Estimating Wind-Induced Responses of the Sightseeing Tower in the West Taihu Lake, Changzhou

Xuping Zhu[1], Lingli Qin[2*], Wenhua Liu[2], Yaying Zhang[1], Li Jing[1], Kai Lou[1]

[1] Beijing Polytechnic College, Beijing 100042, China
[2] Beijing College of Finance and commerce, Beijing 100042, China

*Corresponding author: Lingli Qin, email: 46690595@qq.com

Abstract. The wind tunnel test was carried out for the Sightseeing Tower of the West Taihu Lake, Changzhou in this study, and the equivalent static wind loads of the Moon Tower were obtained at different wind angles. Frequency-domain analysis was performed to reveal the wind-induced response dynamics. The distribution diagram of unfavorable equivalent static wind loads was worked out to provide reference for the design of the main structure.

Keywords: wind tunnel test, wind-induced response, the sightseeing tower , cross-wind, along-wind, frequency-domain analysis

1 Introduction

With the development of modern material technologies and construction techniques, a growing number of high-rise buildings have emerged, with large span, flexibility, lightweight, low damping, and hence a greater sensitivity to wind[1]. For these buildings, wind load is an important design load. The damage and destruction of civil engineering structures caused by wind loads represent the major losses in a wind disaster. The extent of damage includes the destruction of roof, exterior wall systems, exterior glass coating and even the malfunction of the entire structure and lifeline facilities[2][3]. Therefore, studies of the impact of wind on structures are essential to engineering structural design and calculations[4].

2 An Overview of the Moon Tower

The Moon Tower of the West Taihu Lake is located in the Moon Bay area of the West Taihu Lake in Changzhou（Fig. 1）, Jiangsu Province. It is 88m in total height, and the base is 4.2m high and 33m in diameter; the main body of the tower is 65.8m high and 6.85m in diameter, and the glass sphere on the top is 18m in diameter. The sightseeing platform at the bottom is 60m in diameter and has an absolute elevation of 3.70m. The roof terrace has an absolute elevation of 7.9m, which is 3.53m higher than that of the highest water level that is statistically possible every 50 years.

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Fig. 1. Perspective view of the Moon Tower

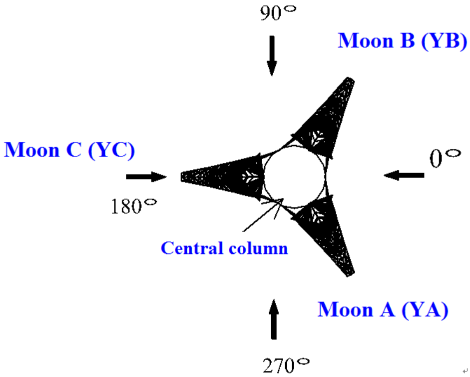


Fig. 2. Definition of wind angles

The structure of the Moon Tower of the West Taihu Lake has a large height-to-width ratio as well as low natural frequency and low damping[5]. The structural vibration caused by the fluctuating wind load cannot be ignored, thus giving rise to the necessity of the wind-induced response analysis[6]. However, the calculation methods for wind-induced vibration coefficient described in the current architectural design specifications are generally applicable to high-rise buildings with regular shapes[7]. Therefore, on the basis of wind-induced response estimation, the wind-induced vibration coefficients of the Moon Tower were given in this study to facilitate a reasonable, safe and reliable wind resistant design of the structure[8].

3 Estimation of Wind-Induced Responses and Wind-Induced Vibration Coefficients

3.1 Definition of Wind Angles, Design Wind Load and Wind Load Coefficient

The definition of wind angles in the estimation of wind-induced responses is shown in Fig. 2.

The design wind load is set as 0.45kN/m2, which is expected to occur every 100 years in Changzhou according to statistics.

3.2 Wind-Induced ResponsE at 0-Degree Wind Angle

Fig. 3 shows the fluctuating wind-induced response curves at 0-degree wind direction (X-axis direction) for the top floor of the sphere; Fig. 4 shows the contributions of vibration mode of each order to the wind-induced response of the structure.

