

Article

Non-Stationary Wind Loading Identification for Large Transmission Tower Based on Dynamic Finite-Element Model Updating

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Abstract: An effective approach to deal with the structural failures of transmission towers in tornadic events is to develop good structural health monitoring (SHM) systems for them. However, the strategy for SHM of transmission towers against tornados should be different from the conventional atmospheric boundary layer (ABL) winds oriented ones, as the non-stationary nature of the tornados significantly differentiates them from the ABL winds. To satisfy the need of obtaining the highly time-varying whole-field stress on the structure in the course of the tornadic event for effective SHM, an innovative transient tornadic load distribution identification method is proposed for use, which is based on the field structural mode shape measurement and dynamic finite-element (FE) model updating. Via a numerical case study, it is noted that good effectiveness is achieved for the new load distribution identification method. Employing the Modal Assurance Criteria based FE model updating technique, the new method has the advantage of being easily embraced in practical SHM systems. It is found that when the transient tornadic velocity profile to be identified is noticeably different from the mode shape of the structure without undertaking external loads, the identified load pattern is very accurate for the new approach.



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Keywords: transmission tower; tornado; structural health monitoring; load distribution; parameter identification; numerical analysis; field modal experiment

1. Introduction

With the global warming, the infrastructures in China's power grids suffer from more serious meteorological events. According to statistics released recently by Chinese government, more than 20 convective weather events took place in east China in each year including a certain proportion of ruinous tornado disasters, leading to many steel transmission towers' collapses. As failures of transmission lines seriously harm the economic development of the region and the normal life of the general public, useful techniques for protecting steel transmission towers against tornados are expected from the engineering circle.

A good approach to deal with the structural failures of transmission towers in strong winds is to develop good structural health monitoring (SHM) systems. In history, some innovative SHM strategies have been formulated in this circle. For example, Fei et al. [1] proposed to use the decreased modal frequencies measured on the transmission towers to inverse the wind speeds on location and give the alarms to the cases with the identified speeds crossing the critical threshold. The rationality behind this practice is the rule

observed that the fundamental modal frequency calculated for a transmission tower dramatically decreases when the applied static wind load is increased to the instability critical value. Ref. [2] proposed to use a static force equilibrium equation to calculate the whole structure's real-time stress distribution according to its real-time behavior, as captured by the global positioning system in SHM of transmission towers. Ref. [3] reconstructed the global displacements of a transmission tower based on the simultaneous acceleration and strain measurements in corporation with the Kalman filter based data fusion algorithm to evaluate the structural performance.

Although the above mentioned methods developed are proven to be effective in use, we should note that they are all targeted for protecting the towers against the traditional atmospheric boundary layer (ABL) wind actions. In fact, tornados are different from ABL winds due to its strong non-stationary nature. According to Ref. [4], the time-consuming computational fluid dynamics technique was utilized to generate the non-stationary tornadic wind load samples for the required dynamic analyses to take place. The authors in [5] developed a non-stationary analytical method for tornadoes acting on structures with reference to some early observations and calculated the tornado-induced dynamic responses of several buildings. Refs. [6–8] used an advanced small-scaled non-stationary vortex simulation facility to undertake physical tests of tornadic events. In nature, the small-scale core vortex of the tornado travels along an unpredictable route during the tornadic event, leading to the changes of the distribution patterns of the loads acting on the affected tower with time. Therefore, the monitoring strategy for towers affected by tornados should be different from those subjected to ABL winds with the determinate load patterns. We suppose the focus of SHM methods proposed for transmission towers against tornados should be the real-time load distribution identifications which will facilitate accurate calculations of the transient whole-field stress on the structure for the effective reliability estimations of the real-time structural conditions, as per Ref. [2].

With the engineering background of a 131-m high free-standing transmission tower located in east China, this article first applies the finite-element (FE) model updating technique to the identification of load magnitude of the traditional ABL wind based on a field modal experiment. Then a novel SHM oriented load distribution identification practice for transmission towers against tornados based on the real-time mode shape measurement and FE model updating is proposed, supposing the target tower is subjected to a typical tornado. Via the numerical simulation, the effectiveness of the new method is validated. To the writer's knowledge, these works have not been reported by the engineering community so far, but their practical significance is immeasurable.

2. Engineering Background and Numerical Model

A self-supported transmission tower located in east China is chosen as the engineering background, which has a $26\text{ m} \times 26\text{ m}$ base and a height of 131 m (see Figure 1). Its main body consists of four upright main chords interconnected via cross bars, diagonal bracings and web members. Four cross arms are attached to the top part of the main body in order to shoulder the transmission lines.

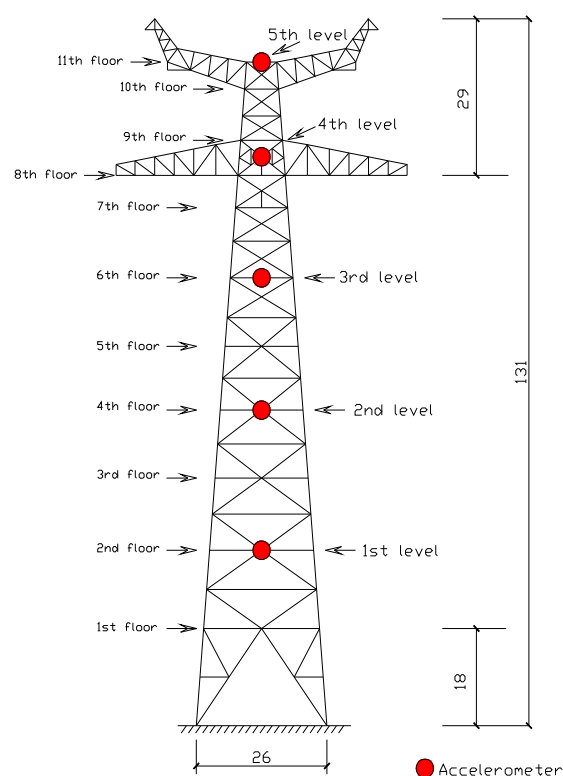


Figure 1. Dimensions of the 131-m high tower and sensors arrangement (unit: m).

In accordance with the design drawings, the transmission tower's 3D FE model is established using a commercial numerical platform. A beam-truss hybrid element simulation approach is adopted with members connected by flanges and welds modeled by beam elements and those connected by bolts modeled by truss elements to ensure their correct end stiffness respectively (see Figure 2).

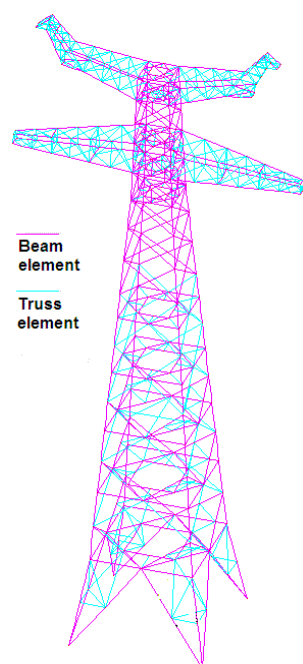


Figure 2. FE model of the large transmission tower.

After the FE model is so established, a modal analysis is followed with the calculated results listed in Table 1 and presented in Figure 3. Table 1 compares the calculated modal

frequencies and those measured on location using the modal experiment reported in Ref. [9], which suggests the comparatively good agreement in between them. As can be seen in Figure 3, the 1st–8th modes calculated are 1st sway, 1st bending, 1st torsion, 2nd sway, 2nd bending, 2nd torsion, 3rd sway, 3rd bending, respectively.

Table 1. Measured and computed modal frequencies.

