Analysis of wind-resistant and stability for cable tower in cable-stayed bridge with four towers

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Abstract. Wind speed time history simulation methods have been introduced first, especially the harmonic synthesis method introduced in detail. Second, taking Chishi bridge for example, choosing the particular sections, and combined with the design wind speed, three-component coefficient simulate analysis between -4°and 4°has been carry out with the Fluent software. The results show that drag coefficient reaches maximum when the attack Angle is 1°. According to measured wind speed samples, time history curves of wind speed at bridge deck and tower roof have been obtained, and wind-resistant time history analysis for No.5 tower has been carry out. Their results show that the dynamic coefficients are different with different calculation standard, especially transverse bending moment, pulsating crosswind load does not show a dynamic amplification effect. Under pulsating wind loads at bridge deck or tower roof, the maximum displacement at the top of the tower and the maximum stress at the bottom of the tower are within the allowable range. The transverse stiffness of tower is greater than that of the longitudinal stiffness, therefore wind-resistant analysis should give priority to the longitudinal direction. Dynamic coefficients are different with different standard, the maximum dynamic coefficient should be used for the pseudo-static analysis. Finally, the static stability of tower is analyzed with different load combinations, and the galloping stabilities of cable tower is proved.

1 Introduction
Recently, the long span bridges especially for the cable-stayed bridges with multi-tower take a more important role in highway bridges. How to design a reasonable wind-resistant structure become a hot issue of engineering design[2]. Until now, the research on wind resistance of main girder in long span bridges is more than ever, by contrast, the research on wind resistance of four tower cable-stayed bridge and its stability is less.

In the past, complex bridge structures are often designed by wind tunnel tests, which can not obtain all information needed for the design and the model parameters are not easy to modify. So, combined Chishi bridge’s wind speed samples on site, wind speed time history is simulated by numerical analysis. Combining with Fluent software, Three-component coefficient of wind-resistant for cable tower and its stability are analyzed.

2 Summarization of Wind Speed Time History Simulation Methods
A wind speed process is a stochastic process in nature, and its load also is a stochastic process. Under random wind loads, the static effects and dynamic effects of structures (such as chattering, vortex and galloping, flutter) are produced.

At present, there are two kinds of methods for wind speed time history simulation that are the linear filter method and the harmonic synthesis method. Linear filter method that is a kind of stable
simulation method includes AR model (autoregressive model) and ARMA model (autoregressive moving average model). The principle is that through appropriate mathematic transform, white noise random sequences with a mean value of zero is converted into a stochastic process with spectral features. Harmonic synthesis method that is a kind of unconditional stability of high precision simulation method classifies single point simulation and multi-point simulation according to the number of simulation point. The principle is that the wind speed process is simulated as a sum of a series of cosine functions[1].

The harmonic synthesis method is used to simulate the random wind field, and its main principles are summarized as follows.

The single point stationary random process \( u(t) \) can be simulated by formula (1).

\[
u(t) = \sqrt{2} \sum_{n=0}^{N-1} A_n \cos(\omega_n t + \phi_n)
\]

Where \( S(\omega) \) is a bilateral power spectrum density function and \( \omega_n \) is the truncation frequency. \( A_n = 2S(\omega_n)\Delta\omega \), \( \omega_n = n\Delta\omega \), \( n=0,1,2,\ldots,N-1 \), \( \Delta\omega = \omega_N / N \), \( A_0 = 0 \), or \( S(\omega_n=0)=0 \).

\( \phi_n, \phi_1, \phi_2, \ldots, \phi_{N-1} \) in uniform distribution \([0, 2\pi]\), independent respectively.

Based on the Shinozuka theory[10], multi-point stationary random process \( u(t) \) can be simulated by formula (2).

\[
u_j(t) = \sqrt{2\Delta\omega} \sum_{m=1}^{M} \sum_{l=1}^{L} H_{jm}(\omega_{ml}) \cos[\omega_{ml}t - \theta_{jm}(\omega_{ml}) + \phi_{ml}], (j=1, 2, \ldots, n)
\]

Where \( H_{jm}(\omega_{ml}) \) is the cholesky decomposition of power spectral density matrix and Lower triangular matrix, \( \omega_{ml} \) is the circular frequency, \( \omega_{ml} = (l-1)\Delta\omega + \frac{m}{n} \Delta\omega, l=1,2,\ldots,N \). \( \theta_{jm}(\omega_{ml}) = \arctan \left[ \frac{\text{Im}[H_{jm}(\omega_{ml})]}{\text{Re}[H_{jm}(\omega_{ml})]} \right] \), \( \phi_{ml} \) in uniform distribution \([0, 2\pi]\), independent respectively[3]. The rest of symbolics meaning are of the same as formula (1).