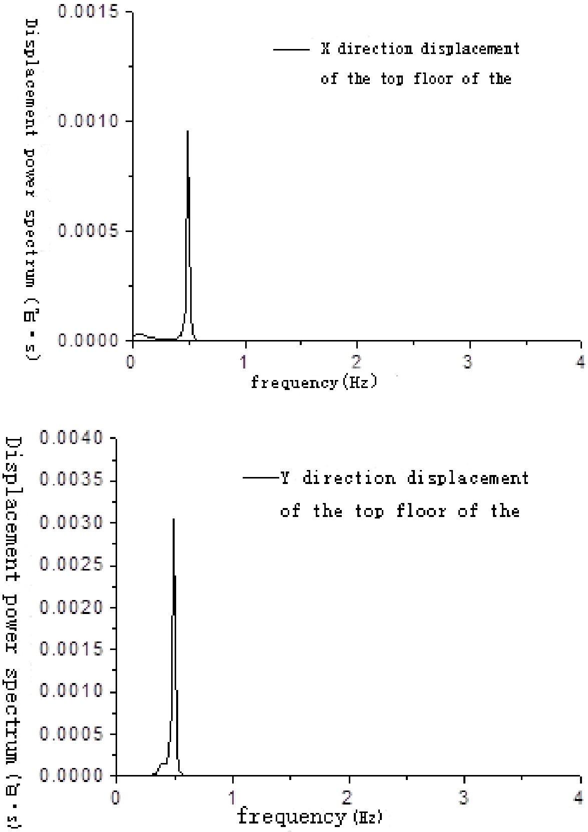


Fig. 3. Displacement response spectrum of the top floor under fluctuating wind

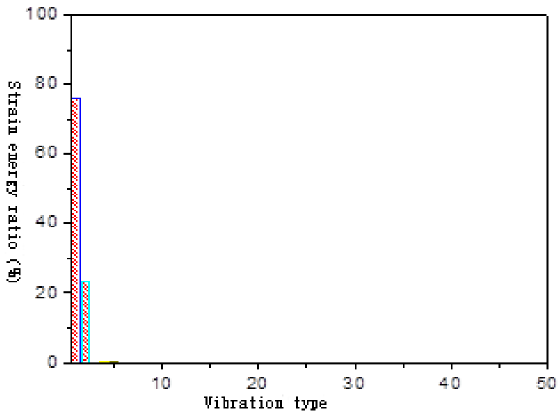
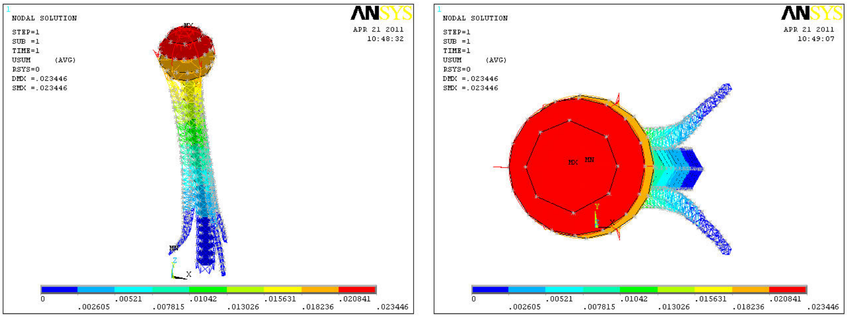


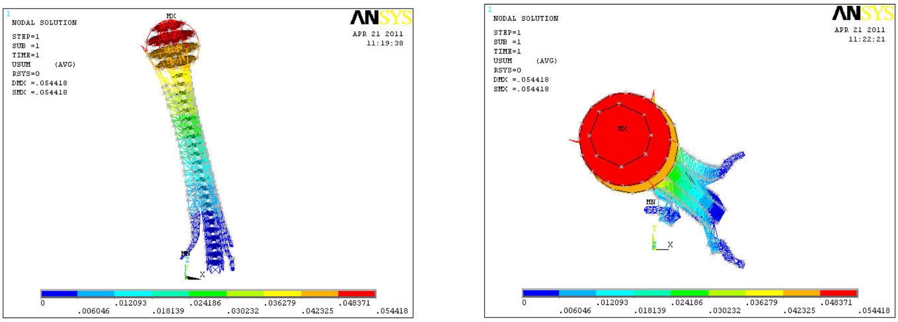
Fig. 4. Contribution of vibration mode of each order to the wind-induced strain energy

It can be seen that the first and second-order vibration modes make the greatest contribution to the structural response induced by fluctuating wind. The contribution of the first-order vibration mode is as high as 75%. The peak value appears near the frequency o 0.49Hz.