Mode No.	Mode Shape	Measured Results (Hz) [9]	FE Model (Hz)	Difference (%)
1	1st Sway	0.7	0.809	15.54
2	1st Bending	0.725	0.827	14.10
3	1st Torsional	1.325	1.241	6.33
4	2nd Sway	2.175	2.073	4.68
5	2nd Bending	2.675	2.636	1.47
6	2nd Torsional	3.475	2.800	19.43
7	3rd Sway	3.55	2.816	20.68
8	3rd Bending	4.075	3.663	10.12
rms				13.29

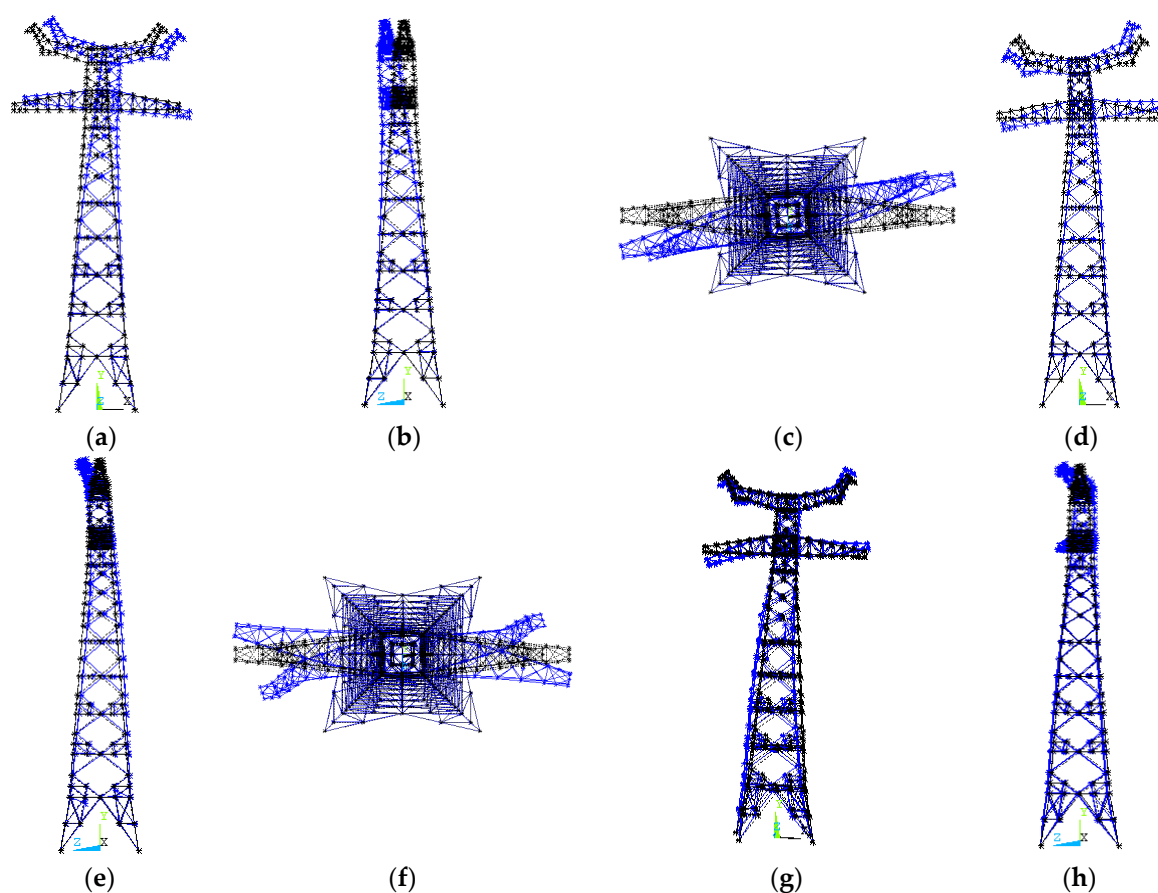


Figure 3. 1st–8th mode shapes calculated for the tower. (a) 1st mode; (b) 2nd mode; (c) 3rd mode; (d) 4th mode; (e) 5th mode; (f) 6th mode; (g) 7th mode; (h) 8th mode.

3. Field Modal Experiment in Moderate ABL Wind

As reported in Ref. [9], before putting into operation, a field modal experiment was conducted on the target free-standing tower without electric conductors installed using moving sensor modal test method by Southeast University (see Figure 4). 11×4 measuring points were arranged on the 11 tower floors (see Figure 1) and in 4 positions on each floor

to measure the longitudinal and lateral directional structural vibrations. 4 channels were available for measuring the two-directional structural responses at one shifting measuring point and at an immobile reference measuring point simultaneously. The overall structural responses excited by the environmental loads, mainly moderate ABL wind, were recorded in the form of velocity time-histories.



Figure 4. View of technical personnel undertaking field modal test.

Two directional time-histories and their auto-spectra measured at 2nd measuring point and 6th measuring point (immobile reference measuring point) are shown in Figs. 5 and 6, respectively. As can be seen, the velocity responses are greater at the higher floor (the 6th measuring point), and the first a few modal frequencies are identifiable on all power-spectral density (PSD) plots shown in Figures 5 and 6. Cross-spectra between time-histories measured at 2nd and 6th measuring points are shown in Figure 7, which indicates that the signs of the longitudinal and the lateral fundamental modes should be minus and plus on the 2nd point, respectively, according to Appendix A. Following the steps introduced in Appendix A, the low-order modal frequencies (see Table 1) and mode shapes (see Figure 8 for 1st~3rd modes) are successfully identified from the field modal test for further use. The modes shown in Figure 8 agree with those calculated via modal analysis (Figure 3a–c).

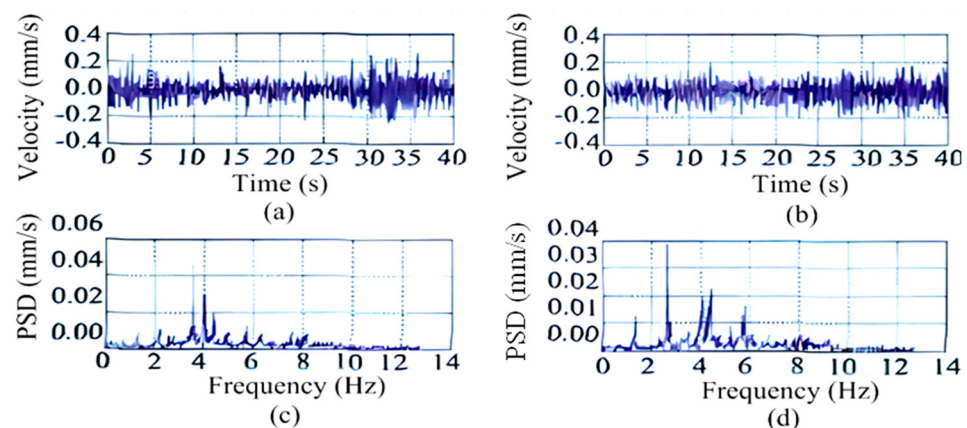


Figure 5. Two directional time-histories and auto-spectra measured at 2nd measuring point (modified based on Ref. [9]): (a) time-history in longitudinal direction; (b) PSD in longitudinal direction; (c) time-history in lateral direction; (d) PSD in lateral direction.

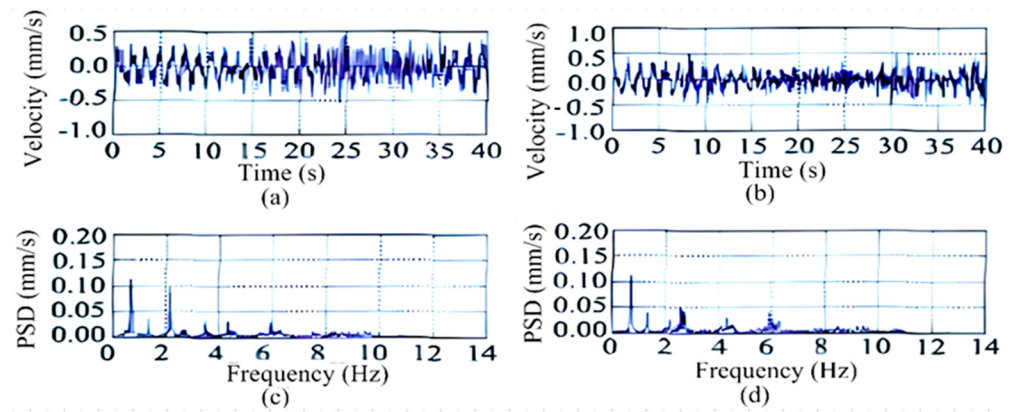


Figure 6. Two directional time-histories and auto-spectra measured at 6th measuring point (modified based on Ref. [9]): (a) time-history in longitudinal direction; (b) PSD in longitudinal direction; (c) time-history in lateral direction; (d) PSD in lateral direction.