3 Three-Component Coefficient Identification Of Cable Tower In Chishi Bridge

3.1 Project summary

The main bridge of Chishi bridge is a long span prestressed concrete cable-stayed bridge with four towers. The span arrangement is 165m+3×380m+165m with side tower support and middle tower fixed, the ratio of side span to middle span is 0.4342. All towers are hyperbolic reinforced concrete towers. The total width of bridge deck is 28.0m, it is about 180m above the caps. 23 pairs of stayed cable are arranged in main towers with a vertical fan-shaped layout.

Elevation of Chishi bridge is shown in Figure 1.

Figure 1. elevation of Chishi bridge
In order to effectively analyze the effects of pulsating wind load in Chishi bridge, No.5 tower is chosen for analyzed model, its finite element model is shown in Figure 2. The first four order frequency and vibration mode of the tower are shown in Figure 3.

![Figure 2. No.5 tower's finite element model](image)

a. longitudinal bending, $f=0.1732 \text{ Hz}$

b. transverse bending, $f=0.2744 \text{Hz}$
3.2 Three-component coefficient identification

Selecting half height section and top section of the tower\textsuperscript{[8-9]}, respectively, combined with fluent fluid analysis software and the design wind speed, three-component coefficient identification analysis of the two sections are made within the scope of attack angle between - 4° and 4°. Calculation model is shown in Figure 4, the results are shown in Table 1 and Figure 5.

* boundary conditions: left and right ends with far field pressure, top and bottom interface with symmetry, internal section of cable tower with solid wall boundary. Reynolds number is \( Re = \rho UL/\mu = 2.9 \times 10^7 \), thus the turbulence model is used to calculate.

Figure 3. The first four order frequency and vibration mode for No.5 tower

Figure 4. calculation model
Table 1. three-component coefficient change with attack Angle

| attack angle | Cd      | Cl      | Cm      |
|--------------|---------|---------|---------|
| -4           | 1.1701595 | 0.079633455 | -0.000367419 |
| -3           | 1.1753549 | 0.061504564 | -0.000641703 |
| -2           | 1.1793528 | 0.043114871 | -0.000911928 |
| -1           | 1.182162  | 0.024526 | -0.001186 |
| 0            | 1.183309  | 0.005869 | -0.001462 |
| 1            | 1.184142  | -0.012701315 | -0.001745761 |
| 2            | 1.1832955 | -0.031357045 | -0.002021972 |
| 3            | 1.180009  | -0.049816399 | -0.002300521 |
| 4            | 1.1780349 | -0.068115789 | -0.002577028 |

Figure 5. diagram of three-component coefficient change with attack Angle

With the change of attack angle from -4° to 4°, the drag coefficient increases first and then decreases (Figure 1), when the attack angle is 1°, the maximum drag coefficient is 1.184142 (Table 1). The lift coefficient and moment coefficient decrease with the increase of attack angle (Figure 1), and both are in linear relationship with the attack Angle.

4 Wind-Resistant Time History Analysis
Chishi bridge that belongs to RuChen expressway is located between Xiaou village and Yuxi village of Chishi town Yizhang county, which is on the terrace and flood land. According to Table 3.2.2 in “Wind-resistant Design Specification for Highway Bridges” (JTG/T D60-01-2004), the surface classification is B, surface roughness coefficient $\alpha$ is 0.16, rough height $Z_0$ is 0.05, the design wind speed $V_{10}$ is 24.1 meters per second. The power spectral density function (Kaimal spectrum) is calculated by formula 3 and formula 4 for bridge deck and tower roof, respectively.

\[
S_u(n)_{180} = \frac{3199.537}{(1+241.629n)^{5/3}}
\]

\[
S_u(n)_{280} = \frac{4722.261}{(1+356.625n)^{5/3}}
\]

Where $n$ is the wind pulse frequency.

Three dimensional ultrasonic anemometers are installed on the top of tower and bridge deck, respectively. Collected data for each instrument is verified, so, it is effective and reliable. Combined with the data collected by site devices, analysis shows that the average wind speed $U$ is 25.9 meters per second and its main wind direction Angle is $\Phi$ is 13.1 ° NNE.

The wind speed frequency curve of power spectral density $S_u(n)$ is fitting as shown in Figure 6. Simulation of wind speed at bridge deck and the top of tower are as shown in Figure 7.

![Power spectral density-frequency curve Su(n)-n](image-url)

Figure 6. Power spectral density-frequency curve Su(n)-n
For the wind time history analysis, we must convert wind speed to wind load first, then put the load on the corresponding position of the bridge. According to “Wind-resistant Design Specification for Highway Bridges” (JTG/T D60-01-2004), the conversion formula (5) is shown as follows.

$$ F_H = \frac{1}{2} \rho V^2 C_H A_n $$  

(5)

Where $C_H$ is the resistance coefficient of each bridge component, the maximum value of simulation analysis is used to calculate. $A_n$ is a projected area of each bridge component along the wind, for suspenders and stayed cables and main cables of the suspension bridge, is multiplied by the projected height and its diameter\(^5\).