Fig. 5 shows the equivalent static wind load-induced response at 0 degree, as well as the maximum wind-induced response under the action of equivalent static wind load plus fluctuating wind. It can be seen that under the action of equivalent static wind wind load alone, the cross-wind displacement is very small; but when the fluctuating wind is considered, the cross-wind displacement is large.



Equivalent static wind load-induced response at 0 degree



Maximum wind load-induced vibration response at 0 degree

Fig. 5. Wind-induced vibration response at 0 degree

3.3 Wind-Induced Responses of The Structure at All Wind Angles

The wind loading deformation diagrams of the structure under the corresponding maximum wind-induced response at different wind angles are shown below.

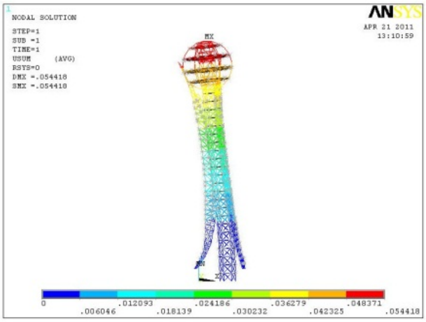


Diagram of maximum deformation at 0 degree

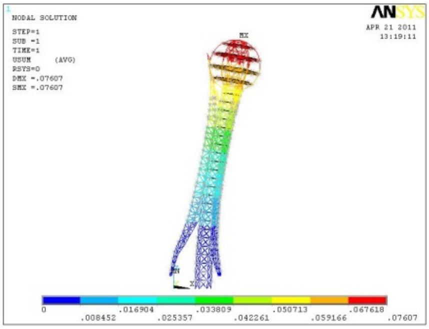


Diagram of maximum deformation at 90 degrees

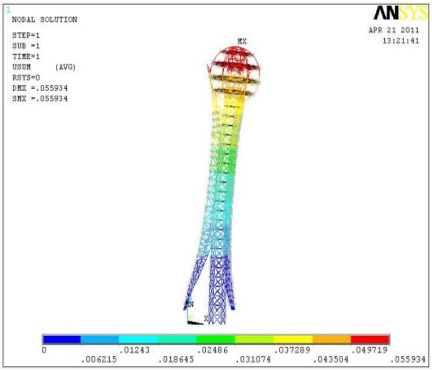


Diagram of maximum deformation at 120 degrees

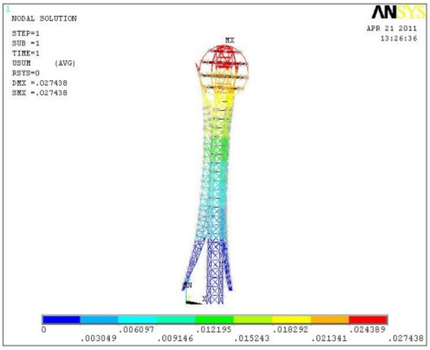


Diagram of maximum deformation at 180 degrees

Fig. 6. Maximum deformations at four representative wind angles

The maximum displacements along X (0 degree) and Y (90 degree) directions at different wind angles are given in Table 1.

Table 1. Moon Tower's maximum wind-induced displacements at different wind angles

Wind angle	X-axis Direction		Y -axis Direction		Resultant Displacement
	Average wind-induced dis-placement (Unit: m)	Maximum wind-induced displacement (Unit: m)	Average wind-induced displacement (Unit: m)	Maximum wind-induced displacement (Unit: m)	$\sqrt{X^2 + Y^2}$ (Unit: m)
0 degree	-0.0229	-0.0412	0.0006	0.0336	0.0531
15 degrees	-0.0235	-0.0405	0.0017	0.0443	0.0600
30 degrees	-0.0235	-0.0421	0.0052	0.0427	0.0599
45	-0.0168	-0.0306	-0.0035	-0.0192	0.0361

