional on the wind hazard intensity, and D is the domain where $R_{\text{peak}} < \bar{R}$.

If the random variables μ_R , g_R and σ_R are considered independent, then

$$p_{\mu g \sigma | \mathbf{v}}(\mu_R, g_R, \sigma_R) = p_{\mu | \mathbf{v}}(\mu_R) p_{g | \mathbf{v}}(g_R) p_{\sigma | \mathbf{v}}(\sigma_R) \quad (5)$$

Therefore, to calculate the fragility it is necessary to estimate the marginal distributions $p_{\mu R | \mathbf{v}}(\mu_R)$, $p_{g R | \mathbf{v}}(g_R)$ and $p_{\sigma R | \mathbf{v}}(\sigma_R)$. The PDF $p_{g R | \mathbf{v}}(g_R)$ has been widely studied; under the hypotheses of stationary and Gaussian process, it can be expressed by the following

$$p_{g R | \mathbf{v}}(g_R) = g_R \nu_0 T \exp \left[-1/2 g_R^2 - \nu_0 T \exp \left(-1/2 g_R^2 \right) \right] \quad (6)$$

where ν_0 is the mean zero-crossing rate, approximated by the first natural frequency of the building. Approaches for estimating the marginal distributions $p_{\mu R | \mathbf{v}}(\mu_R)$ and $p_{\sigma R | \mathbf{v}}(\sigma_R)$, are proposed in the following Section 3.

3 PROPOSED MODEL

3.1 General features

The aim of this section is to illustrate the proposed models for the propagation of the aerodynamic uncertainties in the floor loads to the structural mean and dynamic response. To reach this objective, models for the unknown floor loads (mean values and spectral structure) depending on a limited set of parameters are defined. Different values assumed by the parameters describe, although in an approximate fashion, all the possible floor load structures for every possible building and flow field. Secondly, the marginal PDFs and correlation structures for the random vectors collecting the parameters are assigned. If a large number of floor load structures would be known for buildings of different shapes and in different surrounding environments, the marginal PDFs of the governing parameters could be obtained by fitting the set of available data with the floor load model. Due to the insufficiency of this kind of data, the models for the statistical properties of the random parameters are only based on the knowledge of the physics of the problem, therefore it is of key importance that the governing parameters have a clear physical meaning.

3.2 Model for the mean response

With the aim of identifying the marginal PDF $p_{\mu R | \mathbf{v}}(\mu_R)$ introduced in Equation (5), in this section a probabilistic model of the unknown mean floor load distribution over the height of the building, calibrated through the five measured base reactions, is proposed. The basic idea is to use a Monte Carlo simulation to study the propagation of the uncertainties in the mean response parameters, μ_R , starting from the probabilistic description of a limited number of parameters that govern the mean floor load field.

3.2.1 Model for the along-wind direction

The floor loads in the along-wind direction, F_{Li} , $i = 1, \dots, n$, where n is the number of the floors, are expressed in terms of the mean wind speed profile $U_i = U(z_i)$ and of the force coefficients in the along-wind direction c_{Li} at different heights z_i , as follows:

$$F_{L,i} = 1/2 \rho U_i^2 c_{Li} h_i L_{ai} \quad (7)$$

where h_i is the height of the tributary area of the i th floor, L_{ai} is the width of the building orthogonal to the wind direction at height z_i and ρ is the air density. The mean wind speed profile indicated by the European Standards [15] is assumed. In this model, the vector of the random variables, $\{c_L\}$, contains the n values of the force coefficients, one for each floor:

$$\{c_L\} = \{c_{L1}, c_{L2}, \dots, c_{Ln}\} \quad \text{with} \quad c_{L,\min} \leq c_{Li} \leq c_{L,\max} \quad (8)$$