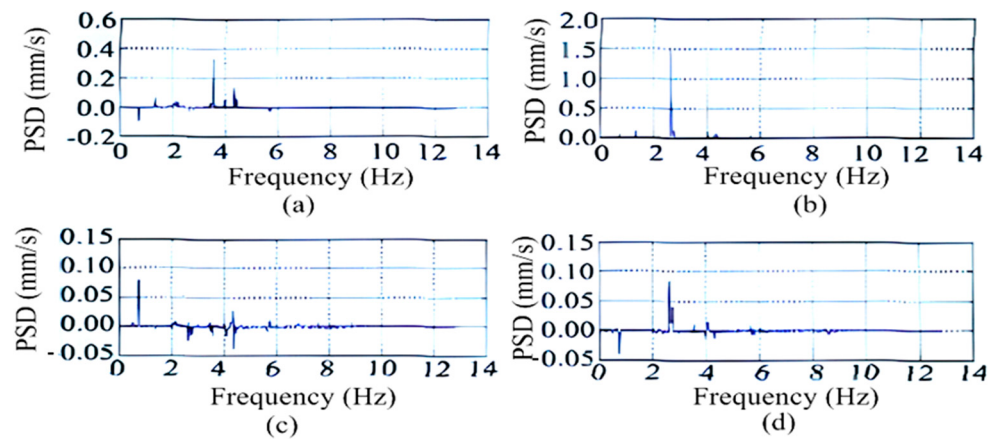


Figure 7. Cross-spectra between time-histories measured at 2nd and 6th measuring points (modified based on Ref. [9]): (a) real part of longitudinal cross-spectrum; (b) real part of lateral cross-spectrum; (c) imaginary part of longitudinal cross-spectrum; (d) imaginary part of lateral cross-spectrum.

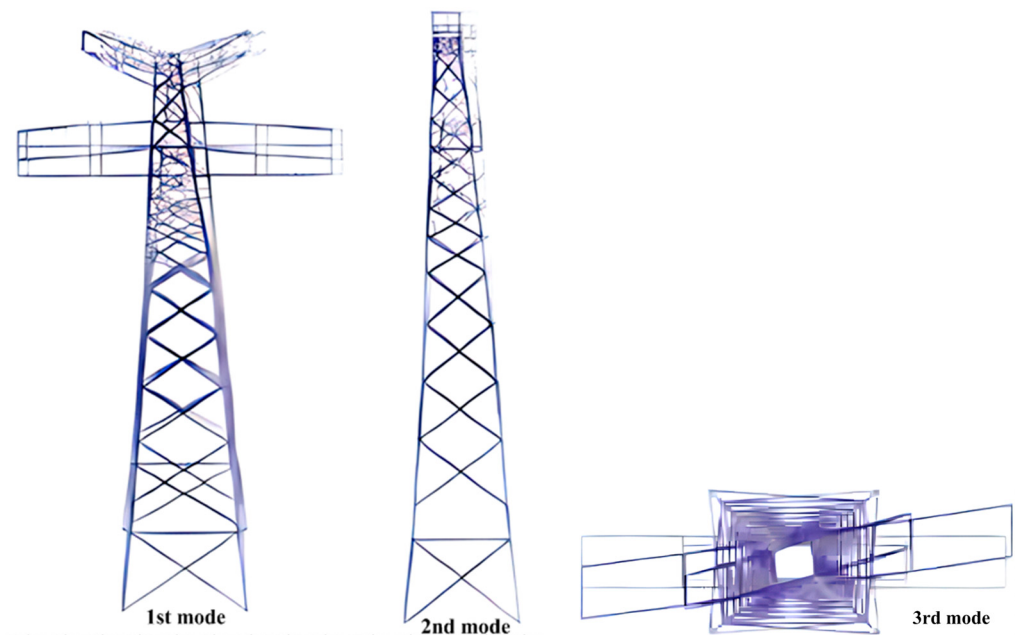


Figure 8. 1st~3rd (fundamental lateral and longitudinal bending and 1st torsional) modes identified from field modal test (re-produced from Ref. [9]).

4. Load Magnitude Identification Using Structural Modal Frequency Measurement

Different from the traditional practice of using modal parameters to identify the structural parameters via the FE model updating, Ref. [1] proposed the method of using modal parameters to identify the external loads acting on the structure based on the new observation that the fundamental modal frequency calculated for a transmission tower decreases when the applied static wind load is increased (see Section 1). In this portion of study, the magnitude of the wind loading on the transmission tower (or the mean wind speed on location) at the time the field modal test has been undertaken (see Section 3) is identified using the numerical model established in Section 2 following the model updating practice. It is assumed that the mean wind velocity profile (or the wind load distribution pattern) is determinate in accordance with the Chinese design standard [10] which can be expressed using a power-law formula with an exponent of 0.16. Regarding an overall amplification factor of the design wind loads applied to the FE model and the stiffness of the three-dimensional supports at the tower bottom as the updated parameters and the root-mean-square value between the calculated and the measured 1st and 2nd modal frequencies as the objective function, dynamic FE model updating is conducted using the first order method (see Appendix C) via minimizing the penalty function-based unconstrained objective function by iteratively changing the updated parameter. The objective function is gradually minimized and converged after seven iterations (Figure 9). The optimized overall amplification factor is obtained as being 0.338, and the basic wind speed at the height of 10 m is therefore calculated as being 13.2 m/s which is close to the result measured on location using a handheld anemometer (≈ 10 m/s). This proves the effectiveness of the dynamic model updating approach in load magnitude identification for traditional ABL wind events. Besides, it is found that the model with the optimized three-dimensional supports arranged at the tower bottom can generate commensurate modal analysis results with a model with the foundation modeled in detail, which also indicates the effectiveness of the performed FE model updating.

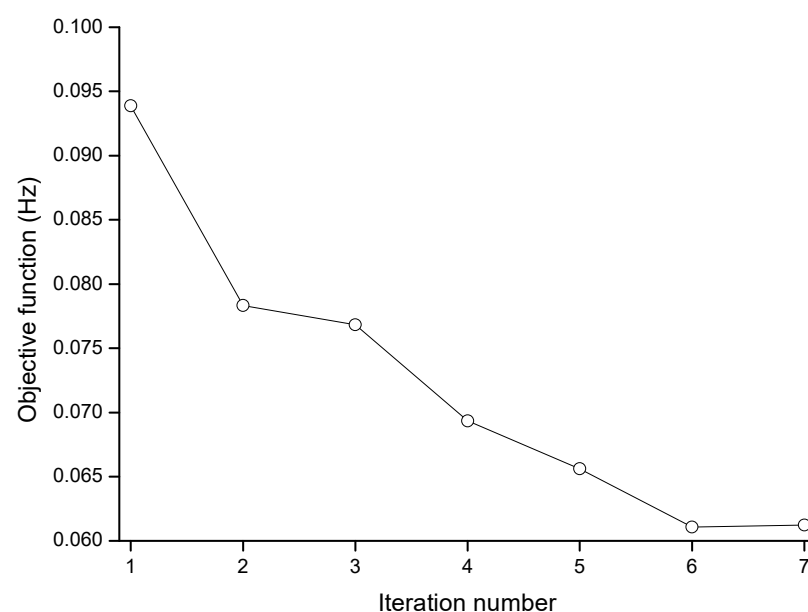


Figure 9. Convergence of the objective function for ABL wind case.

5. Tornadoic Velocity Model

The tornadoic velocity model traditionally employed in structural analyses in history is the two dimensional Rankine's model, which wrongfully ignores the axial flow movements. The three dimensional model proposed by Wen [11] which well considers the axial flow movements is more rational in this regard. According to Wen's model, the thickness of the tornadoic boundary layer $\delta(r')$ can be written as:

$$\delta(r') = \delta_0[1 - \exp(-0.5r'^2)] \quad (1)$$

in which, $r = r'/r_{\max}$; r' is the distance from the targeted position to the center of the tornado; $r_{\max}$ is the radius corresponding to the maximum tangential velocity; δ_0 is the thickness of the tornadoic boundary layer for $r \gg 1$, which is chosen as being $\delta_0 = 457m$ according to the general practice.