degrees					
60 degrees	-0.0067	-0.0203	-0.0107	-0.0245	0.0318
75 degrees	0.0055	0.0221	-0.0148	-0.0284	0.0359
90 degrees	0.0129	0.0636	-0.0172	-0.0394	0.0748
105 degrees	0.0108	0.0450	-0.0197	-0.0395	0.0599
120 degrees	0.0100	0.0359	-0.0195	-0.0411	0.0546
135 degrees	0.0099	0.0332	-0.0206	-0.0504	0.0604
150 degrees	0.0076	0.0325	-0.0223	-0.0631	0.0709
165 degrees	0.0096	0.0182	-0.0117	-0.0320	0.0368
180 degrees	0.0106	0.0181	0.0010	0.0198	0.0268
195 degrees	0.0091	0.0181	0.0129	0.0313	0.0362
210 degrees	0.0074	0.0283	0.0204	0.0669	0.0727
225 degrees	0.0100	0.0298	0.0191	0.0572	0.0645
240 degrees	0.0107	0.0317	0.0190	0.0430	0.0534
255 degrees	0.0115	0.0487	0.0188	0.0414	0.0639
270 degrees	0.0144	0.0552	0.0164	0.0379	0.0670
285 degrees	0.0053	0.0224	0.0144	0.0254	0.0339
300 degrees	-0.0061	-0.0214	0.0088	0.0212	0.0301
315 degrees	-0.0157	-0.0322	0.0011	0.0145	0.0353
330 degrees	-0.0220	-0.0486	-0.0042	-0.0477	0.0681
345 degrees	-0.0224	-0.0427	-0.0005	-0.0329	0.0539

Note: In $\sqrt{X^2+Y^2}$, X and Y represent the wind-induced displacements in X-axis and Y-axis directions, respectively.

As shown in Table 1, the winds with the angles of 90 degrees, 210 degrees and 330 degrees are the most unfavorable to the structure.

The three moons are arranged evenly around the center of the Moon Tower at an interval of 120 degrees. Therefore, the peak values of equivalent static wind loads and wind-induced structural responses occur also at an interval of 120 degrees. According to rotational symmetry, it can be known that the most unfavorable wind angle of 0 degree is equivalent to 120 degrees and 240 degrees.

3.4 Wind-Induced Vibration Coefficients and Equivalent Static Wind Loads in Along-Wind Direction

The wind-induced vibration coefficients of nodal displacement of the Moon Tower were calculated according to the results of the wind-induced response analysis in 3.3 and the equation (22).

Fig. 7 shows the wind-induced vibration coefficients of nodal displacement in the along-wind direction at 0 degree (i.e. X-axis direction), and Fig. 8 shows the wind-induced vibration coefficients of nodal displacement in the along-wind direction at 90 degrees (i.e. Y-axis direction).

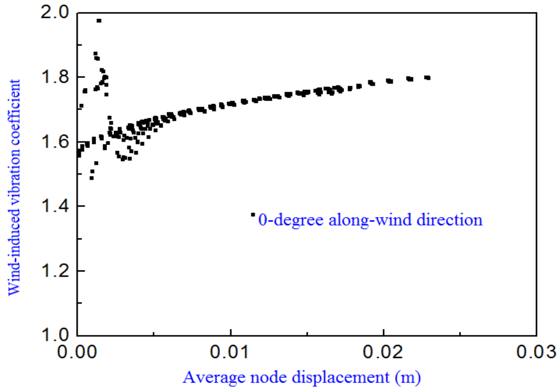


Fig. 7. Wind-induced vibration coefficients in the along-wind direction at 0 degree

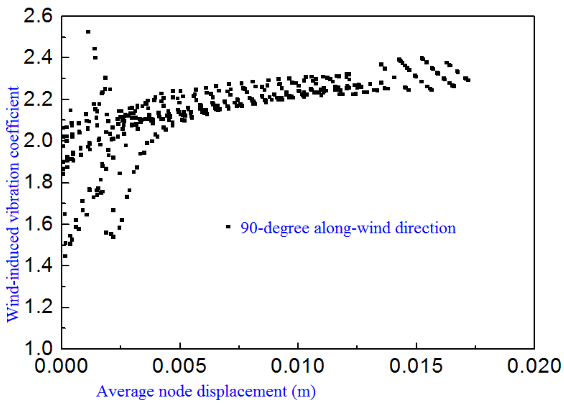


Fig. 8. Wind-induced vibration coefficients in the along-wind direction at 90 degrees

As shown in the figures, the wind-induced vibration coefficients at the nodes with larger wind-induced response amplitude are similar, and these nodes play a critical role in structural design[9]. At 0 degree, the wind-induced vibration coefficient of the entire structure can be 1.80, while at 90 degrees the wind-induced vibration coefficient of the entire structure is 2.4.

