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A new framework for evaluating along-wind responses of a transmission tower

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Abstract: In this paper, an analytical framework to evaluate the along-wind-induced dynamic responses of a transmission tower is presented. Two analytical models and a new method are developed: (1) a higher mode generalized force spectrum (GFS) model of the transmission tower is deduced; (2) an analytical model that includes the contributions of the higher modes is further derived as a rational algebraic formula to estimate the structural displacement response; and (3) a new approach, applying load with displacement (ALD) instead of force, to solve the internal force of transmission tower is given. Unlike conventional methods, the ALD method can avoid calculating equivalent static wind loads (ESWLs). Finally, a transmission tower structure is used as a numerical example to verify the feasibility and accuracy of the ALD method.

Keywords: along-wind-induced dynamic responses; transmission tower; generalized force spectrum; equivalent static wind loads

1 Introduction

Lattice high-rise structures such as transmission towers are lightweight, very tall, flexible structures characterized by low damping and sensitivity to wind loads (Li and Bai, 2006). The wind load must be accurately calculated prior to structural design. The wind load can be qualified through multiple-point synchronous scanning of pressures on the surface of the structural model in a wind tunnel, or by high frequency fore balance (HFFB) measurement. However, unlike for ordinary buildings, the wind load on transmission towers is difficult or may be impossible to measure using multiple-point synchronous scanning of pressures due to its high hollowness rate, while the HFFB measurement is usually used to estimate the generalized force of the fundamental mode and fundamental mode generalized force spectrum (GFS). Then, according to random vibration theory, the variance of the responses including only the first mode can be further obtained. However, higher modes may provide noticeable contributions to the responses, especially for slender structures like transmission towers. Systematic studies on evaluating the along-wind responses of latticed towers were conducted by Holmes (1994; 1996a, b). To better understand the response of the transmission tower-line systems to wind, a novel approach for wind tunnel aero elastic modeling of conductors was introduced in detail (Loredo-Souza and Davenport, 2001; Loredo-Souz, 1996).

HFFB measurements on three semi-rigid tower models were performed in the TJ-1 boundary layer wind tunnel at Tongji University. The fundamental mode GFS of the transmission tower was obtained first, and then the higher mode GFS was expressed in analytical form (Zou, 2006). In order to obtain the structural internal forces, such as can be achieved with the conventional processing approach, the equivalent static wind loads (ESWLs) that include only the fundamental modal contribution of the transmission tower were also established (Yu, 2006). In this paper, a practical numerical higher mode GFS model is presented for applications in design practice. It was developed on the basis of the fundamental mode GFS obtained from a wind tunnel model experiment, and adopted the height-independent fluctuating wind power spectral density and Shiotami-Type spatial coherence function. Then, an analytical model for the displacement response is further deduced. In the formulation, the contributions of higher modes are included and the conversion relationship between the unilateral and bilateral power spectral densities is taken into account. Moreover, unlike the conventional method, the applying load with displacement (ALD) method, which will be described in the next section, is adopted to calculate the internal forces. The two models and the ALD method constitute a new practical analytical framework for use in the design of transmission towers.

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2 Practical analytical framework

2.1 Practical higher mode GFS model

The dynamic equilibrium equation of a structural system with a fixed base can be written as

$$\boldsymbol{M}\ddot{\boldsymbol{y}}(t)+\boldsymbol{C}\dot{\boldsymbol{y}}(t)+\boldsymbol{K}\boldsymbol{y}(t)=\boldsymbol{F}(z,t) \tag{1}$$

where $\boldsymbol{M}$, $\boldsymbol{C}$ and $\boldsymbol{K}$ denote the mass, damping and stiffness matrices; $\boldsymbol{y}(t)$, $\dot{\boldsymbol{y}}(t)$ and $\ddot{\boldsymbol{y}}(t)$ are the displacement, velocity and acceleration vectors, respectively; and $\boldsymbol{F}(z,t)$ is the external force vector. Then, the typical uncoupled modal equation for the structural system is expressed in the following form.

