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Analysis of three dimensional equivalent static wind loads of high-rise buildings based on wind tunnel tests

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ABSTRACT

The research on three dimensional equivalent static wind loads of high-rise buildings is an urgent topic for wind-resistant design. Based on synchronous surface pressure scanning of rigid model in wind tunnel test, the three dimensional wind load models of high-rise buildings are obtained. Furthermore, the random internal force responses of buildings in along-wind, across-wind and torsional directions are acquired by applying mode acceleration method, which express the restoring force solution in terms of quasi-static force item and inertia force item. Accordingly the calculation methods of equivalent static wind loads, in which the contributions of the higher mode can be considered, of high-rise buildings in along-wind, across-wind and torsional directions are deduced based on internal forces equivalence. Finally by analyzing the equivalent static wind loads of an actual high-rise building obtained by this method, and comparing with the along-wind equivalent static wind loads obtained by China National Standard, some conclusions, which have the reference value to wind-resistant design of high-rise buildings, are obtained.

1. INTRODUCTION

Wind effects on high-rise buildings are three-dimensional (Solari 1985, Liang 2002 2004), including wind loads in along-wind and across-wind directions, as well as torsional direction around the vertical axis. Therefore, wind induced structural responses shall be three-dimensional as well. In case of the structural wind-resistant

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design, the combined action of wind loads in the three directions must be considered. The current research on the equivalent static wind loads of high-rise buildings is mainly focused on the along-wind direction. In 1960s, Davenport (1978) proposed the along-wind equivalent wind load calculation method-----Gust Loading Factor (GLF). After that, the deficiencies of the method have been researched, and scholars from various countries have pushed forward the research on structural along-wind equivalent static wind load of high-rise buildings continuously. Solari(1990), Kasperski and Niemann(1992) have successively proposed the method of Load Response Correlation (LRC method) for the structural equivalent wind load calculation of low-rise buildings. China's Zhang proposed wind-induced inertial force method (GBJ method) to describe the structural along-wind equivalent static wind load and it was applied to the load code of China (GB50009-2001). Although the equivalence of GLF and GBJ is achieved through first order displacement response, there is a significant difference on their load distributions. Zhou and Gu etc (1999，2000) proposed the model of equivalent static wind load composing of the calculation of background component by LRC method and the calculation of resonance component by GBJ method, which makes great progress compared with the above-mentioned methods. Huang and Chen (2007) made researches on wind load effect and equivalent static wind load of high-rise buildings based on the surface pressure scanning of the wind tunnel test and analyzed the contributions of the high order modes to structure equivalent static wind load. Wang and Liang(2010) calculated the random internal force response of high-rise building structures under the fluctuating wind loads by the mode acceleration method, then established the equivalent static wind load model of high-rise buildings in along-wind direction based on the internal force equivalent principal, and further developed the calculation methods of equivalent static wind load of high-rise buildings. A large number of studies have shown that there are significant differences on the amplitude, distribution along the height and the corresponding internal force of the structural along-wind equivalent static wind load obtained by the various above-mentioned methods.

For the across-wind and torsional directions, Recommendations for Loads on Buildings, Architecture Institute of Japan (AIJ, 1996) recommended the calculation methods of equivalent static wind load and wind induced vibration response in across-wind and torsional direction of high-rise buildings; GU , QUAN and YE etc.(2004, 2006, 2006). also proposed the equivalent static wind load model in across-wind direction of super high-rise buildings, which is composed of background component calculated by LRC method and resonance components calculated by GBJ method.

Although the above literatures show that wind engineering circle has made extensive researches on the equivalent static wind load of high-rise buildings, the multiple existing calculation methods are not mature and fine enough. Along with the universal application of multipoint scanning surface pressure on rigid model in wind tunnel tests in Chinese wind engineering field, a fine equivalent static wind load calculation method which considers the three-dimensional wind-induced vibration effects should be established based on the surface wind pressure tunnel data for high-rise buildings of more than 200m in height. As the high-rise buildings have become higher and slender, their natural frequencies of the lower-order vibration modes become lower and lower, and the contributions of the second-order or even high-order vibration mode to the wind-induced dynamic responses cannot be neglected (Zou and Liang 2011). Studies show that when the structure and mass of high-rise buildings are symmetrically distributed around the central axis or the mutual eccentricity of the mass center, rigidity center and geometry center on each cross section of high-rise buildings is rather small, its wind-induced dynamic responses and the equivalent static wind loads in along-wind, across-wind and torsional directions can be treated as non-correlated, so that they can be calculated and analyzed separately (AIJ 1996, Liang wang 1991, 2005, 2009). When the mutual eccentricity of the mass center, rigidity center and geometry center on each cross section of high-rise buildings is rather higher (10% more than the width of the cross section), strong coupling shall occur among the wind-induced vibrations in along-wind, across-wind and torsional directions. At this time, its three-dimensional wind-induced dynamic responses and the equivalent static wind load must consider the contributions of three-dimensional coupling effects among along-wind, across-wind and torsional directions (Liang 1998, Tallin 1985, Islam 1992, Kareem 1992). Based on the mode acceleration method, this paper deduced the calculation method of internal force between floors of high-rise buildings under the actions of the three-dimensional wind loads, and proved that the statistical correlation of the three-dimensional fluctuating wind loads of high-rise buildings does not make contributions to the internal force between floors of the structure in each direction. The calculation method of equivalent static wind load in along-wind, across-wind and torsional directions, which can consider the contributions of high-order mode, is proposed based on the above mentioned work, and an engineering example shows, this method and its calculation results are valuable reference for the wind-resistant design of super high-rise buildings.