As shown in Figure 10, the boundary layer interface divides the tornadoic flow field into two parts. Therefore, the nature of the flow depends on the height of the targeted position (the z coordinate). Above the interface ($z > \delta$), the following equations can describe the velocity components along the three directions:

$$\begin{cases} T(\eta, r) = f(r) = 1.4V_{\max}[1.0 - \exp(-1.256r^2)]r^{-1} \\ R(\eta, r) = 0 \\ W(\eta, r) = 93r^3 \exp(-5r)V_{\max} \end{cases} \quad (2)$$

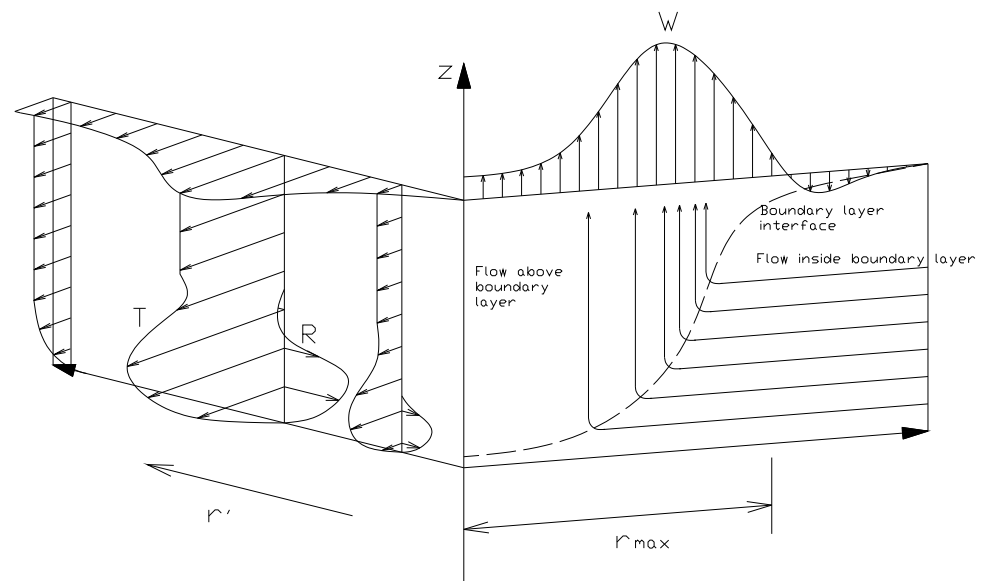


Figure 10. Diagram of parameters describing a tornadoic velocity field.

Below the interface ($z \leq \delta$), the velocity components are formulated as:

$$\begin{cases} T(\eta, r) = f(r)[1.0 - e^{-\pi\eta} \cos(2b\pi\eta)] \\ R(\eta, r) = f(r)\{0.672e^{-\pi\eta} \sin[(b+1)\pi\eta]\} \\ W(\eta, r) = 93r^3 \exp(-5r)V_{\max}[1.0 - e^{-\pi\eta} \cos(2b\pi\eta)] \end{cases} \quad (3)$$

in which, $T(\eta, r)$, $R(\eta, r)$, $W(\eta, r)$ are the tangential velocity, the radial velocity, the vertical velocity, respectively; $V_{\max}$ is the maximum tangential velocity; $b = 1.2e^{-0.8r^4}$; $\eta = z/\delta(r')$; z is the vertical distance from the ground to the targeted position.

According to Ref. [12], when calculating the tornadoic velocity time-history at a targeted position on the tornado passage, T and R formulated in Equations (2) and (3) should be

adjusted to consider the net effect of the no-zero translating of the tornado with velocity V_T (see Figure 11 for the required adjustments). With reference to Ref. [11], the F3 grade tornado with $V_{\max} = 74.1$ m/s and $r_{\max} = 50$ m is selected for the following case study of the tornado-induced structural response of a free-standing transmission tower. With reference to the F3 tornado observed in Mullinville, KS on 7 May 2002 [13], V_T is chosen as being 5.7 m/s for the case study. As shown in Figure 11, the free-standing transmission tower described in Section 2 is assumed to be located on the pass of the F3 grade tornado with its front directly facing the tornado for the case study.

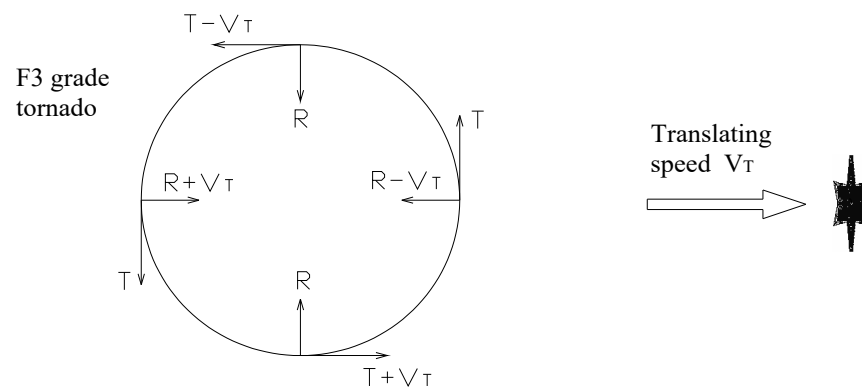


Figure 11. Diagram of a F3 grade tornado approaching a free-standing transmission tower.

6. Calculation of Structural Responses to Tornadic Actions

An earlier research (Ref. [14]) has calculated the structural response of the tower to tornadic loads in the same event based on Wen's velocity model described in Section 5. According to Ref. [14], the process of the calculation include the following steps: (1) calculate the tornadic velocity field based on Wen's model; (2) calculate the time-varying tornadic loads acting on the tower using the standard formula stipulated in standards to transform the velocity samples to load samples; (3) conduct the time-history analysis via applying the loads to the FE model. In this course of the dynamic calculation, the main software function employed is Newmark- β method whose details of analysis are presented in Appendix D.

The present calculation process basically follows the above, but with an appropriate improvement on a weak point of the calculation conducted by Ref. [14], i.e., only the low-frequency fluctuations are taken into account in calculating the tornadic velocity/load samples based on the time-varying means calculated using Wen's model, while the high-frequency fluctuations are ignored in their dynamic analysis. The reason behind their treatment is that no empirical model has been proposed to describe the high-frequency fluctuations of the tornadic velocities for use so far due to the limited field measurements of tornados.

As the alternative method, we use the conventional ABL spectrum to generate the high-frequency velocity fluctuations for the tornado which are then added to the time-varying means to obtain the tornadic velocity samples with both high-frequency and low-frequency components included. The ABL velocity fluctuations are generated according to the method described in Ref. [14]. After that, the longitudinal and the lateral load samples at 20 heights are calculated from velocity samples in accordance with the practice adopted by Ref. [14], and FE time-history analysis is then conducted using the Newmark- β method with 0.1 s time step via applying the load time-histories to the tower body of the model assuming that the tornado travels from an upstream position at 200 m in front of the tower to a downstream position at 200 m behind the tower.

The lateral and longitudinal displacement responses so calculated at the tower top are shown in Figure 12. As can be seen, at the beginning, the lateral displacement gradually

increases and reaches the maximum value (0.5 m) when the tornado is at an upstream position of around 50m in front of the tower. Then, the lateral response significantly decreases. After the tornado passes the transmission tower, the lateral displacement changes its direction. When the tornado moves away from the tower, the lateral displacement first significantly increases and subsequently gradually decreases. In addition, as shown, the longitudinal response is negligible throughout the loading process. These observations agree well with those reported in Ref. [14]; however, a noticeable difference is also observed between the displacement responses calculated for the present study and those shown in Ref. [14], i.e., high-frequency fluctuations can be seen on the response curves shown in Figure 12, which were not seen on those shown in Ref. [14]. This is obviously due to the present treatment in calculating the tornadic velocity samples with both high-frequency and low-frequency components included. In addition, Figure 12 also compares the velocity samples calculated at 113-m height against the responses so obtained at the tower top. As shown, they show similar variation tendencies in both longitudinal and lateral directions.

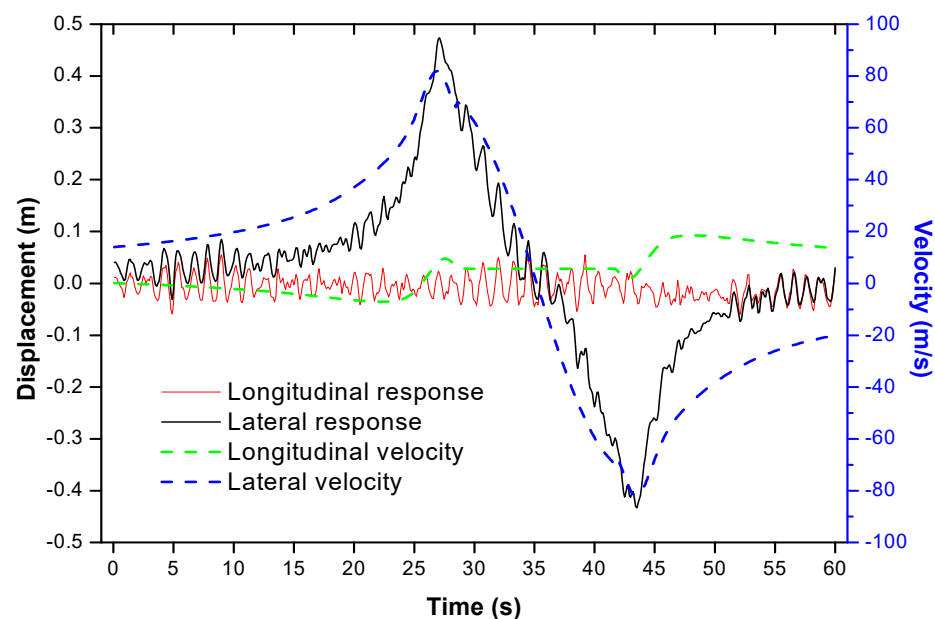


Figure 12. Displacement responses calculated at the tower top.