$$M_n\left[\ddot{q}_n(t)+2\xi_n\omega_n\dot{q}_n(t)+\omega_n^2q_n(t)\right]=F_n^{\mathrm{G}}(z,t) \tag{2}$$

in which

$$M_n=\boldsymbol{\Phi}_n^{\mathrm{T}}\boldsymbol{M}\boldsymbol{\Phi}_n$$

$$F_n^{\mathrm{G}}(z,t)=\boldsymbol{\Phi}_n^{\mathrm{T}}\boldsymbol{F}(z,t)$$

where M_n is the generalized mass; $\boldsymbol{\Phi}_n$ and $q_n(t)$ are the modal and generalized displacement vectors; ξ_n and ω_n represent the damping ratio and circular frequency, respectively; and $F_n^{\mathrm{G}}(z,t)$ is the generalized force, where the subscript (n) refers to the nth mode. According to the theory of quasi-stability, the nth modal generalized force is expressed as follows

$$F_n^{\mathrm{G}}(z,t)=\frac{1}{2}\rho C_{\mathrm{D}}\boldsymbol{\Phi}_n^{\mathrm{T}}\left\{B(z)d(z)R(z)U(z,t)^2\right\} \tag{3}$$

where ρ denotes the air density; C_{D} means the air damping coefficient; $B(z)$ denotes the calculated width and $d(z)$ implies the calculated height at altitude z; and $R(z)$ refers to the ratio between the actual area of unit height and contour area and

$$U(z,t)^2=\bar{U}(z)^2+2\bar{U}(z)\tilde{U}(z)+\tilde{U}(z)^2 \tag{4}$$

where $\bar{U}(z)$ and $\tilde{U}(z)$ are the mean wind speed and the fluctuating wind speed, respectively. Actually, compared with $\bar{U}(z)^2$ and $2\bar{U}(z)\tilde{U}(z)$, $\tilde{U}(z)^2$ is very small and can be neglected (American Society of Civil Engineers,1999). Here, the influence of $\bar{U}(z)$ is equivalent to static force, which does not need to be considered. Thus, substituting Eq. (4) into Eq. (3) yields

$$F_n^{\mathrm{G}}(z,t)=\rho C_{\mathrm{D}}\boldsymbol{\Phi}_n^{\mathrm{T}}\left\{\varGamma(z,t)\right\} \tag{5}$$

in which

$$\varGamma(z,t)=B(z)d(z)R(z)\bar{U}(z)\tilde{U}(z,t)=\kappa(z)\tilde{U}(z,t) \tag{6}$$

and

$$\kappa(z)=B(z)d(z)R(z)\bar{U}(z) \tag{7}$$

Strictly speaking, it is more likely that fluctuating wind speed spectrum density would vary with the height, z. However, according to criteria specified for engineering applications, the influence of z on the structural responses is very small and can be neglected (Zhang, 2006). Hence, according to random vibration theory and matrix theory, the power spectrum of the nth generalized force can be expressed as

$$S_n^{\mathrm{G}}(f)=\rho^2C_{\mathrm{D}}^2\boldsymbol{\Phi}_n^{\mathrm{T}}\boldsymbol{\Lambda}\boldsymbol{\Phi}_n \tag{8}$$

where

$$\boldsymbol{\Lambda}=\begin{bmatrix}\varDelta(z_1,z_1,f) & \cdots & \varDelta(z_1,z_i,f) & \cdots & \varDelta(z_1,z_H,f)\\ \vdots & \ddots & \vdots & \ddots & \vdots\\ \varDelta(z_i,z_1,f) & \cdots & \varDelta(z_i,z_i,f) & \cdots & \varDelta(z_i,z_H,f)\\ \vdots & \ddots & \vdots & \ddots & \vdots\\ \varDelta(z_H,z_1,f) & \cdots & \varDelta(z_H,z_i,f) & \cdots & \varDelta(z_H,z_H,f)\end{bmatrix}S(f) \tag{9}$$

$$\varDelta(z_i,z_j,f)=\rho(z_i,z_j,f)\kappa(z_i)\kappa(z_j) \tag{10}$$

where $S(f)$ represents the fluctuating wind speed spectrum density, which is independent of z, and $\rho(z_i, z_j, f)$ is the correlation coefficient between z_i and z_j. In theory, the correlation coefficient is a multivariant function, related to both position and frequency. Here, Eq.(11) suggested by Shiotami is adopted because it was established on the basis of actual testing and is applicable to high-rise structures (Zhang, 2006), and is independent of frequency, making it convenient to use.