2. CACULATION OF EQUIVALENT WIND LOAD BY MODE ACCELERATION METHOD

2.1. Calculation of internal force response

It can be known from the theories of structural dynamics that the structural internal force response under the actions of the dynamic loads can be regarded as the results of negative restoring force. Then when wind acts along a main horizontal axis of high-rise buildings, and the coupling of the three-dimensional vibration of high-rise buildings does not occur (coincidence of structure mass center and rigidity center), the kinetic equation of the wind induced dynamic response in along-wind, across-wind and torsional directions of high-rise buildings can be expressed by the inter-floor model as follows:

$$\{F\} = [K]\{Y\} = \{P\} - [M]\{\ddot{Y}\} - [C]\{\dot{Y}\} \tag{1}$$

In which, $\{F\}$ is the negative restoring force or restoring moment vector in a principle horizontal axial or torsional direction of the structure; $\{Y\}$ is the structural horizontal displacement or angular displacement vector; $\{P\}$ is the random wind load vector in along-wind, across-wind and torsional directions; $-[M]\{\ddot{Y}\}$ is the inertia force or inertial moment of each floor, among which [M] is structure mass or moment of inertia matrix; $-[C]\{\dot{Y}\}$ is the damping force (or moment) of each floor, among which [C] is the damping matrix of each directions. The external load and the reacting force induced by the movement of each floor to the structure compose the negative restoring force, which causes structural distortion, and is also the direct reason for the generation of internal force responses.

Thus, the approximate solution of horizontal displacement and angular displacement obtained by mode acceleration method is as follows:

$$\{\tilde{y}(t)\} = [K]^{-1}\{P\} - [K]^{-1}[C]\sum_{i=1}^{j}\dot{y}_i(t)\{\phi\}_i - [K]^{-1}[M]\sum_{i=1}^{j}\ddot{y}_i(t)\{\phi\}_i \tag{2}$$

In which, $y_i(t)$ is the generalized coordinate of vibration mode i , and j is truncation vibration mode number; ω_i is the natural circular frequency of vibration mode i and $\{\phi\}_i$ is the mode shape of the i th mode.

As the damping of the high-rise building is generally rather small, the damping item can be neglected, i.e.:

$$\{\tilde{y}(t)\} = [K]^{-1}\{P\} - \sum_{i=1}^{j}\frac{1}{\omega_i^2}\ddot{y}_i(t)\{\phi\}_i \tag{3}$$

The first item on the right side of the equation is the quasi-static portion of the displacement, and is the displacement effect when the dynamic loads have not been dynamically amplified by the structure system; the quasi-static displacement is only related to $\{P(t)\}$ at the same time. As the natural frequency of the high-rise building structure generally changes quickly along with the mode order increases, $1/\omega_i^2$ in the second item on the right side of the equation improves the astringency. The elastic force or moment response by the mode truncation can also be obtained as follows:

$$\{\tilde{f}(t)\} = \{P(t)\} - [M]\sum_{i=1}^{j}\ddot{y}_i(t)\{\phi\}_i \tag{4}$$

where $\{f_B(t)\} = \{P(t)\}$ can be regarded as quasi-static item; $\{f_I(t)\} = [M]\sum_{i=1}^{j}\ddot{y}_i(t)\{\phi\}_i$ can be regarded as inertia force item. Then

$$\{\tilde{f}(t)\} = \{f_B(t)\} - \{f_I(t)\} \tag{5}$$

When the contribution of the multiple order modes is considered and j is the truncation vibration mode number, the obtained elastic force (moment) response of the multi-degree-of-freedom system can be expressed as:

$$\{\tilde{f}(t)\} = \{P(t)\} - \ddot{y}_1(t)[M]\{\phi\}_1 - \ldots - \ddot{y}_j(t)[M][\phi]_j \tag{6}$$

Without a loss of generality, for the linear high-rise structural system, any internal force response r_i of the section i is linearly related to elastic force (moment) on the mass (the moment of inertia) of Floor j. The response function s_j is to express the internal force response of Floor i caused by unit force (moment of couple) on Floor j. Based on the random vibration theory, it is deduced from Eq.(6) that the variance of any internal force responses of Floor i of high-rise buildings is as follows:

$$\sigma_{r_i}^2 = \sum_{j=i}^{N}\sum_{k=i}^{N}\sigma_f^2(j,k)s_j s_k \tag{7}$$

Due to the space limitations, the reference by Wang and Liang (2010) can be referred to for the specific deduction process of the Eq. (7). In the above equation, N is the total number of floors. $\sigma_f^2(j,k)$ is the covariance of the elastic force (moment) f_j and f_k on Floor j and Floor k of the structure respectively. As the natural frequency of each mode of high-rise buildings is separated from one another, the correlation between each vibration mode can be omitted. Meanwhile, the correlation of the wind load and the acceleration response of each mode is also rather small and can be neglected. Then the covariance matrix $\left[\sigma_f^2\right]$ of structural elastic force (moment) can be expressed as follows:

$$[\sigma^2{}_f] = \sigma^2{}_P + \sigma^2{}_I = \begin{bmatrix} \sigma^2{}_{P11} & \sigma^2{}_{P12} & \cdots & \sigma^2{}_{P1n} \\ \sigma^2{}_{P21} & \sigma^2{}_{P} & \cdots & \sigma^2{}_{P2n} \\ \cdots & \cdots & \cdots & \cdots \\ \sigma^2{}_{P2n} & \cdots & \cdots & \sigma^2{}_{Pnn} \end{bmatrix}$$
$$+\sigma_{\ddot{y}1}^2 \begin{bmatrix} m_1^2\varphi_{11}^2 & m_1m_2\varphi_{11}\varphi_{12} & \cdots & m_1m_2\varphi_{11}\varphi_{1n} \\ m_1m_2\varphi_{12}\varphi_{11} & m_2^2\varphi_{12}^2 & \cdots & m_1m_2\varphi_{12}\varphi_{1n} \\ \cdots & \cdots & \cdots & \cdots \\ m_nm_1\varphi_{1n}\varphi_{11} & \cdots & \cdots & m_n^2\varphi_{1n}^2 \end{bmatrix}$$
$$+\ldots+\sigma_{\ddot{y}j}^2 \begin{bmatrix} m_1^2\varphi_{21}^2 & m_1m_2\varphi_{21}\varphi_{22} & \cdots & m_1m_2\varphi_{21}\varphi_{2n} \\ m_1m_2\varphi_{22}\varphi_{21} & m_2^2\varphi_{22}^2 & \cdots & m_1m_2\varphi_{22}\varphi_{2n} \\ \cdots & \cdots & \cdots & \cdots \\ m_nm_1\varphi_{2n}\varphi_{21} & \cdots & \cdots & m_n^2\varphi_{2n}^2 \end{bmatrix} \tag{8}$$

In the above equation, σ_P^2 is the covariance matrix of the wind load in along-wind, across-wind and torsional directions respectively, which is the first item on the right side of the above equation; σ_I^2 is the covariance matrix of the structural inertia force, and corresponds to the sums of the items from the second item to the last item on the right side of the above equation. For the internal force responses concerned in the design, for example, the shear force or torque variance of Floor i can be expressed as follows:

$$\sigma_{Vi}^2 = \sum_{j=i}^{N}\sum_{k=i}^{N}\sigma_f^2(j,k) \tag{9}$$

The bending moment variance of Floor i can be expressed in the same manner as:

$$\sigma_{Mi}^2 = \sum_{j=i}^{N}\sum_{k=i}^{N}\sigma_f^2(j,k)(z_j - z_i)(z_k - z_i) \tag{10}$$

It can be proved that when the coupling of three-dimensional vibration of high-rise buildings does not occur, the statistical correlation among three-dimensional wind loads of high-rise buildings, i.e., wind loads of high-rise buildings in along-wind, across-wind and torsional directions does not make contributions to the internal force between the floors. According to the structural random vibration theories, the structural internal force response R（z，t）of the multi-degree-of-freedom system induced by the dynamic wind loads can be expressed as follows:

$$R(z,t) = \sum_{j=1}^{M} A_j(z) q_j(t) \tag{11}$$

in which, $A_j(z)$ is the internal force response function of Mode j; $q_j(t)$ is the jth generalized coordinates function. M is the number of vibration mode. $A_j(z)$ can be obtained by the following equation:

$$A_j(z) = \int_0^H m(z')\omega_j^2\phi_j(z')i(z,z')dz' \tag{12}$$

where H is the position of the last node in the vibration mode shape function, and is the position of the structure top for the floor model of high-rise buildings. $i(z,z')$ is the influnce function of the unit load, and represents the internal force at z caused by the unit load at z'. m(z') is the mass or moment of inertia at z'. After $A_j(z)$ is obtained, the following equation can be used to caculate the mean saquare of internal force:

$$\sigma^2(z)=\sum_{i=1}^{M}\sum_{j=1}^{M}A_i(z)A_j(z)\int_0^{\infty}H_i(-in)H_j(in)S_{ij}^*(n)dn \tag{13}$$