where $c_{L,\min}$ and $c_{L,\max}$ are the limits assigned to the force coefficients and can be fixed based on the knowledge of the physical phenomenon. The limits for the range of variability of the F_{Li} are consequently fixed and a range of possible values for the floor loads is obtained from Equations (7) and (8). Due to the lack of information on the actual value of the pressures over the building surface, a uniform distribution is adopted as marginal distribution for each random variable. As for the correlation structure of the random vector $\{c_L\}$, it can be observed that the degree of correlation among two force coefficients c_{Li} (at z_i) and c_{Lj} (at z_j) is in general very low if marked changes in the geometry of the building occur between the two floors at z_i and z_j . To account for this physical phenomenon, the floors of the building are grouped into n_g ideal sectors, based on the changes of geometry over the height of the building. Then, full correlation is assigned between all the force coefficients belonging to the same sector, while the coefficients of different sectors are considered uncorrelated. It should also be observed that the mean forces resulting from one realization of the random vector $\{c_L\}$ will not, in general, respect the base equilibrium conditions (shear and moment). To take this into account, only the randomly generated vectors that satisfy both the equilibrium equations (within a certain tolerance) are considered, while the others are rejected. As a result of this selection process, the marginal distributions of the c_{Li} are no longer uniform.

3.2.2 Model for the across-wind direction

For the across-wind direction the procedure adopted is substantially the same as for the along-wind direction. The floor load components, F_{Ci} , are defined in terms of the coefficients c_{Ci} , that can have positive or negative values. The following relation is adopted:

$$F_{C,i} = 1/2 \rho U_H^2 c_{Ci} h_i L'_{ai} \quad (9)$$

where L'_{ai} is the width of the building parallel to the wind direction at height z_i . If the geometry of the building is symmetric with respect to a vertical plane containing the oncoming flow direction, the mean force coefficients are expected to be zero over the height of the building. Nevertheless, asymmetries in the geometry can generate non-zero mean across-wind forces. For this reason, the limiting values of c_{Ci} are set as $c_{C,\max}$ and $c_{C,\min} = -c_{C,\max}$. Considerations analogous to the along-wind case hold (the division of the buildings in sectors and the respect of the base equilibrium conditions).

3.2.3 Model for the torsional component

Because the torsional loads are generated by the two translational forces when their action is eccentric with respect to the vertical axis of the reference system, for the torsional component the random variables are the normalized eccentricities of the floor loads e_{si}^n ($s = x, y$) defined as:

$$e_{si}^n = e_{si} / L_{si} \quad (10)$$

where e_{si} is the eccentricity of the floor load F_{si} while L_{si} is the width of the building at the i th floor (in direction perpendicular to the s direction). At each floor, the hypothesized torsional force $F_{\theta i}$ can be calculated from:

$$F_{\theta i} = F_{yi} e_{xi}^n L_{xi} - F_{xi} e_{yi}^n L_{yi} \quad (11)$$

where the forces F_{yi} and F_{xi} are obtained by simply projecting the F_{Li} and F_{Ci} , generated as illustrated in 3.2.1 and 3.2.2, on the reference system xy . By fixing the limits for the values of the normalized eccentricities $e_{s,max}^n$ and $e_{s,min}^n = -e_{s,max}^n$ a range of possible values is obtained for the random variables. The limits for the values of the vector of the hypothesized torsional forces result from both the limits on the eccentricities and the limits obtained for the along and across-wind floor loads. Akin to what was done for the along- and across-wind coefficients, the generated aleatory vectors $\{e_x^n\}$ and $\{e_y^n\}$ are considered acceptable if the base torsional equilibrium equation is respected, otherwise they are rejected.