$$\rho(z_i,z_j)=\exp\left(-\frac{\left|z_i-z_j\right|}{L_z}\right) \tag{11}$$

Substituting Eqs. (9), (10) and (11) into Eq. (8) yields

$$S_n^{\mathrm{G}}(f)=\rho^2C_{\mathrm{D}}^2\boldsymbol{\phi}_n^{\mathrm{T}}\boldsymbol{L}_{\mathrm{R}}\boldsymbol{\phi}_nS(f)\,,\qquad(n\geq1,n\in Z) \tag{12}$$

where $\boldsymbol{L}_{\mathrm{R}}$ is defined as the location correlation matrix; the element of which is expressed as follows

$$L_{\mathrm{R}}(i,j)=\rho(z_i,z_j)\kappa(z_i)\kappa(z_j) \tag{13}$$

From Eq.(12), the higher mode GFS expression can be obtained and expressed as

$$S_n^{\mathrm{G}}(f)=\frac{\boldsymbol{\Phi}_n^{\mathrm{T}}\boldsymbol{L}_{\mathrm{R}}\boldsymbol{\Phi}_n}{\boldsymbol{\Phi}_1^{\mathrm{T}}\boldsymbol{L}_{\mathrm{R}}\boldsymbol{\Phi}_1}S_1^{\mathrm{G}}(f)=\Theta_n S_1^{\mathrm{G}}(f)$$

$$(n\geq 1, n\in Z) \tag{14}$$

in which

$$S_1^{\mathrm{G}}(f)=\frac{a(fB/V_{\mathrm{H}})^{1.05}}{f[1+b(fB/V_{\mathrm{H}})^2]^{1.5}}\sigma_1^{\ 2} \tag{15}$$

$$\sigma_1=\frac{1}{2}C_{\mathrm{M}}\rho V_{\mathrm{H}}^2 AR=\sqrt{\int_0^\infty S_1^{\mathrm{G}}(f)\mathrm{d}f} \tag{16}$$

where Θ_n is defined as the generalized force spectral mode coefficient; $S_1^{\mathrm{G}}(f)$ refers to the fundamental mode GFS; A, B and σ_1 denote the contour area, bottom-width of the transmission in the direction of approach flow, and root mean square (RMS) of the fundamental mode generalized force, respectively; and a, b and C_{M} are the fitting parameters, which are, respectively, 14.6, 113.5 and 0.088 (Zou, 2006). Equation (14) shows that Θ_n establishes the constitutive relationship between $S_n^{\mathrm{G}}(f)$ and $S_1^{\mathrm{G}}(f)$, which conforms to the physical meaning. Compared with the analytical model in the form of Eq.(17), Eq.(14) is more practical and easier to adopt by researchers and/or practicing civil engineers.

$$S_n^{\mathrm{G}}(f)=\frac{\int_0^H\int_0^H \Omega_1\Omega_2\rho(z_i,z_j)\phi_n(z_1)\phi_n(z_2)\mathrm{d}z_1\mathrm{d}z_2}{\int_0^H\int_0^H \Omega_1\Omega_2\rho(z_i,z_j)\dfrac{z_1}{H}\dfrac{z_2}{H}\mathrm{d}z_1\mathrm{d}z_2}S_1^{\mathrm{G}}(f) \tag{17}$$

$$\Omega_1=\frac{1}{2}\rho C_{\mathrm{D}}S(z_1)\bar{V}(z_1)^2 \tag{18}$$

$$\Omega_2=\frac{1}{2}\rho C_{\mathrm{D}}S(z_2)\bar{V}(z_2)^2 \tag{19}$$

in which denotes the solid area at the height, z.