In the equation, $H_j(in)$ is the complex frequency response function of Mode j. $S_{ij}^*(n)=\frac{\varphi_i^{'}\ S_P(n)\ \varphi_j}{M_i^* M_j^*}$ is the cross spectral density function of Mode i and j. M_j^* is the generalized mass of Mode j. φ_j is 3N dimensional mode shape vector. When the coupling does not occur for the structure three-dimensional vibration, 2N values of the mode shape φ_j in the two directions in which vibration does not take place become zero. $S_P(n)$ is the spectral density matrix of the structural three-dimensional wind load. If the floor number of the high-rise building is N and the three-dimensional coupling vibration is considered, the vibration mode number of a multi-degree-of-freedom system for the floor model of high-rise buildings shall be M=3N. Due to the statistical correlation of three-dimensional fluctuating wind load of high-rise building, all sub-matrixes or part of the sub-matrixes located at the non- main diagonal in the three-dimensional wind load spectral density matrix $S_P(n)=\begin{bmatrix} S_{uu}(n) & S_{uv}(n) & S_{ut}(n) \\ S_{vu}(n) & S_{vv}(n) & S_{vt}(n) \\ S_{tu}(n) & S_{tv}(n) & S_{tt}(n) \end{bmatrix}$ are not zero generally. Then, whether the coupling of mode shapes in all directions occurs or not, σ_{ij}^2, the covariance of displacement response of Mode i and j is not zero generally, i.e. the integral of Eq.(13) is not zero. However, the internal force response function in Eq.(13) is different. Observing the internal force response function of Mode j, i.e. Eq.(12), if Eq.(13) is to acquire a horizontal principle axial internal force (shear or bending moment), both $A_i(z)$ and A_j (z) are not zero when the three-dimensional mode shape is coupled. When the mass center and rigidity center of the structure and aerodynamic center (i.e. geometry center) are coincident each other, mode shapes in every directions are not mutual coupled. The mode shape function degenerates from 3N dimensional vector into N dimensional vector (N is the number of floors). If $\varphi_i(z)$ is the translational mode shape in the same direction with the horizontal axial internal force obtained in Eq.(13) and $\varphi_j(z)$ is the torsional mode shape, $A_i(z)$ is not zero obviously and $A_j(z)$ must be zero. As the $i(z,z')$ in Eq.(12), the expression of $A_j(z)$, represents the required principle horizontal axial internal force at the floor z caused by the unit torsion (horizontal moment of couple) on any floor z', which is obviously zero. From this we can conclude that when the mass center, rigidity center and geometry center of each section of high-rise buildings are in coincidence, the statistical correlation of three-dimensional fluctuating wind loads of high-rise buildings does not make contributions to the internal force between floors in all directions, i.e. only the internal force contributions of background component induced by fluctuating wind loads and the corresponding resonance component induced by inertia forces in the same direction should be considered.

2.2. Calculation of internal force wind load

The following equation can be used to calculate the internal force equivalent wind loads of high-rise buildings in the two horizontal principal axial and torsional directions under all wind azimuths :

$$P_E(z) = \overline{P(z)} + \mu\sqrt{P_B^2(z) + P_I^2(z)} \tag{14}$$

in which, $\overline{P(z)}$ is the average wind load. When wind approaches along the horizontal principal axial direction of the symmetrical structure, the mean wind loads in the across-wind direction and torsional direction are zero. $P_B(z)$ is the equivalent wind load of background component, which can also be called quasi-static equivalent wind load; $P_I(z)$ is the equivalent wind load of the inertia force (moment) component; μ is the peak factor.

The following equation can be used to calculate the internal force response variance of Floor k caused by the background component:

$$\sigma_B^2(k) = \sum_{i=k}^{N}\sum_{j=k}^{N}\int_0^\infty S_p(i,j,n)dns(i)s(j) \tag{15}$$

In the equation, N is the number of the floors. $S_p(i,j,n)$ is a horizontal principal axial (or torsional) wind load cross spectrum of Floor i and Floor j. s(j) is the internal force response function of Floor k, which represents the internal force at Floor k caused by unit load applied to Floor j in the same direction as the internal force in. The following equation can be used to calculate the Floor k internal force caused by the background component equivalent wind load:

$$p_{\sigma B}(k) = \sum_{i=k}^{N} P_B(i)s(i) \tag{16}$$

in which, $P_B(i)$ is the background component equivalent wind load of Floor i. $p_{\sigma B}(k) = \sigma_B(k)$ is the mean square root of the internal force at Floor k caused by the background component. Through the simultaneousness of the above two equations, the background component equivalent wind load of each floor can be obtained.
The following equation can be used to calculate the internal force response variance of Floor k caused by inertia force (moment):

$$\sigma_I^2(k)=\sigma_{a1}^2[\sum_{i=k}^{N}m(i)\varphi_1(i)s(i)]^2+\sigma_{a2}^2[\sum_{i=k}^{N}m(i)\varphi_2(i)s(i)]^2+\ldots$$
$$+\sigma_{aN}^2[\sum_{i=k}^{N}m(i)\varphi_N(i)s(i)]^2 \quad (17)$$

in which, σ_{ai}^2 is the acceleration response variance of Mode i. The following equation can be used to calculate the internal force of Floor k caused by inertia force (moment) equivalent wind load:

$$p_{\sigma I}(k)=\sum_{i=k}^{N}P_I(i)s(i) \quad (18)$$

where $P_I(i)$ is the inertia force (moment) equivalent wind load of Floor i. $p_{\sigma I}(k)=\sigma_I(k)$ is the mean square root of the internal force at Floor k caused by the inertia force (moment). Through the simultaneousness of Eq.(17) and Eq.(18), the inertia force component equivalent wind load equivalent of each floor can be obtained. It should be noted that the mass of each floor in the above equations shall be changed into moment of inertia of each floor to the central vertical axis, and the linear displacement, linear velocity and linear acceleration along the main horizontal axis shall be changed into angular displacement, angular velocity and angular acceleration.