The displacement responses shown in Figure 12 are then transformed into acceleration responses which are processed using the wavelet analysis with complex Morlet wavelet function. Following the modal parameter identification practice with wavelet analysis reported in Ref. [15] which has the advantage of disclosing the variations of the modal frequencies with time for the non-stationary events, the time-varying fundamental natural frequencies of in both lateral and longitudinal directions are identified in view of the maxima in time-frequency energy spectra obtained using the wavelet analysis from the acceleration responses calculated at the tower top (Figure 13). As can be seen, the first order modal frequencies for both the lateral and the longitudinal directions are at the lowest value (≈ 0.7 Hz) in the time interval [25 s, 40 s], suggesting that the static wind loading effects are most significant in this duration (the overall wind force acting on the structure is the largest). This is supported by the rule that the fundamental modal frequency for a transmission tower decreases when the applied static wind load is increased [1]. Therefore, the following studies focus on identifying the tornadic load patterns on the tower at the local time 25 s.

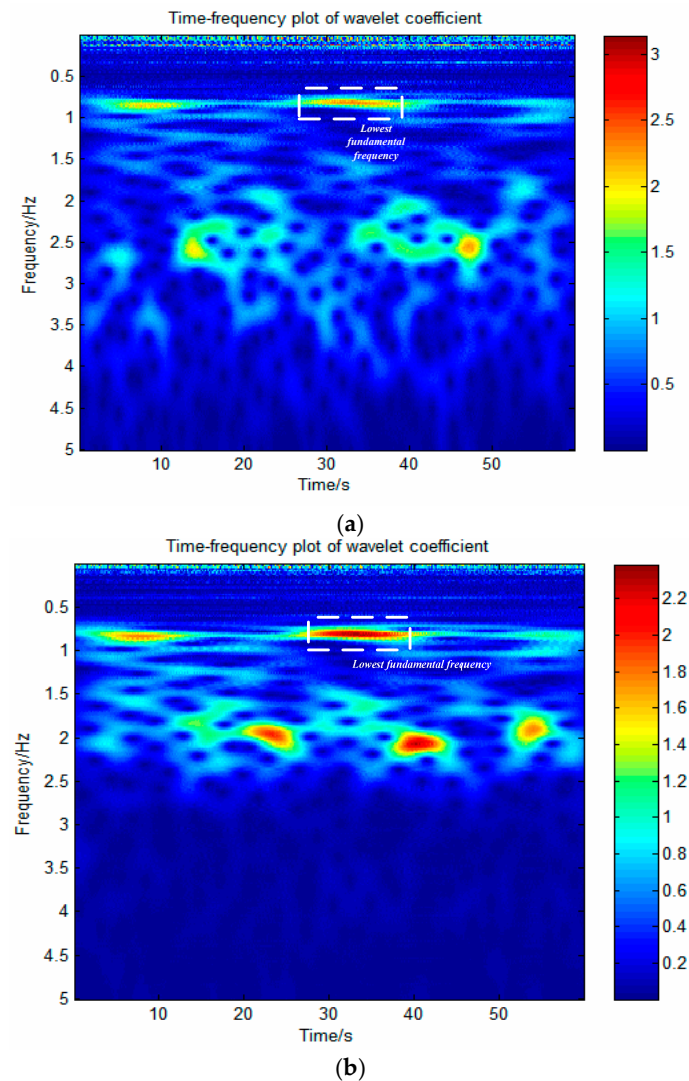


Figure 13. Time-frequency energy spectra for acceleration responses calculated at the tower top via wavelet analysis. (a) Lateral response. (b) Longitudinal response.

7. Load Distribution Identification Using Structural Mode Shape Measurement

Section 4 is basically to deal with ABL winds whose velocity profiles are comparatively determinate that have already been measured by many researchers and generalized into empirical formulae for direct use. Therefore, distribution patterns of wind loads acting on the structure are determinate for ABL winds, and as a result, the load identification work undertaken in Section 4 merely focuses on the magnitude of the load without considering the load pattern. In order to fulfill the purpose of Section 4, only the fundamental frequency of the structure is employed as there is only one load parameter (the magnitude of the reference velocity) to identify. However, with regard to the present research focusing on the tornados, the load distribution pattern consisted of many concentrated forces is changeable over time, which should be simultaneously identified using the modal parameters. As it is found that the structural mode shape contains much more information concerning the load distribution applied to the structure comparing to the structural modal frequency, possibly meeting the need of identifying multiple parameters, the load pattern identification approach employed by the present study is based on the measurement and the utilization of the mode shape.

To obtain the mode shape via the modal experiment, 10 accelerometers are supposedly uniformly arranged at 5 heights along the vertical direction to measure the longitudinal and the lateral structural responses at these positions (see Figure 1). It is supposed that 10 structural acceleration samples are generated by the 10 sensors during the short time interval [22.5 s, 27.5 s] for use, which are actually obtained from the calculated results in Section 6 and presented in Figure 14.

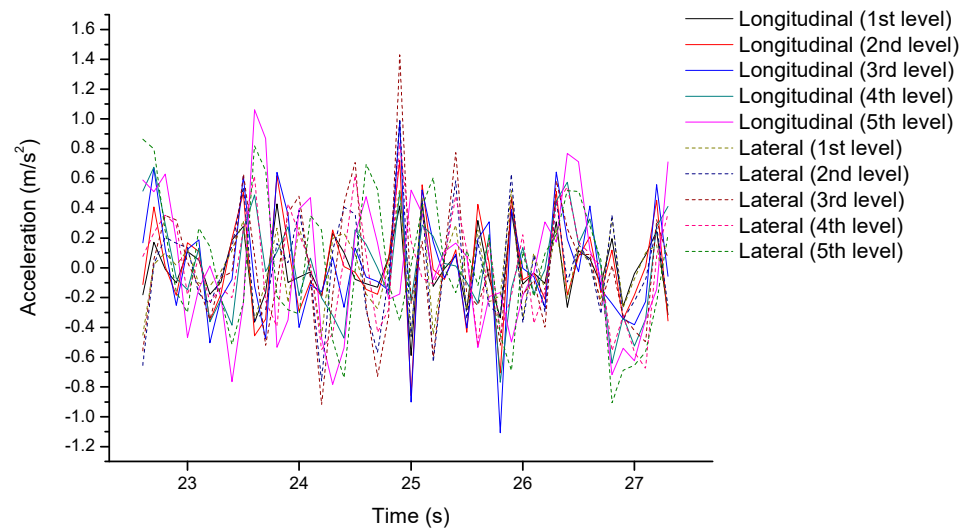


Figure 14. Acceleration samples employed for modal experiment.

Using the Frequency Domain Decomposition (FDD) method with its theories described in Appendix B, the modal parameters are identified for the tower subjected to the tornado at 25 s based on the structural acceleration samples shown in Figure 14. As a nonparametric approach in frequency domain, the modal parameter identification using FDD approach requires the manual peak picking on the first singular values of the PSD matrix calculated for the structural responses, and the peaks of the PSD of the structural response (Figure 15) are reasonably picked to identify the structure's 1st and 2th modal parameters (this step will be automatically executed in actual applications). The 1st and the 2th natural frequencies so identified are 0.625 Hz and 0.6641 Hz respectively which are found to be much smaller than the corresponding results calculated for the tower without external loads in Section 2 (0.809 Hz and 0.827 Hz, respectively). This observation agrees the rule reported in Ref. [1]. The 1st and 2th normalized mode shapes so identified are shown and compared with those calculated in Section 2 for the structure without undertaking wind actions in Figure 16. From Figure 16, it can be learnt that the fundamental lateral (1st) modes for the tower with and without tornadic actions are close together, while the 2nd (fundamental longitudinal) modes calculated are noticeably different between the tornadic case and the case without wind actions. It is therefore suggested that the 2nd mode identified for the tornadic case contains more information concerning the transient load distribution than the 1st mode identified does. In addition, the resemblance between the actual lateral velocity profile (see Figure 9) and the fundamental lateral mode of the structure without loads is greater than that between the actual longitudinal velocity profile (Figure 9) and the corresponding longitudinal mode of the structure without loads. Therefore, we suppose the reason behind the above-mentioned observation that the 1st mode does not change much after the tornadic loads are applied to the structure is that only for the cases that the applied load distributions are noticeably different from the modes of the original structures, the mode shapes significantly alter after the structures are subjected to the external loads. As can

be seen in Figure 16, the 2nd mode identified for the tower with loads at 25 s during the tornadic event resembles the actual longitudinal velocity profile (Figure 9) to some extent.