2.2 Multi-mode analytical model for displacement response

In this section, based on the fundamental mode GFS obtained from the wind-tunnel and the higer mode GFS deduced in section 2.1, a practical formula to evaluate the along-wind displacement response of a structure is developed.

According to the modal superposition method (Chopra, 2000), the displacement of a structure at different heights can be given by

$$y(z,t)=\sum_{n=1}^N \phi_n(z)q_n(t) \tag{20}$$

Accordingly, based on random vibration theory, the transfer function can be written in the following form

$$H_y(z,\mathrm{i}\omega)=\sum_{n=1}^N \phi_n(z)H_n(\mathrm{i}\omega) \tag{21}$$

$$H_n(\mathrm{i}\omega)=\left\{K_n\left[1+\mathrm{i}2\xi(\omega/\omega_n)-(\omega/\omega_n)^2\right]\right\}^{-1} \tag{22}$$

where $q_n(t)$ is the nth modal generalized displacement and $K_n=M_n\omega_n^2$ denotes the generalized stiffness of the nth mode. It is apparent that the physical phenomenon of structural vibration subjected to the wind load may be viewed as a multiple-input-multiple-output random process. Hence, the output power spectrum for the displacement can be written as follows

$$S_y(z,\omega)=\sum_{n=1}^N\phi_n^2(z)\left|H_n(\mathrm{i}\omega)\right|^2 R_n(\omega)+\sum_{\substack{k=1\\k\neq l}}^N\sum_{l=1}^N\phi_k(z)\phi_l(z)H_k(\mathrm{i}\omega)H_l(\mathrm{i}\omega)\gamma_{kl}\sqrt{R_k(\omega)R_l(\omega)} \tag{23}$$

where $R_{j(k,l)}(\omega)$ is the cross-power spectral density of the j (k, l)th mode, which is the bilateral power spectrum in the strict mathematical sense and will be replaced by the corresponding unilateral generalized force spectrum $S_n^{\mathrm{G}}(f)$ mentioned above; and γ_{kl} represents the correlation coefficient between the mth and nth mode and is dependent on both the frequency ratio and damping ratio, and also on the coherence of the generalized force spectrum. It can be estimated as follows (Chen and Kareem 2005a, b; 2007):

$$\gamma_{kl}=\rho_{kl}\iota_{kl} \tag{24}$$

in which

$$\rho_{kl}=\frac{8\sqrt{\xi_l\xi_k}\left(\lambda_{kl}\xi_l+\xi_l\right)\lambda_{kl}^{3/2}}{\left(1-\lambda_{kl}^{\ 2}\right)^2+4\xi_k\xi_l\lambda_{kl}\left(1+\lambda_{kl}^{\ 2}\right)+4\left(\xi_k^{\ 2}+\xi_l^{\ 2}\right)\lambda_{kl}^{\ 2}} \tag{25}$$

$$\iota_{kl}=\mathrm{Re}\left[R_{kl}(\omega)\right]/\sqrt{R_k(\omega)R_k(\omega)}\Big|_{\omega=\omega_m\mathrm{or}\omega_l} \tag{26}$$

in which $0<\lambda_{mn}=f_m/f_n<1$ and Re denotes the real part of the corresponding complex value.