3.3 Model for the dynamic response

In this section, an approach for estimating the PDF of a response parameter standard deviation σ_R taking into account the propagation of aerodynamic uncertainties is proposed. Similarly to other methods that can be found in the literature, the proposed approach is based on the assumption of a model for the floor load spectral structure. However the method developed herein has three distinct advantages that make it particularly attractive: 1) all the components of the floor load spectral matrix $[S_F]$ are modeled: this allows to take into account any number of modes in the response estimate, without approximations on their shape; 2) the calibration of the spectra is different for each wind direction; 3) the model depends on a limited number of parameters; varying the parameter values all the possible spectral structures of the floor load can be (approximately) described. In particular, the third characteristic allows the model to be used for the estimation, through a Monte Carlo simulation, of the PDF of the response parameter standard deviation. To do this, probabilistic models have to be defined for the parameters that govern the spectral structure of the floor loads.

3.3.1 Modeling of the floor load spectral structure

In the proposed approach, the floor load spectrum $S_{F_{spl}}$, i.e. the cross-spectrum between F_{sh} and F_{pl} ($s, p = x, y, \theta$ and $h, l = 1, \dots, n$), is modeled according to the following expression:

$$S_{F_{spl}}(f) = S_{F_{spHH}}(f) \left(\frac{z_h}{H} \right)^{\gamma_s} \left(\frac{z_l}{H} \right)^{\gamma_p} \exp \left(- \frac{k_{zsp} f |z_h - z_l|}{U_H} \right) \quad (12)$$

where $S_{F_{spHH}}$ is the cross-spectrum between the two top floor forces in direction s and p , γ_s and γ_p are the coefficients that govern the variation of the intensity of the spectrum of the components s and p , respectively, over the height, k_{zsp} is the decay factor in the vertical direction that governs the cross-correlation between two spectra belonging to the sub-matrix $[S_{F_{sp}}]$ acting at different heights. Equation (12) is the extension to the case $s \neq p$ of the expression proposed in [5] for the spectra $S_{F_{sshl}}$.

The floor load spectra defined according to equation (12) depend on the values of the parameters k_{zsp} , γ_s and γ_p and also on the value of $S_{F_{spHH}}$. In order to eliminate the dependency on $S_{F_{spHH}}$, it is hypothesized that $S_{F_{spHH}}$ can be obtained by scaling with a proportionality factor, $\alpha_{sp}(f)$, the measured base moment spectrum S_{Msp} (cross-spectrum between the base moments M_s and M_p).

The proportionality factor $\alpha_{sp}(f)$ can then be determined by imposing the equilibrium at the base of the building, once the values for γ_s and k_{zsp} are chosen. In particular, it is imposed that the spectrum resulting from the integration of the spectra $S_{F_{sphi}}$ over the building height coincides with the spectrum of the measured moments at the base $S_{M_{sp}}$. The following equations are consequently obtained:

$$\alpha_{sp} = \left[\sum_{h=1}^n \sum_{l=1}^n \left(\frac{z_h}{H} \right)^{\gamma_s} \left(\frac{z_l}{H} \right)^{\gamma_p} \exp \left(- \frac{k_{zsp} f |z_h - z_l|}{U_H} \right) \right]^{-1} \quad s, p = \theta \quad (13a)$$

$$\alpha_{sp} = \left[\sum_{h=1}^n \sum_{l=1}^n \left(\frac{z_h}{H} \right)^{\gamma_s} \left(\frac{z_l}{H} \right)^{\gamma_p} \exp \left(- \frac{k_{zsp} f |z_h - z_l|}{U_H} \right) z_h \right]^{-1} \quad s = x, y \quad p = \theta \quad (13b)$$

$$\alpha_{sp} = \left[\sum_{h=1}^n \sum_{l=1}^n \left(\frac{z_h}{H} \right)^{\gamma_s} \left(\frac{z_l}{H} \right)^{\gamma_p} \exp \left(- \frac{k_{zsp} f |z_h - z_l|}{U_H} \right) z_h z_l \right]^{-1} \quad s, p = x, y \quad (13c)$$

In this way, the dependency on $S_{F_{spHH}}$ is eliminated and all the spectra of the matrix $[S_F]$ can be modeled by fixing values for the 9 parameters γ_s and k_{zsp} .