Suppose the structure of high-rise building is symmetrical, its mass center, rigidity center and geometrical center are in coincidence, and each floor slab of the building is rigid. Simplify the high-rise buildings into a multi-degree-of-freedom system with each floor center as the mass center. According to the frequency domain calculation method of random vibration theories, the acceleration response variance of Mode i in the two principal horizontal axial and torsional directions of high-rise buildings can be expressed as follows:

$$\sigma_{\ddot{k}_i}^2=\int_0^\infty S_{\ddot{k}_i}(n)dn \qquad \text{k=x, y, θ} \quad (19)$$

in which, $S_{\ddot{k}_i}(n)$ is the spectrum density function of Mode i of k axial acceleration (or angular acceleration), and can be expressed as :

$$S_{\ddot{k}_i}(n)=\frac{(2\pi n)^4|H_i(jn)|^2S_{F_i}^*(n)}{M_i^{*2}} \quad (20)$$

where $H_i(jn)$ is the complex frequency response function of Mode i in each axial direction of the structure. $M_i^*=\phi_i^{'}M\phi_i$ is the generalized mass of Mode i in each axial direction of the structure, $S^*_{F_i}(n) = \phi_i^{'} S_P(n)\phi_i$ is the generalized load spectrum density function of Mode i in each axial direction of the structure, where M and $S_P(n)$ are mass (or moment of inertia) matrix and load spectral density matrix in each axial directions of the structure.; ϕ_i is the ith mode shape in each axial direction.

3. CALCULATION EXAMPLE AND ANALYSIS

In this paper, the actual project of a super high-rise building to be built is taken as an example to study the structural three dimensional internal force equivalent wind load by the method presented in this paper. The super high-rise building is 250m in height, the main building is a rectangle chamfer cross-section construction, and the maximal width and depth of the cross-section are 53.85m and 45.05m respectively. This high-rise building is located in the flow field formed by the complex building groups, thus, the wind tunnel test must be carried out to determine the wind load of the structure.

3.1 Wind tunnel test

This test is carried out in HD-2 wind tunnel laboratory of Hunan University. The wind tunnel aerodynamic contour is 53m long, 18m wide, and it is a duplex testing section boundary layer wind tunnel. This test is carried out in the first test section. The length of this test section is 17m, the cross section of model test area is 3m wide and 2.5m high, and the wind speed of test section is 1 ~ 58m / s which is adjustable continuously. The diameter of its working turntable is 1.8m. Fig. 1 shows the test model in wind tunnel. It adopts the baffle plate, spires and cubic element in front of the test model to simulate the atmospheric boundary layer wind field. According to the local actual terrain, Class C landform in China's code is simulated. The geometry scale ratio of the wind field and the model are 1:200, and the main buildings within 300m surrounding this structure are simultaneously simulated. The wind profile and turbulence intensity of this simulated wind field are shown in Figs 2 and 3. The simulated wind speed spectrum is in good agreement with the Von Karman spectrum, which is shown in Fig. 4.

Fig.1 Model in wind tunnel

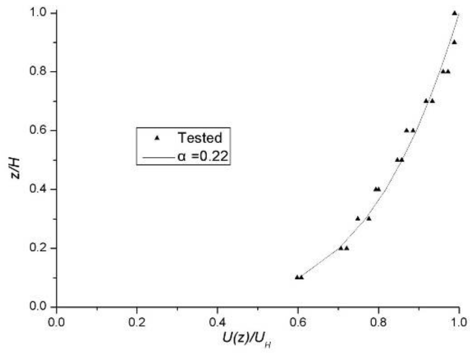
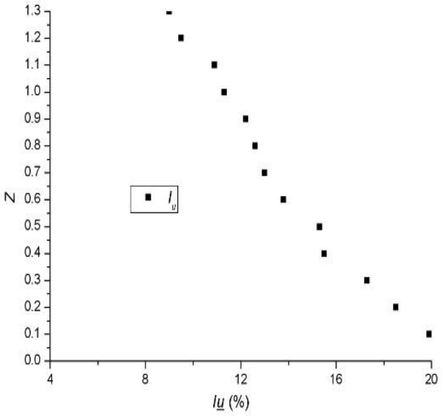


Fig. 2 Average wind speed profile

The cobra anemometer of Probe 100 series produced by Australia TFI Company is installed at 1.0m high in the left front of the model to measure the reference wind speed of wind tunnel tests. In order to measure the time histories of wind pressure throughout the model, 498 pressure transducers on the model surface are arranged in total, and the DTC net electronic pressure scanning valve system made by American PSI Scanning Valve Company is used to measure the model surface pressure. Eight scanning valves are used to scan the pressure signals of all measurement points. The sampling time of fluctuating pressure is 20s, the sampling frequency of each measuring point is 330Hz, and the test wind speed is approximately 10.8m / s. By rotating working turntable to simulate wind azimuths from 0 ° to 360 °, we have 24 testing wind azimuths with angle interval 15 ° in the test, as shown in Fig. 5.