Then according to rotational symmetry and the above analysis results, the along-wind vibration coefficients at some typical angles can be obtained:

(1) At 0, 120 and 240 degrees, the along-wind vibration coefficient of the structure is $\beta = 1.8$;

(2) At 90, 210 and 330 degrees, the along-wind vibration coefficient of the structure is $\beta = 2.4$.

The along-wind vibration coefficients of the structure at other angles can be obtained through linear interpolation..

In actual practice of structural design, the standard value of along-wind equivalent static wind load of the main structure is calculated as follows:

$$w_j^{\max} = \beta \cdot \mu_{z,88m} \cdot \overline{C_j^k} \cdot w_0 \tag{1}$$

In the formula, β is the along-wind vibration coefficient of the structure; w_0 is the design wind load value in Changzhou, which is 0.45kPa, as determined according to the occurrence frequency of once every 100 years; $\mu_{z,88m}$ is the height variation coefficient of wind load at the top of the Moon Tower (at 88m height), and $\mu_{z,88m} = 2.26$ according to the landform; $\overline{C_j^k}$ is the average block wind load coefficient at the top of the Moon Tower.

3.5 Wind-Induced Vibration Coefficients and Equivalent Static Wind Loads in Cross-Wind Direction

The analysis results of wind-induced vibration responses at all wind angles show that the displacement caused by the equivalent static wind load of the structure in the cross-wind direction are very small, while the fluctuating wind-induced response and extreme response are large. In such case, it is not suitable to use the wind-induced vibration coefficient to express cross-wind equivalent static wind load, since the average equivalent static wind load is 0, which is mainly the equivalent component corresponding to fluctuating wind load.

The cross-wind vibration response spectrum at the top of the structure at 0 degree shows that a significant peak appears at the natural frequency is 0.49Hz. This indicates that the inertial force of the structure controls the cross-wind vibration. In other words, the first-order vibration mode plays the dominant role. Therefore, the cross-wind equivalent static wind load can be expressed through the inertial force of the cross-wind induced vibration of first-order mode as follows:

$$\{F\}_e^h = a \cdot \{\omega_1^2 [M]\{\psi\}_1\} \tag{2}$$

In the formula, $[M]$ and $\{\psi\}_1$ represent the mass matrix and the first-order mode function, respectively; $\{\omega_1^2 [M]\{\psi\}_1\}$ represents the shape function for the inertial force of the first-order mode, and a is the proportionality coefficient.

Under the action of cross-wind equivalent static wind load $\{F\}_e^h$, the cross-wind displacement of the structure is the same as the analysis results in 3.3, according to which the proportionality coefficient in the formula (2) can be calculated.

According to the analysis results in the above section, the wind at 90 degrees is most unfavorable for the structure. In this paper, cross-wind equivalent static wind loads at 0 and 90 degrees are mainly analyzed to benefit the structural design. The cross-wind equivalent static wind loads can be worked out by integrating the equivalent wind loads at all nodes of each floor obtained through formula (2).

(1) Cross-wind equivalent static wind load at 0 degree

At 0 degree, as shown in Table 1, the maximum cross-wind displacement is 0.0336m, and the cross-wind equivalent static wind load (i.e. the Y-axis direction) can be obtained according to formula (2) (as shown in Fig. 9).

Fig. 10 shows the comparison between the responses of the Moon Tower at all heights to the equivalent static wind load and the numerical value in cross-wind direction. It can be seen that the two are in good agreement.