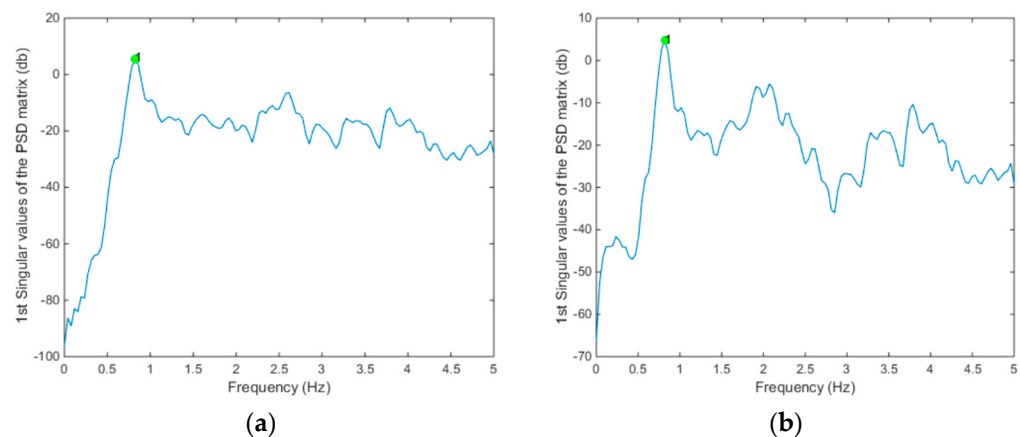


Figure 15. 1st singular values of the PSD matrixes. (a) Lateral direction; (b) Longitudinal direction.

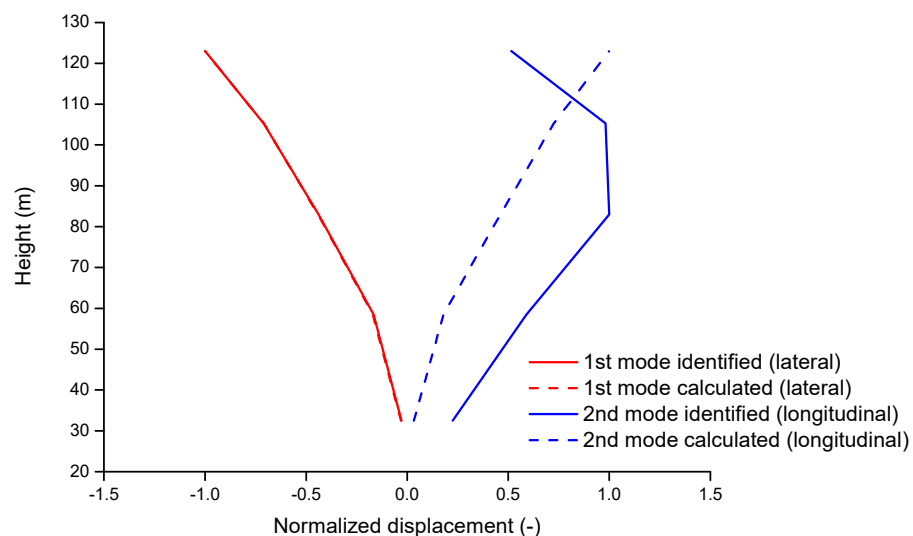


Figure 16. Lateral and longitudinal modes identified for the tower subjected to tornado and calculated for the tower without undertaking wind actions.

With the mode shapes identified for the tower subjected to the tornado at 25 s, the transient tornadic load distributions at that local time are identified using the FE model updating on ANSYS platform. In detail, the first order optimization algorithm provided by the ANSYS platform with its theories presented in Appendix C is employed. During the optimization process, the wind loads are applied to the numerical modal, and the 1st and 2th modes are calculated via the modal analysis in each iteration. The updated parameters are chosen to be the longitudinal and the lateral tornadic loads acting on the main body of the tower at 20 heights ($2 \times 20 = 40$ parameters in total). The initial values of the updated parameters are set to be the design values according to the Chinese structural design standard DLT5154-2002 [10] for the case of the 45 degrees skewed wind. The constraints of the updated parameters are set to be $[0.5 \times \text{initial value}, 1.5 \times \text{initial value}]$. The constrained objective functions are set to be $Q = 2 - \text{MAC1} - \text{MAC2}$ (MAC1 and MAC2 are the longitudinal and the lateral Modal Assurance Criteria (MAC) values [16] between the modes calculated in the current iteration and the identified modes shown in Figure 16, respectively). The state-variables were the first two order modal frequencies whose lower/upper bounds were $\pm 20\%$ based on the values identified above for the tower

subjected to the tornado at 25 s. Model updating using first order optimization technique was utilized to minimize the penalty function-based unconstrained objective function through the iterations. The objective function was gradually decreased and converged after 9 iterations (see Figure 17). With the optimized parameters (40 concentrated forces acting on the tower body), both the longitudinal and the lateral tornadic velocity profiles at 25 s are calculated, and compared with the accurate profiles in Figure 18. As can be seen, the longitudinal profile identified via the model updating is close to the accurate profile along the height; while the lateral profile so identified is partly different from the accurate profile at the middle and the upper parts.

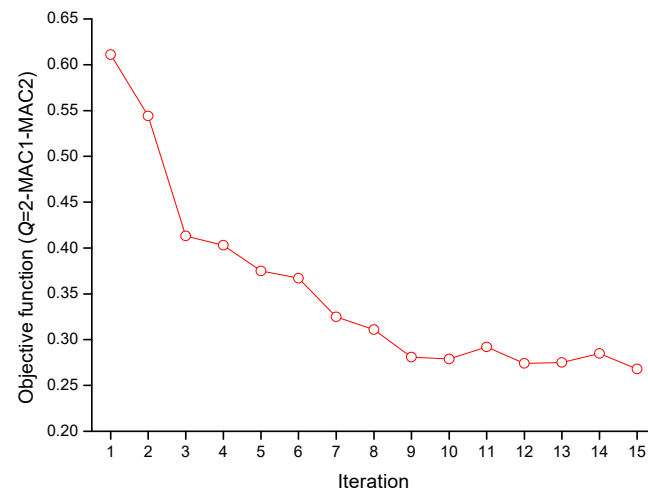


Figure 17. Convergence of the objective function for tornado case.

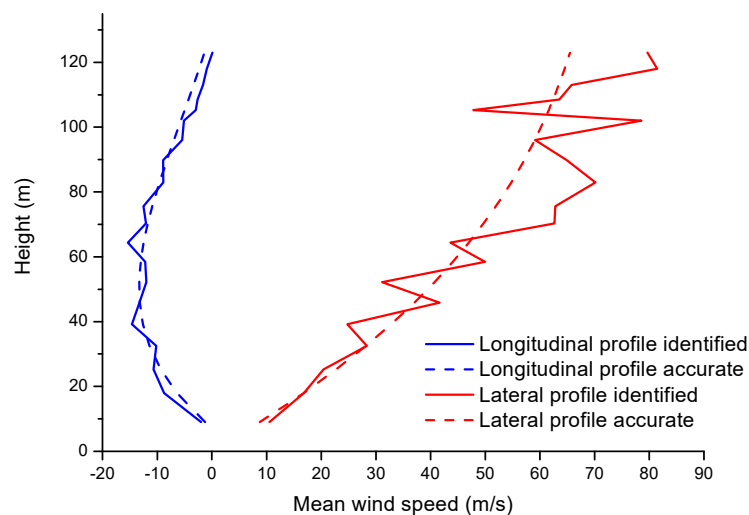


Figure 18. Tornadic velocity profiles identified using the approach based on mode shape measurement in comparison to accurate profiles.

8. Comparison with Conventional Method in Load Distribution Identification

The conventional method for load distribution identification basically relies on wind measurement using the anemometer and empirical velocity model. It is supposed that the longitudinal and the lateral velocities have been measured at 25 s by an anemometer arranged at the free-end of a lower cross-arm, which are 18.03 m/s and 63.61 m/s, respectively, based on the input to the numerical calculation described in Section 6. As shown in Figure 19, the maximum tangential velocity of the tornado $V_{\max}$ and the radius corre-

sponding to the maximum tangential velocity $r_{\max}$ are assumed to be correctly provided by the timely local weather forecast (74.1 m/s and 50 m, respectively), and the distance from the target tower to the center of the tornado r' and the inclined angle between the line connecting the target tower and the center of the tornado and the transmission lines α are regarded as the two unknown parameters to be identified. By feeding the known parameters to Equation (3), an equation system with two unknowns (r' and α) are obtained. By solving the equation system using a numerical method, the r' and α are identified for the time 25 s as being 55.82 m and -7.36 degrees respectively with 2.92% and 2.04% relative errors from the accurate values. Using the two identified unknowns, the lateral and the horizontal tornadic velocities profiles at the target position at 25 s are identified and compared with the accurate ones in Figure 20, which suggests that their agreements are comparatively good.