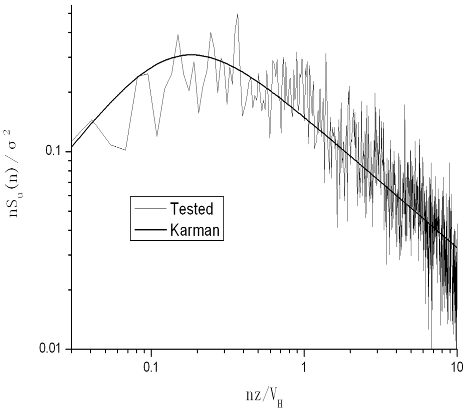


Fig. 3 Turbulence Intensity profile   Fig. 4 Normalized wind speed spectrum

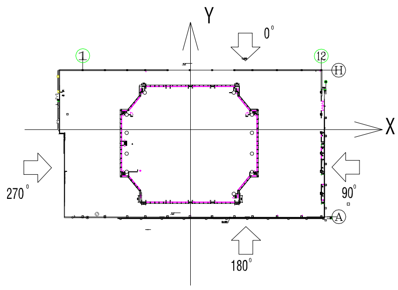


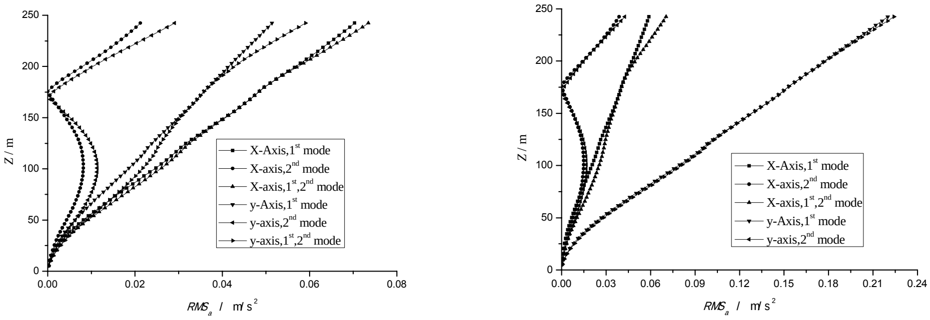
Fig. 5 Testing wind azimuths

3.2 Calculation and analysis of internal force equivalent wind load

After obtaining the time history of wind pressure on the structural surface from the wind tunnel test, the time history of structural three-dimensional wind loads and three-dimensional wind load spectra can be acquired by the method introduced by Liang, Zou and Guo(2009). Then, combining with the structural dynamic characteristics obtained from the finite element model of the structure, the internal force equivalent wind loads of the building are calculated by using the method introduced in this paper.

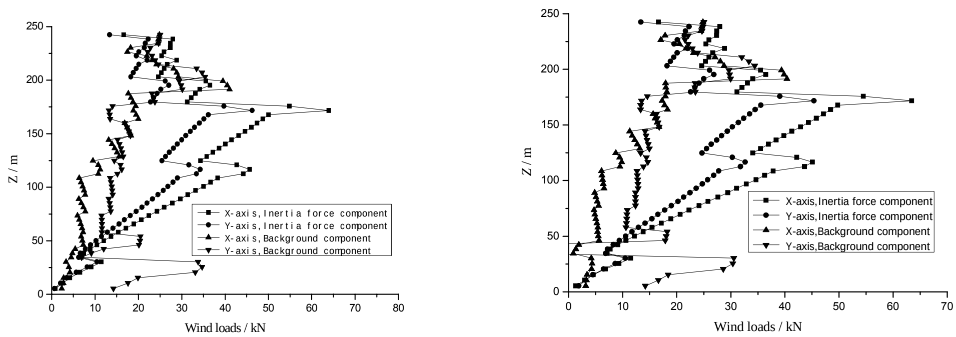
Fig. 6 shows the contribution to the structural acceleration from each vibration mode along X axis and Y axis under 0 ° wind azimuth (i.e. the wind is perpendicular to X axis, and the long side is windward). It can be seen from Fig.6, in the case of 0° wind azimuth, when considering the contribution of the second-mode of the structure, the RMS acceleration responses along X-axis (across-wind direction) and Y axis (along-wind direction) at the top of the structure increase by 5.0% and 15.7% respectively. In case of 90° wind azimuth, (i.e. the wind is perpendicular to Y axis, and the short side is windward), the contributions of the second mode of the structure along X axis and Y axis at the top of the structure are 18.6% and 1.8% respectively. Obviously, ignoring the contribution of the second mode would underestimate wind-induced vibration responses of the structure to a rather great extent. Similarly, during the calculation of structural equivalent wind loads of inertia force item, the obtained results will be inclined to danger if the contribution of the second vibration mode is neglected.

