Wind-induced Buffeting and Vibration Reduction Control Design of Qingshan Extra Large Span Cable-Stayed Bridge

Lingjun Kong, Xiangliang Ning and Huaishe Ruan

ABSTRACT
In order to reduce the maximum cantilever construction state and completed bridge state buffeting effect of Qingshan extra large span cable-stayed bridge, the vibration reduction control design of the bridge is carried out. Through the numerical calculation and wind tunnel test, the wind-induced response of the bridge is obtained, under the maximum cantilever construction state and completed bridge state. The vibration reduction control design of the two states is conducted. And the bridge comfort index is determined. The results show that the buffeting response of the main girder is larger under the maximum cantilever construction and the completed bridge state. MTMD is used to design the vibration reduction control of the bridge, and the MTMD reducing effect is obvious. After the vibration reduction control design, the bridge can meet the construction and bridge safety requirements.

INTRODUCTION
Wind exists in nature all the time, and is closely related to human society. Mankind's understanding and use of the wind have contributed greatly to the development of human society, but frequent and strong winds also brought us a great disaster. The vibration response of bridge will increase the internal force of the bridge and may cause the local fatigue damage of the bridge, and even cause the bridge to completely destroy, due to the action of the wind. The effect of wind on the bridge is a more complicated aerodynamic phenomenon, which is affected by dynamic characteristics of the bridge, natural characteristics of the wind and interaction between the wind and the bridge. The buffeting of bridge was proposed by Scruton in the 1950s, when studying bridge dynamic response, to study the
forced vibration caused by the wake. The buffeting of bridge refers to the random vibration of the structure under turbulence \(^2\). Buffeting is forced vibration, whose amplitude will not diverge. But too large amplitude will lead to local cracking of bridge, affecting traffic safety and construction quality. Long-term buffeting effect caused to bridge fatigue damage \(^3\). Based on the engineering background of Qingshan extra large span cable-stayed bridge, the wind-induced buffeting and damping control design of the bridge in the largest cantilever construction state and the bridge state are studied, and the results are compared with the wind tunnel test. The bridge safety are determined in turbulence field.

**THEORY OF BUFFETING FORCE**

Buffeting force formula was first studied and proposed by Davenport \(^5\). The wind-induced response of a flexible elongated structure is solved by using the method of probability statistics, stochastic vibration theory based on slice theory and quasi-constant aerodynamic theory. The theory of Davenport was modified by Scanlan and other scholars, and the influence of drag coefficient and derivative was considered in the buffeting force. The buffeting force per unit length of the stiffened beam in the turbulence is written as \(^6\)\(^7\)\(^8\)

\[
L_b = \frac{1}{2} \rho U^2 B \left[ 2 C_L \chi_{Lu} \frac{u}{U} + (C_L' + C_D) \chi_{Lw} \frac{w}{U} \right],
\]

\[
D_b = \frac{1}{2} \rho U^2 B \left[ 2 C_D \chi_{Du} \frac{u}{U} + C_D' \chi_{Dw} \frac{w}{U} \right],
\]

\[
M_b = \frac{1}{2} \rho U^2 B^2 \left[ 2 C_M \chi_{Mu} \frac{u}{U} + C_M' \chi_{Mw} \frac{w}{U} \right],
\]

where \(L_b, D_b, M_b\) are respectively lift, drag and torque of unit length under unit length. \(\rho\) is air density. \(U\) is average wind speed in perpendicular plane of unit. \(B=2b\), which is the width of the bridge. \(C_L, C_D, C_M\) are respectively the static wind coefficient of lift, drag and torque. \(C_L', C_D', C_M'\) are respectively the derivative of wind attack angles of the \(C_L, C_D, C_M\), namely,

\[
C_L' = \frac{dC_L}{d\alpha}, \quad C_D' = \frac{dC_D}{d\alpha}, \quad C_M' = \frac{dC_M}{d\alpha}
\]

\(\alpha\) is wind attack angles. \(\chi_{Lu}, \chi_{Lw}, \chi_{Du}, \chi_{Dw}, \chi_{Mu}, \chi_{Mw}\) are aerodynamic admittance, which are related to wind speed reduction and cross section geometry. Longitudinal and vertical component of fluctuating wind are expressed \(\mu\) and \(w\) in element coordinate system.
The displacement of vertical vibration is expressed by \( u(x, t) \). The general differential equation as follows:

\[
mu(x, t) + cu(x, t) + ku(x, t) = L(x, t)
\]

where \( m \) is mass per unit length, \( c \) is the damping of the unit length, \( k \) is the stiffness per unit length, \( L(x, t) \) is the fluctuating wind load of unit length.

Assuming that the frequency \( \omega_1 \) of the first-order vertical vibration mode of the bridge is known, the mode function is \( \eta(x) \) and the mode damping ratio is \( \zeta_1 \), the vibrational coordinate transformation is

\[
\bar{u}(x, t) = h(t)\eta(x)
\]

In the formula, \( h(t) \) is generalized coordinate of first-order vertical-mode.