Taking into account the low damping and discrete natural frequencies of the lattice tower, the cross-terms in Eq.(23) can be neglected. Therefore, the RMS value for the displacement response is given by

$$\sigma_y(z)=\sqrt{\int_{-\infty}^{+\infty}\mathrm{S}_y(z,\omega)\mathrm{d}\omega}=\sqrt{\sum_{n=1}^N\phi_n^2(z)\int_{-\infty}^{+\infty}\left|H_n(\mathrm{i}\omega)\right|^2 R_n(\omega)\mathrm{d}\omega} \tag{27}$$

$$\left|H_n(\mathrm{i}\omega)\right|^2=\left\{K_n^2\left[\left(1-\left(\omega/\omega_n\right)^2\right)^2+4\xi^2\left(\omega/\omega_n\right)^2\right]\right\}^{-1} \tag{28}$$

Phasing is adopted to transfer Eq.(27) into an algebra expression and full integration can be separated into three integrating ranges: (1) while $|H_n(\mathrm{i}\omega)|^2$ is over the resonant frequency range, ω_n-δ/2≤ω≤ω_n+δ/2, the co-vibration is significant and the dynamic response becomes prominent; (2) while $\omega<\omega_n$-δ/2, the response may be approximately considered as quasi-static since $|H_n(\mathrm{i}\omega)|^2$ is a narrow-band random process and decreases rapidly because of the low damping values (Yu, 2006; Zhang, 2006); and (3) while $\omega>\omega_n$+δ/2, $R_n(\omega)$ and $|H_n(\mathrm{i}\omega)|^2$ exhibit a sharp decreasing tendency to approach zero (Yu, 2006), so the influence of this integral range can be neglected on $\sigma_y(z)$. Then, $|H_n(\mathrm{i}\omega)|^2$ and $|H_n(\mathrm{i}\omega)|^2R_n(\omega)$ can approximately be estimated as

$$\left|H_n^{'}(\mathrm{i}\omega)\right|^2\cong 1/(4K_n^{2}\xi_n^2)=1/(4M_n^2\omega_n^{4}\xi_n^2),$$
$$\omega\in[\omega_n-\delta/2,\omega_n+\delta/2]$$
$$\left|H_n^{"}(\mathrm{i}\omega)\right|^2\cong 1/K_n^{2}=1/(M_n^2\omega_n^{4}),\ \omega\in\ 0,\omega_n-\delta/2 \tag{29}$$

$$\left|H_n(i\omega)\right|^2R_n(\omega)\cong 0,\ \omega\in\ \omega+\delta/2,+\infty \tag{30}$$

Introducing the dynamic magnification factor $\beta=1/2\xi_n$ under resonance conditions and substituting Eqs.(28), (29) and (30) into Eq.(27) yields

$$\sigma_y(z)=\sqrt{\sum_{n=1}^{N}\phi_n^2(z)\left[\left|H_n^{'}(\mathrm{i}\omega)\right|^2\int_{-\infty}^{+\infty}R_n(\omega)\mathrm{d}\omega+\left(1-\frac{1}{\beta}\right)^2\left|H_n^{"}(\mathrm{i}\omega_n)\right|^2R_n(\omega_n)\delta\right]}$$
$$=\sqrt{\sum_{n=1}^{N}\frac{\phi_n^2(z)}{M_n^2\omega_n^4}\left[\int_{-\infty}^{+\infty}R_n(\omega)\mathrm{d}\omega+\frac{(1-2\xi_n)^2}{4\xi_n^2}R_n(\omega_n)\delta\right]} \tag{31}$$

Here, the repetitive calculation of static response in the resonant frequency range is avoided, which was not considered in some previous research, for instance Yu (2006). However, the form of Eq.(31) is still not convenient for use in practice. Hence, it is valuable to find the specific algebraic expressions of δ and $\int_{-\infty}^{+\infty}R_n(\omega)\mathrm{d}\omega$.

(1) Determine δ: $|H_n(\mathrm{i}\omega)|^2$ is a narrow-band process and $R_n(\omega)$ is approximately regarded as white noise within the range of δ (Zhou and Gu, 2006), in which $R_n(\omega)$ is a constant and expressed by R_{const}. Then, δ is evaluated as

$$\delta=\frac{\int_{-\infty}^{+\infty}\left|H_n(\mathrm{i}\omega)\right|^2R_{\mathrm{const}}\mathrm{d}\omega}{2\left|H_n(\mathrm{i}\omega)\right|^2R_{\mathrm{const}}}=\frac{\int_{-\infty}^{+\infty}\left|H_n(\mathrm{i}\omega)\right|^2\mathrm{d}\omega}{2\left[1/(4M_n^2\omega_n^{4}\xi_n^2)\right]}=\pi\xi_n\omega_n$$
$$\Rightarrow\quad \delta=\pi\xi_n\omega_n \tag{32}$$