3.3.2 Response prediction

The probabilistic approach illustrated in 3.3.1, being based on the modeling of the entire floor load spectral structure $[S_F]$, allows the assessment of any number of generalized force spectra as:

$$S_{Qij} = \{\Phi^i\}^T [S_F] \{\Phi^j\} \quad (14)$$

Therefore, any number of modes can be taken into account in the assessment of the structural response. Equation (14) will be used for the assessment of the resonant response, while to calculate the total background component of the response, R_b , the following expression will be adopted:

$$S_{Rb} = \{\Gamma_R\}^T [S_F] \{\Gamma_R\} \quad (15)$$

where S_{Rb} is the spectrum of the background response and $\{\Gamma_R\}$ is the vector of the influence coefficients of the response R .

3.3.3 Modeling of the parameters

In the proposed probabilistic model, the 9 parameters γ_s and k_{zsp} that govern the floor load spectrum model of Equation (12) are the components of a random vector whose marginal distribution and correlation structure have to be assigned. In this work, extending what proposed in [12], the marginal distributions of the governing parameters are obtained through the definition of analytical models describing the range of physically meaningful floor load correlation structures.

By observing that the value of k_{zss} is directly related, through the cross-power spectral densities $S_{F_{sshl}}$, to the level of correlation between the floor load F_{sh} (acting at z_h) and F_{sl} (at z_l), the distribution of k_{zss} can be chosen as to result in a uniform distribution of the correlation coefficient at a selected distance $D_{hl} = |z_h - z_l|$. The link between the correlation coefficient of the forces F_{sh} and F_{sl} , $r_{sshl} = \sigma_{sshl}^2 / \sigma_{sshh} \sigma_{ssll}$, and the parameter k_{zss} can be found using Equation (12). With some simplifications it is possible to obtain:

$$r_{sshl} = \sigma_{sshl}^2 / \sigma_{sshh} \sigma_{ssll} = \int_0^\infty S_{FssHH}(f) \exp(-k_{zss} f D_{hl} / U_H) df / \int_0^\infty S_{FssHH}(f) df \quad (16)$$

where σ_{sshh}^2 , σ_{ssll}^2 and σ_{sshl}^2 are the variances and covariances of the forces $F_{sh}(t)$ and $F_{sl}(t)$. Having stated this, the marginal distribution of k_{zss} can be chosen as to result in a uniform distribution of the correlation coefficient between the floor loads at a selected distance D . In Equation (16), S_{FssHH} can be obtained by imposing the equilibrium of the moments at the base of the building. Considering for instance $s = x$, the relation

$$S_{Myy}(f) = \int_0^\infty \int_0^\infty S_{FxxHH}(f) z_h z_l dz_h dz_l = \frac{S_{FxxHH}(f)}{H^{2\gamma_x}} \int_0^\infty \int_0^\infty z_h^{\gamma_x+1} z_l^{\gamma_x+1} \exp\left(\frac{-k_{zss} f |z_l - z_h|}{U_H}\right) dz_h dz_l \quad (17)$$

can be used to estimate S_{FxxHH} from the knowledge of the spectrum S_{Myy} . Analogous equations hold for the other components. It should be noted that the relation between the correlation coefficient and k_{zss} depends also on the value of γ_s , but this dependency is extremely weak and can therefore be neglected [17]. The choice of a uniform distribution for r_{sshl} ensures an impartial description of the unknown correlation between the forces, for which precise statistical information is extremely scarce. The proposed model for the marginal distribution of k_{zss} depends on D : it is suggested that this should be taken within the same order of magnitude of the average integral turbulence scales of the oncoming unsteady flow [12]. Considering the marginal distributions for the parameters k_{zsp} , with $s \neq p$, that govern the decay of the correlation between two floor loads acting in different directions, the simplest possibility is to assume their values as intermediate between k_{zss} and k_{zpp} . In this way, the k_{zsp} cease to be independent random variables. More in-depth studies on the nature of the cross-correlation between forces acting in different directions would be necessary in order to establish a more sophisticated model for these parameters.