In order to study the characteristics of structural equivalent wind loads in case of 0° and 90° wind azimuths, each layer shearing force and bending moment equivalent wind loads along X axis and Y axis of the building and each layer torsional equivalent wind load are analyzed. Fig. 7 is the comparison diagram between the quasi-static force item equivalent wind loads and inertia force item equivalent wind loads along X axis and Y axis under 0°wind azimuth; Fig. 8 is the comparison diagram between the quasi-static force item equivalent wind loads and inertia force item equivalent wind loads in torsional direction under 0°wind azimuth; Fig. 9 is the deviation between the shearing force equivalent wind loads and bending moment equivalent wind loads under 0° wind azimuth.



（a）Wind azimuth of 0°　　（b）Wind azimuth of 90°

Fig. 6 RMS acceleration responses along X Axis and Y Axis



（a） Shearing force equivalence　　（b） Bending moment equivalence

Fig. 7 Comparison of Inertia force Item and quasi-static force Item equivalent wind loads along X Axis and Y Axis under 0° wind azimuth

It can be observed from Fig. 7 and 8, that the inertia force item of equivalent wind loads is slightly larger than the quasi-static force item of equivalent wind loads, but in the same order of magnitude. While carrying out the structural wind resistant design, only considering the contribution of the inertia force item and ignoring that of the quasi-static force item to structure will greatly underestimate the wind loads of the structure. As Fig. 9 shows, the difference between shearing force equivalent wind loads and bending moment equivalent wind loads is smaller, and the error is generally less than 5%; when carrying out the structural wind resistant design, either of these internal forces equivalent wind loads can be adopted.

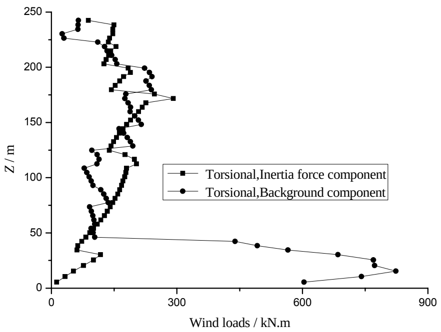


Fig. 8 Comparison of Inertia force Item and quasi-static force Item equivalent wind loads in torsional direction under 0° wind azimuth

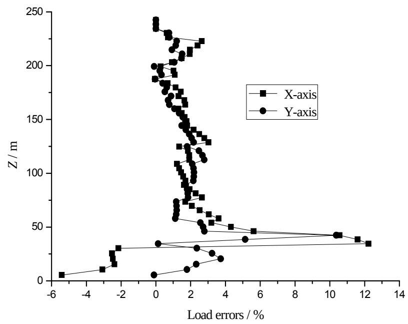


Fig. 9 Deviation between the shearing force and bending moment equivalent wind loads under 0° wind azimuth

The structural equivalent wind loads distributed along the height of the building are shown as Fig. 10-12. It can be observed from Figure 10-12, when wind direction is perpendicular to the principal axis, the equivalent wind loads of the building in along-wind, across-wind and torsional directions, all of these three loads cannot be neglected. When wind flow acts perpendicular to X axis (the long side is windward), its equivalent wind loads in along-wind direction is larger than the equivalent wind load in cross-wind direction, and when wind flow acts perpendicular to Y axis (short side is windward), the magnitudes of the equivalent wind loads in along-wind and in across-wind directions

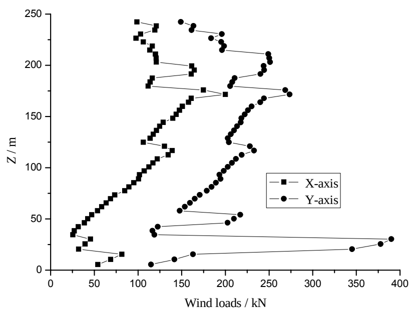


Fig. 10 Equivalent wind load along X Axis and Y Axis under 0° wind azimuth

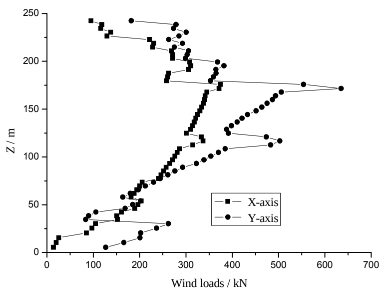


Fig. 11 Equivalent wind load along X Axis and Y Axis under 90° wind azimuth

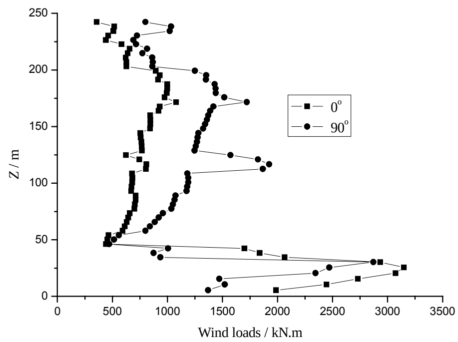
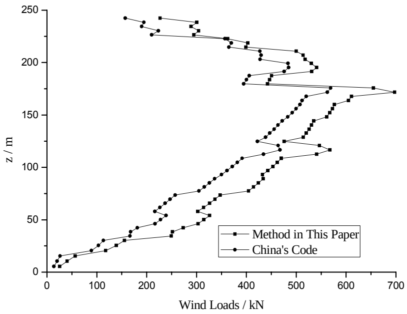
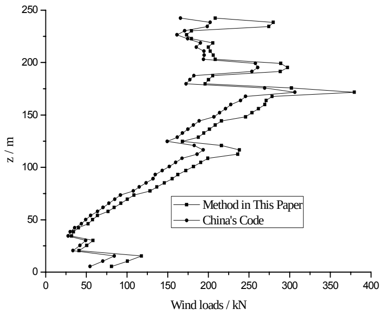