(2) Determine $\int_{-\infty}^{+\infty}R_n(\omega)d\omega$: since $R_n(\omega)$ is an even function, substituting it into Eq.(14) gives

$$\int_{-\infty}^{+\infty}R_n(\omega)\mathrm{d}\omega=2\int_0^{\infty}R_n(\omega)\mathrm{d}\omega=2\int_0^{+\infty}\frac{\Theta_nS_1(f)}{4\pi}2\pi\mathrm{d}f$$
$$=\Theta_n\int_0^{+\infty}S_1(f)\mathrm{d}f=\Theta_n\sigma_1^2 \tag{33}$$

The term Θ_n represents the constitutive relationship between $\int_{-\infty}^{+\infty}R_n(\omega)\mathrm{d}\omega$ and σ_1^2. Substituting Eqs.(32) and (33) into Eq.(31) obtains the following algebraic expressions:

$$\sigma_y(z)=\sigma_1\sqrt{\sum_{n=1}^{N}\frac{\Theta_n\phi_n^2(z)}{M_n^2\omega_n^{4}}\left[1+\frac{\pi a(1-2\xi_n)^2(\omega_nB/2\pi V_{\mathrm{H}})^{1.05}}{8\xi_n\left[1+b((\omega_nB/2\pi V_{\mathrm{H}})^2\right]^{1.5}}\right]} \tag{34}$$

$$y(z)=\mu\sigma_y(z) \tag{35}$$

where μ is the peak factor and is obtained as suggested by Kareem and Zhou (2003).

The advantage of this model is that it includes the contributions of the higher vibration modes and the algebraic expression allows it to be directly adopted by practitioners.

2.3 Conventional method

Generally, structural internal forces are needed in engineering design. For this purpose, current design practice often requires dynamic wind loads to be represented in terms of ESWLs, which is called the conventional method herein. In this method, the internal force solution is generally implemented in two steps as follows:

(a) Calculate the ESWLs

The wind load and the associated ESWLs are represented in terms of the contributions of vibration modes. If only the first mode is considered, the ESWLs (concentrated forces acting on the structural nodes) are given by (Zou, 2006; Zhang, 2006).

$$\boldsymbol{P}_1(z)=\omega_1^2\left\{m(z)y_1(z)\right\}=\mu\omega_1^2\left\{m(z)\sigma_{y1}(z)\right\} \tag{36}$$

where $\boldsymbol{P}_1(z)$ and $m(z)$ denote the vector of the ESWLs and the nodal mass at height, z, respectively.

(b) Calculate the internal forces

As is well known, the finite element method (FEM) is widely used to calculate the internal forces of a structure. To show the advantages of the ALD method, a brief description of this step is given here. $\boldsymbol{K}$ is the structural stiffness matrix, $\boldsymbol{K}^{\mathrm{e}}$ is the element stiffness matrix in a local coordinate system, $\boldsymbol{d}$ is the global nodal displacement vector, $\boldsymbol{d}^{\mathrm{e}}$ is the element nodal displacement vector in the local coordinates, where $\boldsymbol{d}$ needs to be transformed from the global coordinate system, and $\boldsymbol{F}^{\mathrm{e}}$ and $\boldsymbol{f}^{\mathrm{e}}$ are the rod-end internal force vector and fixed end reaction vector, respectively.