Under the pulsating longitudinal wind load at bridge deck, the longitudinal displacement-time curve at the top of tower is shown in Figure 8, the longitudinal bending moment-time curve at the bottom of tower is shown in Figure 9. Under the pulsating crosswind load at bridge deck, the lateral displacement-time curve is shown in Figure 10, the transverse bending moment-time curve is shown in Figure 11.
Under the same pulsation wind load, time history curves of displacement and bending moment are similar in shape (Figure 8-10), which shows that the tower’s dynamic response is within the elastic range.

Under the pulsating longitudinal wind load at bridge deck, the maximum longitudinal displacement at the top of tower is 1.921mm, the maximum longitudinal bending moment at the bottom of tower is 10570kN.m. Under pulsating crosswind load at bridge deck, the maximum lateral displacement at the top of tower is 0.508mm, the maximum transverse bending moment at the bottom of tower is 6516kN.m.

Under the maximum equivalent static wind loads, the maximum displacement at the top of tower and the maximum bending moment at the bottom of tower and their dynamic coefficients are shown in Table 2, respectively. From the Table, using different calculation standard, dynamic coefficients are different, especially the transverse bending moment, pulsating crosswind load does not show a dynamic amplification effect.
Table 2. List of time history analysis results

|                          | Pulsation wind loads | Maximum equivalent static wind loads |
|--------------------------|-----------------------|--------------------------------------|
|                          | Displacement/mm       | Bending moment/kN.m                   |
|                          |                       | Displacement/mm                       | Bending moment/kN.m | Dynamic coefficient |
| Along the bridge         | 1.921                 | 10570                                | 1.021              | 6594.7              | 1.881 | 1.603 |
| Cross the bridge         | 0.508                 | 6516                                 | 0.371              | 6594.7              | 1.369 | 0.988 |

* Under the maximum wind speed equivalent static wind load, the tower’s stress at the bottom is 0.74kPa (along the bridge) and 0.62kPa (cross the bridge), respectively.

Under pulsating longitudinal wind load at the top of tower, the longitudinal displacement-time curve at the top of tower is shown in Figure 12, the longitudinal bending moment-time curve at the bottom of tower is shown in Figure 13. Under the pulsating crosswind load at the top of tower, the lateral displacement-time curve is shown in Figure 14, the transverse bending moment-time curve is shown in Figure 15.
Under pulsating longitudinal wind load at the top of tower, the maximum longitudinal displacement at the top of tower is 5.428mm (Figure 12), the maximum longitudinal bending moment at the bottom of tower is 29860kN.m (Figure 13). Under pulsating crosswind load at the top of tower, the maximum lateral displacement is 1.729mm (Figure 14), the maximum transverse bending moment is 22160kN.m (Figure 15).

Under the maximum equivalent static wind loads, the maximum displacement at the top of tower and the maximum bending moment at the bottom of tower and their dynamic coefficients are shown in Table 3, respectively.

Table 3. list of time history analysis results

|                  | Pulsatation wind load | Maximum equivalent static wind loads |
|------------------|-----------------------|--------------------------------------|
|                  | Displacement/mm       | Bending moment/kN.m                  | Displacement/mm | Bending moment/kN.m | Dynamic coefficient |
| Along the bridge | 5.428                 | 29860                               | 3.01            | 12990.5             | 1.803                |
| Cross the bridge | 1.729                 | 22160                               | 1.606           | 12990.5             | 1.077                |

* Under the maximum wind speed equivalent static wind loads, the tower’s stress at the bottom is 1.46kPa (along the bridge) and 1.22kPa (cross the bridge), respectively.

Considering pulsating wind load along the bridge and cross the bridge simultaneously, the displacement-time curve is shown in Figure 16, the bending moment-time curve is shown in Figure 17. Thus, The maximum displacement at the top of tower is 7.348mm (Figure 16), it is 4.031mm under the maximum wind speed equivalent static wind loads. The maximum bending moment at the bottom of tower is 19585.2kN.m (Figure 17) and its stress is 2.20kPa, their dynamic coefficients are 1.823 and 2.064, respectively.
Figure 17. Bending moment-time history curve at the bottom of tower

It is easy to conclude that, under pulsating wind loads on bridge deck or tower roof, the maximum displacement at the top of tower and the maximum stress at the bottom of tower are within the allowable range.

Analysis shows that the transverse stiffness of tower is greater than that of the longitudinal stiffness (transverse dynamic coefficient less than that of longitudinal direction). So, the wind resistance analysis should give priority to the longitudinal direction. Dynamic coefficients are different with different standard, the maximum dynamic coefficient should be used for the pseudo-static analysis.