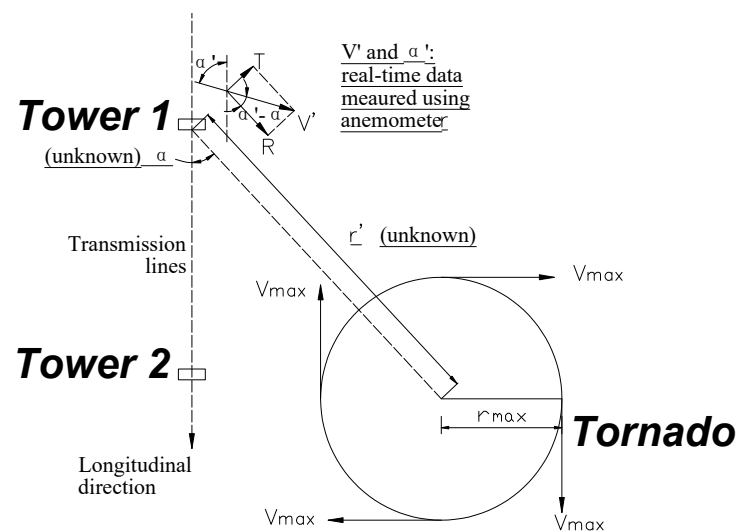


Figure 19. Diagram for the approach based on wind measurement and empirical velocity model.

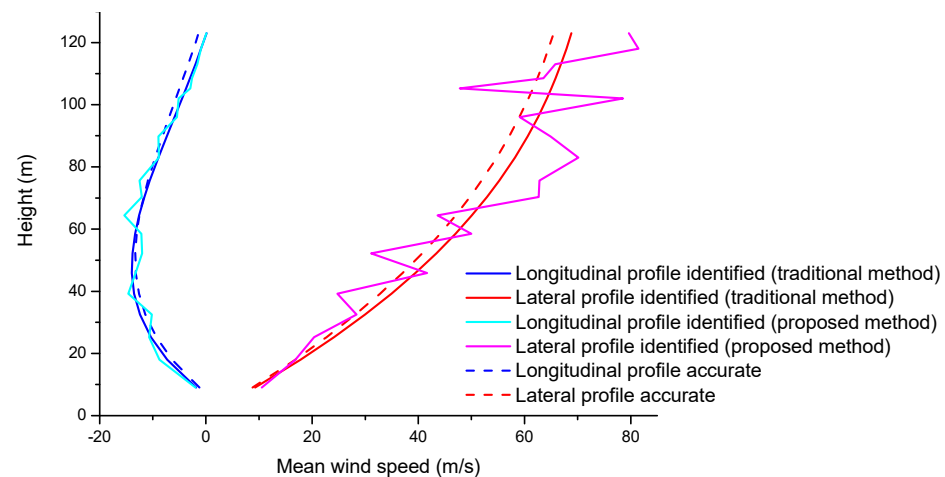


Figure 20. Tornadic velocity profiles identified using the approach based on wind measurement and empirical velocity model in comparison to proposed method.

Figure 20 further compares tornadic velocity profiles identified using the approach based on wind measurement and empirical velocity model (the traditional method) with those of the proposed method. As can be seen, better effectiveness is achieved for the load distribution identification method based on the flow measurement than for the method based on structural mode shape measurement. However, the traditional method is subjected to the issue that accurate prior knowledge concerning detailed information of the

tornado (the maximum tangential velocity of the tornado and the radius corresponding to the maximum tangential velocity) must be provided by the local weather station. Therefore, its applicability is limited with respect to the fact that the meteorological information of a tornado cannot be forecasted with up-to-date measurement techniques at present.

9. Conclusions

The main findings of this study concerning load distribution identifications for transmission tower are summarized below:

- (1) The strategy for SHM of transmission towers against tornados should be different from the conventional ABL winds oriented ones, as the non-stationary nature of the tornadic events significantly differentiates them from the ABL wind events. To obtain the highly time-varying whole-field stress on the structure for effective SHM, the transient load distribution should be identified for the tornado case.
- (2) Two case studies are undertaken to demonstrate the effectiveness of the innovative idea of utilizing the modal parameters measured on location to identify the wind loadings on the structure via FE model updating for SHM purposes, which employ the ABL wind event and the tornadic event, respectively. Basically, the ABL wind case according to the field measurement work does not have anything to do with the simulated tornadic case. The first case is targeted for the wind load magnitude identification, while the second case is for the wind load distribution identification. These are due to the fact that the ABL wind is of the deterministic velocity profile, while the tornadic velocity profile is highly time-varying. It is noted that dynamic FE model updating is applicable to both the wind load magnitude identification and the wind load distribution identification, which require the structural modal frequency and the structural mode shape measured on location to construct the objective functions, respectively.
- (3) The proposed load distribution identification method has the advantage of being easily embraced in practical SHM systems. Figure 21 presents relative errors of the identified tornadic velocity profiles using the proposed method (see Figure 18), which is the evidence that demonstrates the effectiveness of the proposed method. It is found that when the transient tornadic velocity profile to be identified is noticeably different from the mode shape of the structure without undertaking external loads, the identified load distribution pattern is very accurate.

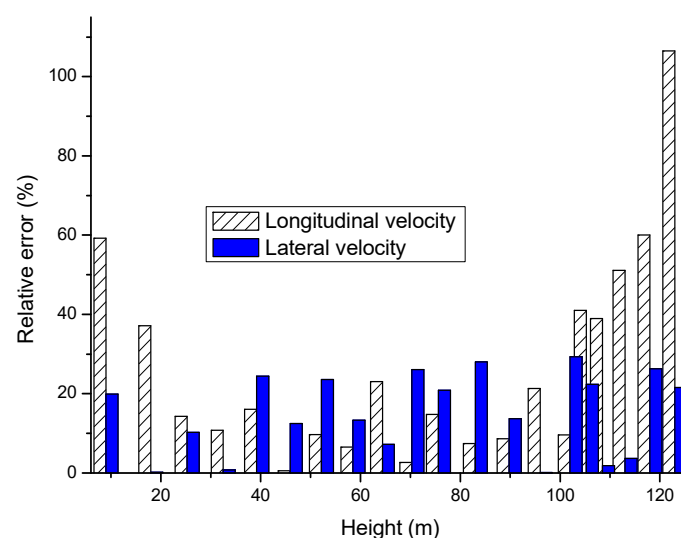


Figure 21. Relative errors of the identified tornadic velocity profiles.

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Appendix A. Theory of Moving Sensor Modal Test

The moving sensor modal test requires two accelerometers in the modal test, including one fixed sensor and one moving sensor. By displacing the moving the sensor among different measuring points and repeated simultaneous measurements by the fixed sensor and the moving sensor, the data acquisition can be undertaken for the modal test. Modal parameters can then be identified through single-channel or double-channel Fast-Fourier-Transforms. In practice, the modal frequencies f_i should be identified by peak-picking from the auto power spectrums of measured acceleration samples. Besides, as shown in Equations (A1) and (A2), the ratio of the spectral amplitude at f_i on the auto power spectrum of the moving sensor to that of the fixed sensor should be regarded as the absolute value of the modal shape $|\phi(f_i)|$, and the phase of the modal shape should be identified from the sign of the cross power spectrum between the acceleration samples measured using the moving sensor and the fixed sensor at f_i :

$$|\phi(f_i)| = \left| \frac{B(f_i)}{A(f_i)} \right| = \left| \frac{B(f_i)\overline{B(f_i)}}{A(f_i)\overline{A(f_i)}} \right|^{\frac{1}{2}} = \left| \frac{G_{bb}(f_i)}{G_{aa}(f_i)} \right|^{\frac{1}{2}} \quad (\text{A1})$$

$$\text{sgn}(\phi(f_i)) = \text{sgn}(\text{Real}(G_{ba}(f_i))) \quad (\text{A2})$$

in which, $A(f_i)$ and $B(f_i)$ are the Fourier transformations of the acceleration samples measured using the fixed sensor and the moving sensor, respectively; $G_{aa}(f_i)$ and $G_{bb}(f_i)$ are the auto power spectrums for the fixed sensor and the moving sensor, respectively; $G_{ba}(f_i)$ is the cross power spectrum between the acceleration samples measured using the moving sensor and the fixed sensor.