Solve for $\boldsymbol{d}$ using

$$\boldsymbol{Kd} = \boldsymbol{P}_1(z) \tag{37}$$

Calculate $\boldsymbol{F}^{\mathrm{e}}$ from

$$\boldsymbol{F}^{\mathrm{e}} = \boldsymbol{K}^{\mathrm{e}}\boldsymbol{d}^{\mathrm{e}} + \boldsymbol{f}^{\mathrm{e}} = \boldsymbol{K}^{\mathrm{e}}\boldsymbol{d}^{\mathrm{e}} \tag{38}$$

Note that the reason $\boldsymbol{f}^{\mathrm{e}}=\boldsymbol{0}$ in Eq.(38) is that the ESWLs are concentrated loads acting on the structural nodes as mentioned above. Additionally, note that it is $\boldsymbol{d}^{\mathrm{e}}$ instead of $\boldsymbol{P}_1(z)$ that becomes the necessary condition to calculate $\boldsymbol{F}^{\mathrm{e}}$.

2.4 ALD method and numerical verification

It is apparent that Eq.(36) does not need to be solved for $\boldsymbol{d}$ to estimate the ESWLs since $\boldsymbol{d}$ is in essence, simply $\boldsymbol{y}(z)$ obtained from Eq.(35). Furthermore, the structural internal forces should be deterministic if the structural nodal displacements are known. Therefore, $\boldsymbol{F}^{\mathrm{e}}$ can be obtained directly by using Eq.(38), and Eqs.(36) and (37) are not needed. This process doesnot require calculating the ESWLs for internal force evaluation and is called the ALD method. The ALD method has been shown to be simple and more convenient to use in practice.

From the analysis above, a flow chart (Fig.1) for computing the response of transmission towers is given below. The framework includes four parts: fundamental mode GFS, higher mode GFS, displacement response, and internal force calculation.

According to Fig.1, the transmission tower shown in Fig.2 is used to verify the feasibility and accuracy of the ALD method. The relevant parameters, calculation results and deformation sketches are given in Fig.3, Tables 1 and 2, respectively. Two nodes and two frames in the tower are investigated and 40 vector components are considered in the $\boldsymbol{\Phi}_1$.

Table 2 shows that the two methods show good agreement. Their deviation may come from adopting the conventional method calculation, $m(z)$, in Eq.(37).