5 Wind-Resistant Stability Analysis

5.1 The static stability

In order to study the wind stability of a single tower structure, No.5 tower of Chishi bridge is chosen to analyzed. For a independent tower structure, selecting bridge deck and tower roof for wind-resistant static stability calculation section[6], using different load combinations, the stability factor are simulated.

Static wind-resistant stability analysis are carried out with MIDAS software[7], its detailed calculation results are shown in Table 4.

Table 4. Static stability calculation results for No.5 cable tower

| load combination | variable load | critical buckling coefficient | Buckling type |
|------------------|---------------|------------------------------|---------------|
| W_t             | W_t           | 1.675                        | longitudinal bending |
| W_t+F_bs        | F_bs          | 3.606×10^5                  | longitudinal bending |
| W_t+F_bh        | F_bh          | 3.213×10^5                  | longitudinal bending |
| W_t+W_b+F_bs    | F_bs          | 3.597×10^5                  | longitudinal bending |
| W_t+W_b+F_bh    | F_bh          | 3.179×10^5                  | longitudinal bending |
| W_t+W_b         | W_b           | 39.33                       | longitudinal bending |
| W_t+W_b+M       | M             | 5.070×10^6 kN.m             | longitudinal bending |
| W_t+F_ts        | F_ts          | 7.0×10^4                    | longitudinal bending |
| W_t+F_th        | F_th          | 7.127×10^4                  | longitudinal bending |
| W_t+W_b+F_ts    | F_ts          | 6.999×10^4                  | longitudinal bending |
| W_t+W_b+F_th    | F_th          | 7.087×10^4                  | longitudinal bending |

* W_t - cable tower weight, W_b - beam weight, F_ts - longitudinal wind load at the top of tower, F_th - crosswind load at the top of tower, F bs - longitudinal wind load at bridge deck; F bh - crosswind load at bridge deck, M - unbalanced bending moment.
From Table 4, under the cable tower’s self weight, when it reached 1.675 times the weight, a longitudinal bending buckling will occur. In the same load combinations, compared considering self weight of main girder and cable tower with considering cable tower’s self weight only, both critical buckling coefficient have little difference. The critical buckling coefficient under self weight of main girder and cable tower is relatively smaller, its minimum value is 69990, far more than the standard requirements, the structure is safe. From the whole analysis, the critical buckling coefficient under the wind load at bridge deck is generally greater than the coefficient under the wind load at the top of tower, which shows that the tower structure is more stable under the wind load at the top of tower.

5.2 Galloping stability
According to “Wind-resistant Design Specification for Highway Bridges” (JTG/T D60-01-2004), the aspect ratio B/H<4 cable-stayed bridge with steel tower should check its galloping stability of independent state.

The test results show that the galloping may occur mainly in the blunt cross section steel bridges and steel towers. Due to this bridge uses concrete cable tower, damping is bigger, the galloping critical wind speed is higher. According to “Wind-resistant Design Specification for Highway Bridges” (JTG/T D60-01-2004), by calculation, \( C_L+C_{1d}=1.165>0 \) galloping stability of cable tower meets the requirements.

6 Conclusions
Combined with Fluent software and design wind speed, selecting half height section and top section of the No.5 tower, three-component coefficients identification analysis of the tower are made within the scope of attack angle between -4° and 4°. Analysis results show that, with the change of attack angle from -4°to 4°, the drag coefficient increases first and then decreases, when the attack angle is 1°, the maximum drag coefficient is 1.184142. The lift coefficient and moment coefficient decrease with the increase of attack angle, and both are in linear relationship with the attack Angle.

According to the wind speed samples collected on site, the wind velocity time history curve is obtained.

The wind-resistant time history analysis of No.5 tower shows that using different calculation standard, dynamic coefficients are different, especially the transverse bending moment, pulsating crosswind load does not show a dynamic amplification effect. Under pulsating wind loads at the bridge deck or the top of tower, the maximum displacement at the top of tower and the maximum stress at the bottom of tower are within the allowable range.

Transverse stiffness of the tower is greater than that of the longitudinal stiffness, therefore wind-resistant analysis should give priority to the longitudinal direction. Dynamic coefficients are different with different standard, the maximum dynamic coefficient should be used for the pseudo-static analysis.

The static stability of tower analysis shows that, under the same load combinations, compared considering self weight of main girder and cable tower with considering cable tower self weight only, both critical buckling coefficient have little difference. The critical buckling coefficient under self weight of main girder and tower is relatively smaller, its minimum value is 69990, far more than the standard requirements, the structure is safe.

From the whole analysis, the critical buckling coefficient under the wind load at bridge deck is generally greater than the coefficient under the wind load at the top of tower, that shows that the tower structure is more stable under the wind load at the top of tower.

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