Appendix B. FDD Modal Identification Algorithm

According to Ref. [17], the first step in FDD identification is to estimate the PSD matrix. The estimate of output PSD $\hat{G}_{yy}(j\omega)$ known at discrete frequencies $\omega = \omega_i$ is then decomposed by taking the Singular Value Decomposition of the matrix

$$\hat{G}_{yy}(j\omega_i) = U_i S_i U_i^H \quad (\text{A3})$$

where the matrix $U_i = [u_{i1}, u_{i2}, \dots, u_{im}]$ is a unitary matrix holding the singular vectors u_{ij} and S_i is a diagonal matrix holding the scalar singular values s_{ij} . Near a peak corresponding to the k th mode in the spectrum, this mode or maybe a possible close mode will be dominating. If only the k th mode is dominating, the first singular vector u_{i1} is an estimate of the mode shape

$$\hat{\phi} = u_{i1} \quad (\text{A4})$$

The corresponding singular value is the auto PSD function of the corresponding single degree of freedom (SDOF) system. The PSD is identified around the peak by comparing the mode shape estimate $\hat{\phi}$ with the singular vector for the frequency lines around the peak. As long as a singular vector is found that has high MAC value with $\hat{\phi}$, the corresponding singular value belongs to the SDOF density function.

From the piece of the SDOF density function obtained around the peak of the PSD, the natural frequency and the damping can be estimated via transforming the SDOF PSD into time domain using inverse fast Fourier transform and calculating the crossing times and the logarithmic decrement of the corresponding SDOF auto correlation function.

Appendix C. Main Steps of First Order Method

The constrained optimization problem can be formed as

$$\text{Minimize } \Pi = \Pi(x) \quad (\text{A5})$$

Based on Equation (A5), three main steps of the first order method are described as follows [18]:

- (1) The constrained problem statement expressed in Equation (A5) is transformed into an unconstrained one using penalty function. An unconstrained form of Equation (A5) is formulated as follows

$$Q(x, q) = \frac{\Pi}{\Pi_0} + \sum_{i=1}^n P_x(x_i) + q \left[\sum_{i=1}^{m_1} P_g(g_i) + \sum_{j=1}^{m_2} P_h(h_j) + \sum_{k=1}^{m_3} P_w(w_k) \right] \quad (\text{A6})$$

where $Q(x, q)$ is dimensionless unconstrained objective function with the design variables and parameter q ; P_x, P_g, P_h, P_w are penalties applied to the constrained design and state variables, and Π_0 refers to the objective function value that is selected from the current group of design sets.

- (2) Derivatives are formed for the objective function and the state variable penalty functions leading to the search direction in design space. For each optimization iteration (j), a search direction vector $d^{(j)}$ is devised. The next iteration ($j + 1$) is obtained from Equation (A7). In this equation, measured from $x^{(j)}$, the line search parameter S_j corresponds to the minimum value of Q in the direction $d^{(j)}$.

$$x^{(j+1)} = x^{(j)} + S_j d^{(j)} \quad (\text{A7})$$

- (3) Various steepest descent and conjugated direction searches are performed during each iteration until the convergence is reached. Convergence is assumed when comparing the current iterations design set (j) to the previous set ($j - 1$) and the best set (b) as shown in Equation (A8), in which τ is objective function tolerance.

$$\left| \Pi^{(j)} - \Pi^{(j-1)} \right| \leq \tau \text{ and } \left| \Pi^{(j)} - \Pi^{(b)} \right| \leq \tau \quad (\text{A8})$$

Appendix D. Newmark- β Method

For a structure under the influence of a time-history load, the vibration equation can be used to represent the whole process:

$$[M]\{\ddot{y}(t)\} + [C]\{\dot{y}(t)\} + [K]\{y(t)\} = \{P(t)\} \quad (\text{A9})$$

In which the $[M]$, $[C]$ and $[K]$ are the system's mass matrix, damping matrix and stiffness matrix, respectively. $\{\ddot{y}(t)\}$, $\{\dot{y}(t)\}$ and $\{y(t)\}$ are acceleration, velocity and displacement vectors, respectively. And $\{y(t_n + \Delta t)\} = \{y(t_{n+1})\}$.

A step-by-step method was proposed by Newmark. The linear interpolation between the acceleration of the time t_n and that of the time t_{n+1} was used to represent the acceleration over the period between time t_n and time t_{n+1} , so:

$$\{\ddot{y}\}_{t_n \rightarrow t_{n+1}} = (1 - \gamma)\{\ddot{y}(t_n)\} + \gamma\{\ddot{y}(t_{n+1})\} \quad (\text{A10})$$

As a result, the velocity and displacement at time t_{n+1} can be expressed as:

$$\{\dot{y}(t_{n+1})\} = \{\dot{y}(t_n)\} + \Delta t[(1 - \gamma)\{\ddot{y}(t_n)\} + \gamma\{\ddot{y}(t_{n+1})\}] \quad (\text{A11})$$

$$\{y(t_{n+1})\} = \{y(t_n)\} + \Delta t\{\dot{y}(t_n)\} + \left(\frac{1}{2} - \beta\right)\Delta t^2\{\ddot{y}(t_n)\} + \beta\Delta t^2\{\ddot{y}(t_{n+1})\} \quad (\text{A12})$$

As can be seen, the parameter γ represents the contributions of $\{\ddot{y}(t_n)\}$ and $\{\ddot{y}(t_{n+1})\}$ to the acceleration within the time step, and the parameter β represents those to the displacement. It can be found that γ controls the artificial damping induced by this step-by-step method. With $\gamma = \frac{1}{2}$, if $\beta = 0$, it is an explicit Newmark method, i.e., central difference method. If $\beta = \frac{1}{6}$, it is linear acceleration method. If $\beta = \frac{1}{4}$, it is mean acceleration method.

The vibration equation for t_{n+1} can be expressed as:

$$[M]\{\ddot{y}(t_{n+1})\} + [C]\{\dot{y}(t_{n+1})\} + [K]\{y(t_{n+1})\} = \{P(t_{n+1})\} \quad (\text{A13})$$

From Equations (A11) and (A12), the expressions for structural velocity and acceleration at time t_{n+1} are:

$$\{\dot{y}(t_{n+1})\} = \frac{\gamma}{\beta\Delta t}(\{y(t_{n+1})\} - \{y(t_n)\}) + (1 - \frac{\gamma}{\beta})\{\dot{y}(t_n)\} + (1 - \frac{\gamma}{2\beta})\Delta t\{\ddot{y}(t_n)\} \quad (\text{A14})$$

$$\{\ddot{y}(t_{n+1})\} = \frac{1}{\beta\Delta t^2}(\{y(t_{n+1})\} - \{y(t_n)\}) + \frac{1}{\beta\Delta t}\{\dot{y}(t_n)\} + (1 - \frac{1}{2\beta})\{\ddot{y}(t_n)\} \quad (\text{A15})$$

They are then fed into Equation (A13) to obtain the following equation:

$$[K]^*\{y(t_{n+1})\} = \{P(t_{n+1})\}^* \quad (\text{A16})$$

In which, the $[K]^*$, $\{P(t_{n+1})\}^*$ can be expressed as:

$$[K]^* = [K] + \frac{1}{\beta\Delta t^2}[M] + \frac{\gamma}{\beta\Delta t}[C] \quad (\text{A17})$$

$$\begin{aligned} \{P(t_{n+1})\}^* = & \{P(t_{n+1})\} + [M]\left[\frac{1}{\beta\Delta t^2}\{y(t_n)\} + \frac{1}{\beta\Delta t}\{\dot{y}(t_n)\} + \left(\frac{1}{2\beta} - 1\right)\{\ddot{y}(t_n)\}\right] + \\ & [C]\left[\frac{\gamma}{\beta\Delta t}\{y(t_n)\} + \left(\frac{\gamma}{\beta} - 1\right)\{\dot{y}(t_n)\} + \left(\frac{\gamma}{2\beta} - 1\right)\Delta t\{\ddot{y}(t_n)\}\right] \end{aligned} \quad (\text{A18})$$

By solving Equation (A16), $\{y(t_{n+1})\}$ can be obtained. By feeding $\{y(t_{n+1})\}$ to Equations (A14) and (A15), $\{\dot{y}(t_{n+1})\}$ and $\{\ddot{y}(t_{n+1})\}$ can also be obtained. Based on iterative methods, when the parameters, the step length and the state of motion in the first step

are provided, the structure's state of motion at any time can be derived using the step-by-step method.

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