Analogously to the k_{zsp} , the γ_s have a well-defined physical meaning: as their value passes from -1 to ∞ the energy of the floor loads progressively moves from the bottom to the top of the structure. Having said this, physically meaningful marginal distributions of the components γ_s can be obtained by imposing a suitable distribution of the value of the auto-spectrum S_{Fsshh} for each frequency (or of the floor load variance σ_{sshh}^2) at a fixed height $0 < z_d < H$. Exploiting indications in the literature concerning the most probable ranges of values for the floor load variances (normalized to the value at the top of the building), in [12] the authors suggested to assume for the floor load variance σ_{sshh}^2 a PDF with uniform value between $\sigma_{sshh}^2(\gamma_s = 0)$ and $\sigma_{sshh}^2(\gamma_s = 0.5)$, and decreasing values outside this region. This leads to a distribution for the parameter γ_s which is well approximated by a lognormal distribution with mode = 0 and standard deviation of the associated normal distribution $\sigma = 0.62$. Further details on this model can be found in [12].

4 APPLICATION OF THE PROPOSED APPROACH TO A CASE STUDY

4.1 Case study

The Bank of China building, Hong Kong, is considered as a case study (Fig. 1). The footprint is square with a side of 52 m and the top floor height is 286 m. The geometry is irregular, with coupled and non-linear mode shapes (Fig. 1c): this is particularly interesting for the proposed application due to the fact that the use of the HFFB technique in presence of irregular shape and mode coupling can determine significant errors in the estimates of the floor loads [6]. Specific wind tunnel tests were carried out [16] using 126 pressure taps measuring synchronously the surface pressure for various wind directions, ranging from 0° to 360° with 10° increments, with a wind speed of 16 m/s and a sampling frequency of 250 Hz. The model length scale was 1/500.

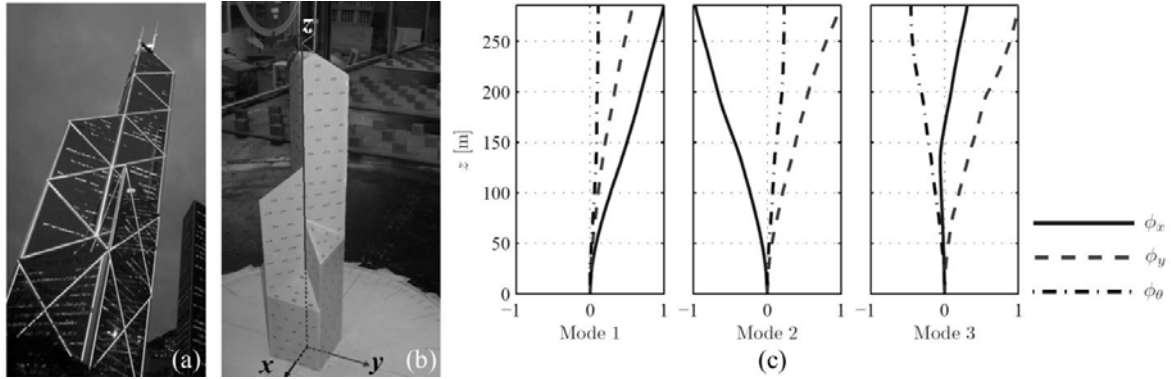


Figure 1. Bank of China building, Hong Kong (a); building model in the wind tunnel (b); mode shapes, with torsional component multiplied by 10 (c).

The pressures were then integrated to obtain virtual base moment measurements; the HFFB technique with the MSC scheme presented in [2] and the approach proposed in this paper were then applied to this virtual set of data to estimate the response parameters. The knowledge of the pressures allowed a reference solution to be determined, to which the approximated results obtained through the HFFB techniques can be compared. The response parameters which are taken into consideration are the top floor acceleration and displacement in the corner at $(x_c, y_c) = (-26 \text{ m}, 26 \text{ m})$. The results are referred to a wind speed of 55 m/s at prototype scale.