Fig. 12 Equivalent wind Load in torsional direction





(a) Wind azimuth of 0° (b) Wind azimuth of 90°

Fig. 13 Comparison of equivalent static wind loads in along-wind direction

differ from each other slightly. When wind direction is perpendicular to Y axis (the short side is windward), the equivalent wind loads in torsional direction is larger than those when wind direction is perpendicular to X axis (the long side is windward).
In order to investigate the equivalent wind loads in along-wind direction acquired by the method proposed by this paper and those by China's national code, the comparison of distribution of the equivalent wind loads in along-wind direction along the height of the building obtained by the method of this paper and that obtained by China's national code, when wind flow acts along 0° and 90°wind azimuths respectively, as shown in Fig. 13.

It can be obtained from Fig. 13 that no matter that the long side is windward or the short side is windward, the calculated equivalent wind loads in along-wind direction along the height of the building by the method of this paper are quite different from the results calculated by China National Standard. The results calculated by the method of this paper are larger and more discrete, and the main reasons for this fact are as follows:

1. The contribution to the equivalent wind loads in along-wind direction by the second mode of the building is not taken into account in China National Standard, which is an important reason to explain why the equivalent wind loads in along-wind direction for this example obtained by the method of this paper are larger.

2.In China National Standard, when the equivalent wind loads of a tall building in along-wind direction are evaluated by its formula, the interference effects induced by the surrounding buildings, the change of the mass distribution along the height of the building and the change of the along-wind force spectra along the height of the building and so on, such influence factors are not taken into consideration, and these factors can directly affect the distribution of the equivalent wind loads in along-wind direction along the height of the tall building. The results obtained by the method proposed in this paper have taken all the above mentioned factors into account, so the distribution of the equivalent wind loads in along-wind direction along the height of the tall building calculated by the method of this paper seems more discrete.

4. CONCLUSION

On the basis of the wind tunnel data, the three-dimensional equivalent wind loads of super high-rise buildings are investigated by applying the mode acceleration method, and the calculation method of the three-dimensional equivalent wind load of high-rise buildings based on internal force equivalence is provided in this paper. An actual super high-rise building is taken as an example to analyze the three-dimensional equivalent wind load of high-rise buildings. The main conclusions are as follows:

1. Based on the mode acceleration method, the wind-induced internal force response formula represented by elastic force solution is deduced. On the basis of the principle of internal forces equivalence, and considering the contributions of structural higher modes, the equivalent wind loads of high-rise buildings in along-wind, across-wind and torsional direction are acquired. The equivalent wind loads obtained by the method of this paper are directly based on the equivalence of each layer internal forces of a high-rise building, and are calculated according to the wind tunnel data of the building model as well as its scaled wind environment; thus they are the internal force equivalent wind loads with higher precision.

2. The equivalent wind loads of an actual super high-rise building are calculated, and the calculated results indicate that the equivalent wind loads of the super high-rise building in along-wind and across-wind directions are at the same order of magnitude, and its equivalent wind load in torsional direction also cannot be ignored. Therefore, the joint actions of three-dimensional wind loads on super high-rise buildings should be considered in the same time for their structural wind-resistant design.

3. For three-dimensional equivalent wind load of super high-rise structures, their quasi-static force item equivalent wind load and inertia force item equivalent wind load are at the same order of magnitude, if only adopting the inertia force item as structural equivalent wind loads will greatly underestimate the due wind loads on super high-rise structures, and accordingly, the structural design will be inclined to danger.

4. The comparison is conducted between the equivalent wind loads of an actual super high-rise building in along wind direction calculated by the method of this paper and

those obtained by China National Standard. The calculated results by the two methods are rather different, the magnitudes of the equivalent wind loads calculated by the method of this paper are larger than those obtained by China National Standard because the contributions of the second mode is taken into account in the former method, but the distribution tendencies of the two kinds of the equivalent wind loads along the height of the building are similar.

5. The established calculation method of three-dimensional equivalent static wind load for high-rise buildings in this paper is only applicable to the high-rise buildings with the symmetrical distribution of structure and mass to the central vertical axis of the building, or the eccentricity among mass center, elastic center and geometry center of each cross-section of the building are quite smaller. For the equivalent static wind load calculation of high-rise buildings with larger eccentricities among mass center, elastic center and geometry centers of each cross-section, the intense coupling effects among the three dimensional wind-induced vibrations of the building, i.e., wind-induced vibrations of the building in along wind, across wind and torsional directions, must be taken into consideration, which is an urgent research topic for further refinement of wind-resistant design for high-rise buildings, and also the next important research goal for the authors of this paper.

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