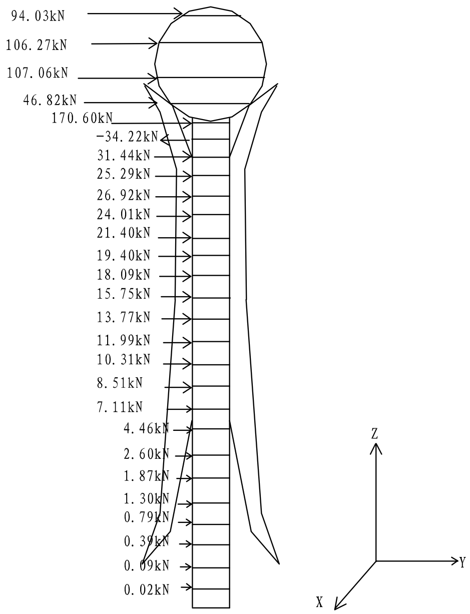


Fig. 9. Cross-wind equivalent static wind load at 0 degree

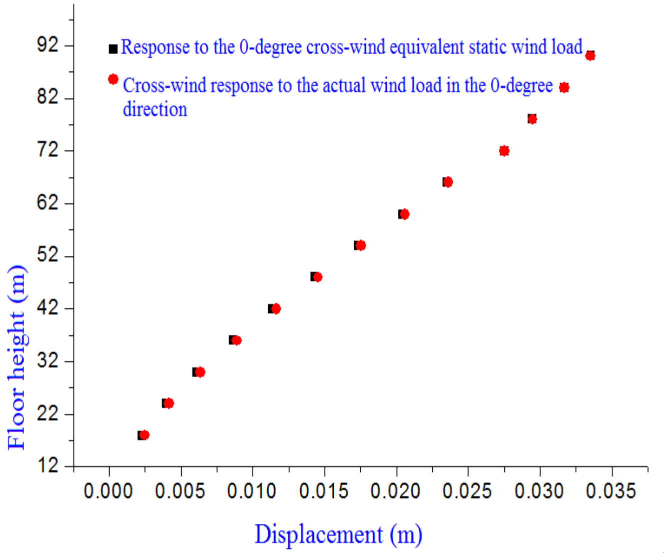


Fig. 10. Comparison between the response to the equivalent static wind load and the actual wind-induced response at 0 degree

(2) Cross-wind equivalent static wind load at 90 degrees

At 90 degree, as shown in Table 1, the maximum cross-wind displacement is 0.0636m, and the cross-wind equivalent static wind load (i.e. the X-axis direction) can be obtained accordingly (as shown in Fig. 11).

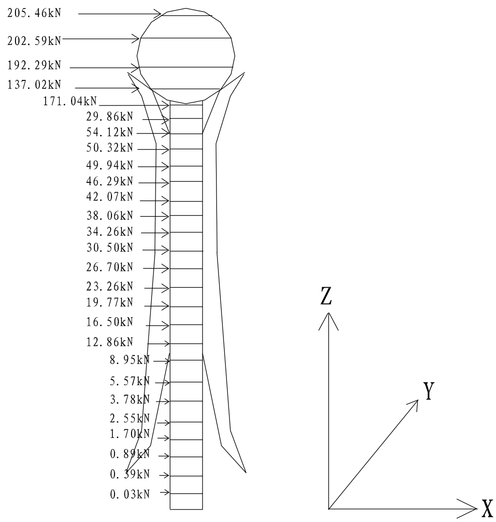


Fig. 11. Cross-wind equivalent static wind load at 90 degrees

Fig. 12 shows the comparison between the responses of the Moon Tower at all heights to the equivalent static wind load and the extreme cross-wind vibration response under the actual wind loads. The two are in good agreement.

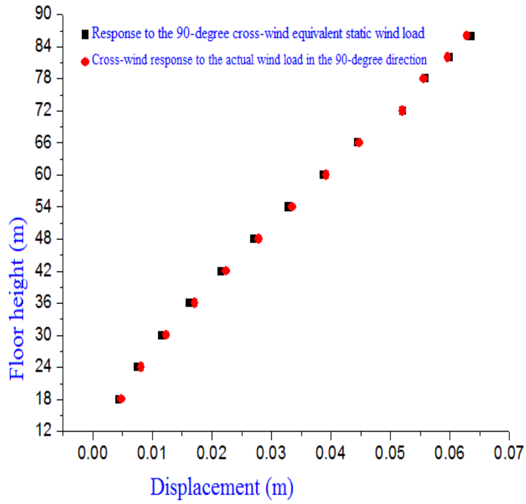


Fig. 12. Comparison between the response to the cross-wind equivalent static wind load and the actual wind-induced response at 90 degrees