4.2 Results

For this application the random parameters γ_x , γ_y and γ_θ are assumed fully correlated. Also k_{zxx} , k_{zyy} and $k_{z\theta\theta}$ are assumed fully correlated, with zero correlation between the γ_s and the k_{zsp} . The following values have also been assumed: $D = H/8$, $c_{L,max} = 1.6$, $c_{L,min} = 1$, $c_{C,max} = -c_{C,min} = 0.8$.

Figure 2 shows a complete overview of the conditional PDFs obtained for the peak values of the response. It is seen that both their mean and their standard deviation vary strongly with the wind angle, according to the directional behavior of the building. For the acceleration (Fig. 2(b)), a bigger spread is seen in the values of the standard deviation of the density functions, than for the displacements (Fig. 2(a)). This can be ascribed to the fact that the resonant response marginal distributions tend to have strong variations for different wind directions.

It is interesting to compare the proposed approach to traditional methodologies (i.e. the MSC approach) and to the reference solution. Figure 3 shows, for $\alpha = 300^\circ$ and $U_H = 55 \text{ m/s}$, the value of the fragility calculated for different intensities of the peak response. The reference solution is obtained considering the reference (deterministic) value of the standard deviation and of the mean. The solution obtained with the “traditional approach” has been determined through the use of the MSC approach of [2] for the standard deviation and with the approach proposed in [13] for the mean. In all cases the PDF for the peak factor is obtained through Equation 8. The presence of the aerodynamic uncertainties in the proposed approach causes the exceedance probability curve to be softer than the others, where only the uncertainties due to the aleatory nature of the wind loads are taken into account. For different wind directions, while the curves obtained with the traditional approach are not always close to the reference curves, and often lie on the unsafe side, for the proposed approach there is always a value of the fragility which is sufficiently low to give a conservative value of the peak response parameter. This is better seen from Figure 4, where the values of the peak response parameters are plotted for all wind angles and $F = 10\%$.

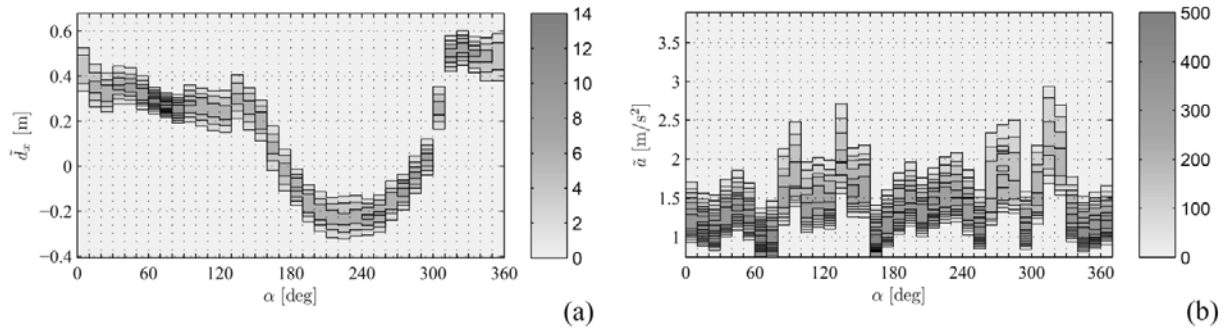


Figure 2. Estimated conditional PDFs of the peak values of the top floor x-displacement (a) and acceleration (b), for all wind angles and $U_H = 55$ m/s.