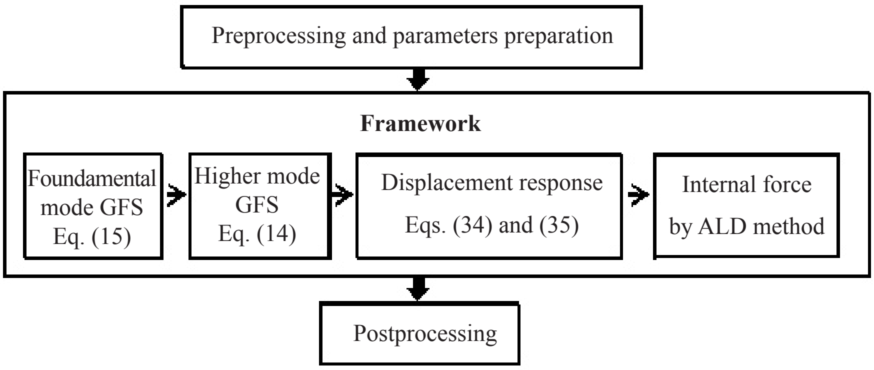


Fig. 1 Flowchart for calculating the responses of transmission towers

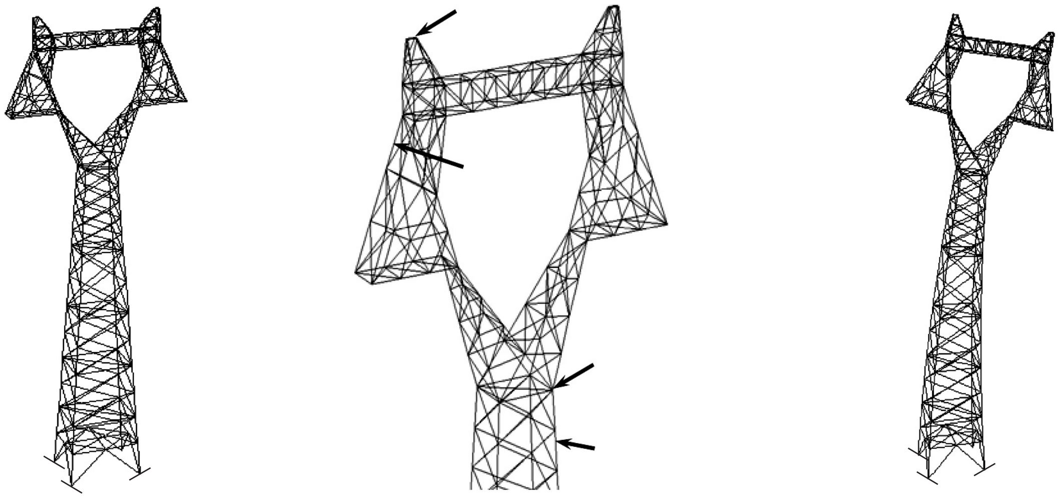
Table 1 General characteristics of the model

H(m)	V_{H}(m/s)	A(m^2)	B_y(m)	ρ(Kg/m^3)	E(GPa)	ρ_s(kg/m^3)	α	R	ξ_1
43	25	90.1	3.7	1.29	200	7.85×10^3	0.16	5%	2%

Note: E: Young's modulus; ρ_s: the density of steel

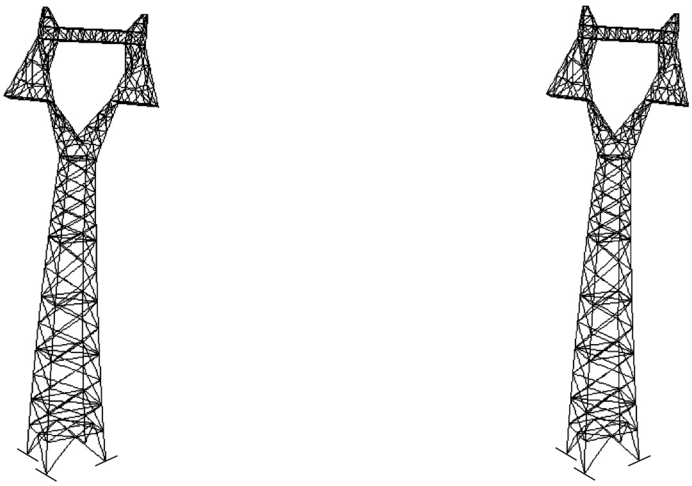
Table 2 Comparison of computational results between the conventional method and ALD method

Method	RMS value of response under the fluctuating wind			
	Joint displacement (m)		Frame axial force (N)	
	1	2	1	2
ALD (a)	0.174[using eq.(34)]	0.130[using eq.(34)]	107.2	-203.5
Conventional (b)	0.182	0.136	100.3	-210.7
(a-b)/a	-4.5%	-4.6%	6.4%	3.5%



(a) Model (H=43 m) (b) Investigation points (c) First mode shape (T_1=0.45 s)

Fig. 2 Transmission tower model & mode shape



(a) ALD method (b) Conventional method

Fig. 3 Transmission tower deformation

3 Concluding remarks

A new framework has been presented to evaluate the along-wind-induced dynamic responses of transmission towers, in which a practical higher mode generalized force spectrum (GFS) model is deduced on the basis of a fundamental mode generalized force spectrum (GFS) obtained from a wind tunnel. The framework also adopts the height-independent fluctuating wind power spectral density and Shiotami-Type spatial coherence function.

In addition, based on random vibration theory, a practical algebraic formula is derived which includes higher mode contributions. The formula can be used to evaluate the RMS value of the displacement response through proper simplification.

Finally, a method called the ALD method is presented to calculate the internal force of a transmission tower by directly using the displacement response obtained from Eq.(34).

It is shown that the results obtained by the ALD method show good agreement with the conventional method. The advantage of this approach is that it is rational and avoids calculating the ESWLs, which makes it easier to use in engineering practice.

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