4 Conclusions

Based on the wind tunnel test, the wind load pressure duration of the Moon Tower is obtained. Frequency-domain analysis is adopted to analyze wind-induced response dynamics and equivalent static wind loads of the Moon Tower on the West Taihu Lake at different wind directions. The unfavorable equivalent static wind load distribution is worked out for the design of the main structure. The main conclusions and suggestions are as follows:

(1) According to the analysis of wind-induced vibration, the winds at 90 degrees, 210 degrees and 330 degrees are most unfavorable to the structure, and these three wind directions are symmetric around the three axes of the center;

(2) The along-wind vibration coefficient of the entire structure at 0 degree is 1.8, while that at 90 degrees is 2.4;

(3) The cross-wind equivalent static wind loads at the two typical unfavorable wind angles of 0 and 90 degrees are given, and the extreme values of actual dynamic response are in good agreement with the responses to the equivalent static loads;

(4) In actual design, along-wind equivalent static wind loads and cross-wind equivalent static wind loads should be taken into consideration, and the effects of cross-wind and along-wind responses should be combined according to the provisions of Clause 8.5.6 of Load Code for the Design of Building Structures (2012). That

is, $S = \sqrt{(0.6 S_A)^2 + S_C^2}$, where S_A and S_C represent along-wind and cross-wind wind load effects, respectively.

This study evaluates wind-induced responses of the Moon Tower in Changzhou's West Taihu Lake area, using wind tunnel testing and frequency-domain analysis to determine wind load effects at various angles. The findings identify critical wind directions and provide key insights into structural design adjustments necessary to withstand adverse wind conditions, improving the safety and resilience of the tower.

5 Summary

Specifically, subsequent research includes comparative analysis of the wind-induced response of the Moon Tower and similar high-rise structures, which can be analyzed and studied from the following aspects:

1. Structural dynamic characteristics: Compare the differences in dynamic characteristics such as natural frequency, damping ratio, and vibration mode between high-rise tower structures and high-rise buildings under wind loads.

2. Wind induced response: Analyze the differences in base shear, displacement, acceleration, and other responses between the two under wind loads, as well as the impact of these differences on structural safety and functional use.

3. Understanding and application of critical wind angle:

-Recognize the importance of critical wind angle and fully consider this angle during the design phase. Designing within the critical wind angle can minimize the adverse effects of wind loads on the structure, thereby reducing displacement caused by wind.

4. Increase structural stability:

-Add structural stability in the design, such as by using higher stiffness structural elements or adding additional support structures to resist the effects of lateral wind loads.

The limitations of using equivalent static wind loads in crosswind analysis for towering iron towers are mainly reflected in the following aspects:

1. Simplified assumptions: The method of equivalent static wind load is based on certain simplified assumptions, such as equating pulsating wind load with static wind load, which ignores the temporal variation characteristics of wind load, especially when considering torsional wind load, this simplification is particularly inadequate.

2. Insufficient consideration of local wind field: The method of equivalent static wind load may not fully consider the complex local wind field effects, such as the occlusion effect between buildings and the influence of terrain, which may lead to additional uncertainty in the actual wind field.

3. Simplification of frequency and mode response: In frequency domain analysis, only considering one or a few order modes may overlook the influence of higher-order modes on wind load and structural response, especially for structures with high flexibility, whose response under wind load may be mainly determined by higher-order modes.

Continuous monitoring of wind-induced response of towering iron towers under real-world conditions is crucial for the continuous evaluation of structural safety, for the following reasons:

1. Real time performance: Continuous monitoring can provide real-time data about high-rise iron towers in actual operating environments, including various parameters such as vibration, displacement, stress, etc. These data can help engineers understand the response status of the structure in real time, so as to detect anomalies and take corresponding measures in the first time.

2. Long term: Through continuous monitoring of towering iron towers, a large amount of historical data can be obtained for analysis. These historical data can help researchers understand the long-term performance of structures under different environmental conditions, evaluate their safety performance, and predict possible degradation trends.

3. Preventive maintenance: Continuous monitoring data can not only be used for monitoring and evaluation, but also assist in preventive maintenance. By analyzing these data, potential structural issues can be predicted and prevented, thereby avoiding accidents.

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