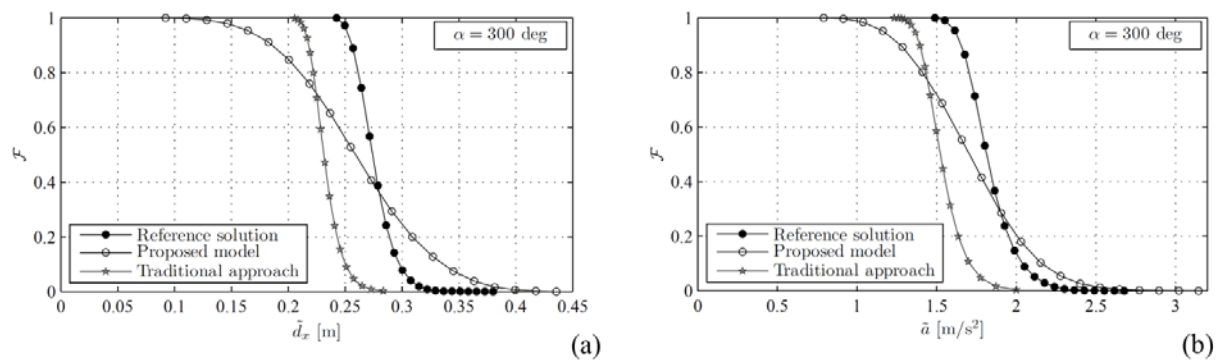


Figure 2. Fragility corresponding to different level of peak response for top floor corner x-displacement (a) and acceleration (b), for $\alpha = 300^\circ$ and $U_H = 55$ m/s.

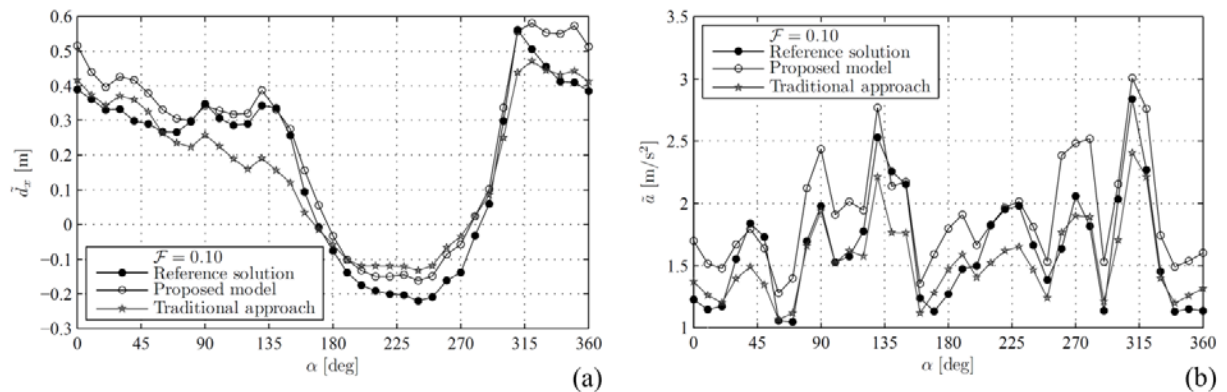


Figure 2. Value of the peak response of the top floor corner x-displacement (a) and acceleration (b) for all wind direction, $U_H = 55$ m/s and $\mathcal{F} = 0.10$.

5 CONCLUSIONS

In this paper a new method for the estimation of the dynamic response of tall buildings based on HFFB measurements is presented. The methodology is set within a performance-based design framework and attempts to overcome the two fundamental shortcomings of the HFFB method, that is the necessity of making assumptions on the unknown floor loads spectral structure (that introduce aerodynamic uncertainties in the estimated responses) and the impossibility of taking into account higher mode contributions and mean values in the response estimates.

A novel probabilistic approach has been proposed for the assessment of the mean response. Also, for the dynamic response, a new model for the floor load spectral structure has been presented, that gives the possibility of including any number of modal contributions. Both models depend on a limited number of physically meaningful random parameters. By probabilistically modeling the random vector collecting these parameters, the propagation of the aerodynamic uncertainties to the response was achieved. A procedure for the probabilistic characterization of the random vector was presented that, being based on the physical meaning of the governing parameters, results to be independent of the mechanical and aerodynamic performance of the building under consideration. A novel fragility model incorporates both the modeled aerodynamic uncertainties and the aleatory nature of the wind load in the estimation of the peak responses.

An application of the model to a case study building with non-linear coupled modes was presented in terms of the exceedance probabilities of peak responses. A comparison to the traditional approach was made demonstrating the importance of the inclusion of aerodynamic uncertainties in the peak response estimation and the effectiveness of the proposed approach in providing safe design values for the response parameters.

REFERENCES

1. P.J. Vickery, S.C. Steckley, N. Isyumov and B.J. Vickery, The effect of mode shape on the wind induced response of tall buildings, in Proc. 5th U.S. National Conf. on Wind Engineering, Lubbock, Texas, 1985.
2. J. Holmes, Mode shape corrections for dynamic response to wind, Eng. Struct., 9 (1987) 210-212.
3. D. Boggs and J.A. Peterka, Aerodynamic model tests of tall buildings, J. Eng. Mech., 115 (1989) 618-635.
4. Y.L. Xu and K.C.S. Kwok, Mode shape corrections for wind tunnel tests of tall buildings, Eng. Struct., 15 (1993) 387-392.
5. X. Chen and A. Kareem, Equivalent static wind loads on buildings: new model, J. Str. Eng., 130 (2004) 1425-1435.
6. S.M.J. Spence, E. Bernardini and M. Gioffrè, Influence of the wind load correlation on the estimation of the generalized forces for 3D coupled tall buildings, J. Wind Eng. Ind. Aerodyn., 99 (2011) 757-766.
7. E. Bernardini, S.M.J. Spence and M. Gioffrè, Wind load correlation and its effects on the second order response properties of 3D coupled tall buildings, in Proc. 11th National Conf. on Wind Engineering, Spoleto, Italy, 2010.
8. S.M.J. Spence, M. Gioffrè and V. Gusella, Influence of higher modes on the dynamic response of irregular and regular tall buildings, in Proc. 6th Int. Colloquium on Bluff Bodies Aerodynamics and Applications (BBAA IV), Milan, Italy, 2008.
9. T. Ohkuma, H. Marukawa, K. Yoshie, H. Niwa, T. Teramoto and H. Kitamura, Simulation method of simultaneous time-series of multi-local wind forces on tall buildings by using dynamic balance data, J. Wind Eng. Ind. Aerodyn., 54/55 (1995) 115-123.
10. D.Y.N. Yip and R.G.J. Flay, A new force balance data analysis method for wind response predictions of tall buildings, J. Wind Eng. Ind. Aerodyn., 54/55 (1995) 457-471.
11. G. Solari, L. Pagnini and G. Piccardo, A numerical algorithm for the aerodynamic identification of structures, J. Wind Eng. Ind. Aerodyn., 69/71 (1997) 719-730.
12. E. Bernardini, S.M.J. Spence and M. Gioffrè, Dynamic response estimation of tall buildings with 3D modes: a probabilistic approach to the high frequency force balance method, J. Wind Eng. Ind. Aerodyn., In press (2012)
13. X. Chen and A. Kareem, Dynamic wind effects on buildings with 3D coupled modes: Application of high frequency force balance measurements, J. Eng. Mech., 131 (2005) 1115-1125.
14. S.M.J. Spence and M. Gioffrè, Large scale reliability-based design optimization of wind excited tall buildings, Prob. Eng. Mech., 28 (2012) 206-215
15. EN-1991-1-4, Eurocode 1, 2005: Actions on structures, 2005.
16. S.M.J. Spence, Time domain non-Gaussian optimization of wind excited tall buildings under vulnerability constraints, University of Florence and TU Braunschweig: PhD Thesis, 2009.
17. E. Bernardini, Reliability models for tall buildings with coupled modes: a probabilistic approach to the high frequency force balance technique, University of Perugia: PhD Thesis, 2012.