Using equation (2-2), multiplied by \( \eta(x) \) on both sides of the equation, then on \( x \) integral,

\[
\tilde{h}(t) + 2\zeta_1\omega_1\tilde{h}(t) + \omega_1^2\tilde{h}(t) = \dot{L}(t)
\]

Modal mass is introduced, \( M = m\int_{0}^{L} \eta^2(x)dx \),

\[
\dot{L}(t) = \frac{1}{M} \int_{0}^{L} L(x, t)\eta(x)dx
\]

Cable-stayed bridge lift as per unit span, \( L = L_{se} + L_b \)

Self-excited force \( L_{se} \) can be expressed as

\[
L_{se} = \frac{1}{2} \rho U^2 B \left[ KH_1^* \frac{h}{U} \right]
\]

Buffeting force \( L_b \) can be represented as

\[
L_b = \frac{1}{2} \rho U^2 B \left[ 2C_L \frac{U}{U} + (C_L' + C_D \frac{w}{U}) \right]
\]

Using equation (2-7), (2-6) and (2-5) to the (2-4), the self-excited force will be moved to the left end of the equation, to merge:

\[
\tilde{h}(t) + 2\zeta_1\omega_1\tilde{h}(t) + \omega_1^2\tilde{h}(t) = \dot{L}_b(t)
\]

Where, \( \tilde{\zeta}_1 = \zeta_1 - \frac{\rho B^2}{2m} H_1^* \), \( H_1^* \) is expressed the vertical aerodynamic derivative, \( \dot{L}_b(t) \) is the general buffeting force.

BUFFETING WIND POWER SPECTRUM AND SIMULATION

According to the situation, Simiu spectrum is used to the horizontal power spectrum,

\[
\frac{nS_{uu}}{u^2} = \frac{200f}{(1 + 50f)^{5/3}}
\]
Lumley-Panofsky spectrum is used to the vertical power spectrum,

\[ \frac{nS_{uu}}{u_c^2} = \frac{6f}{(1+4f)^2} \]  \hspace{1cm} (12)

where, \( f = nz/V(z) \), for similarity coordinates as the height from the ground, \( V(z) \) for average wind speed in height \( Z \), \( n \) as the frequency, \( u^* = KV(z)/\ln(z/z_0) \), for the friction wind speed, associated with surface roughness height, \( K=0.4 \), \( z_0 \) for ground roughness height.

**ENGINEERING BACKGROUND**

Qingshan large span cable-stayed bridge is the eleventh Yangtze River Bridge in Wuhan, which is the important link over the Yangtze River channel. The bridge adopts a two-way eight-lane highway standard construction, with a design speed of 100 kilometers per hour. The bridge is a double tower cable-stayed bridge with a span of 66m + 108m + 156m + 938m + 156m + 108m + 66m. A streamlined flat steel box girder is used as the main girder. The main span is up to 938m. The height of the main girder is 30.5m from ground. Rough height of the ground is 0.01. The average wind speed of the main girder is 29.6 m/s. The elevation of the bridge is shown in Figure 1. The ternary force coefficient and the ternary force coefficient slope of the main girder section are shown in TABLE I below, under different angle of attack.

![Figure 1. The bridge elevation.](image)

**TABLE I. AERODYNAMIC PARAMETERS OF THE GIRDER SECTION UNDER DIFFERENT ANGLES OF ATTACK.**

<table>
<thead>
<tr>
<th>stage</th>
<th>three aerostatic coefficient</th>
<th>three aerostatic coefficient slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lift coefficient</td>
<td>Resistance coefficient</td>
</tr>
<tr>
<td></td>
<td>( C_L )</td>
<td>( C_D )</td>
</tr>
</tbody>
</table>

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WIND VIBRATION RESPONSE AND VIBRATION CONTROL DESIGN OF THE BRIDGE

WIND-INDUCED VIBRATION RESPONSE OF THE MAXIMUM CANTILEVER CONSTRUCTION STATE

In construction stage of long-span cable-stayed bridge, the structural system is constantly changing and not yet forming, which may appear more disadvantage than completed bridge, including smaller stiffness, bigger deformation, poorer stability, larger vibration response. The maximum double cantilever state and the maximum single-arm state of the cable-stayed bridge are the most disadvantage states [8]. According to results of wind tunnel test, when wind-induced vibration is the buffeting effect, the main girder of the bridge has the same degree of buffeting, and the buffeting displacement increases with the increase of the wind speed, and under maximum wind speed, the buffering response is larger. The results of wind tunnel test and numerical calculation are listed below. The envelope value is the maximum wind-induced vibration response value of three groups of buffering wind data at different angles. TABLE II shows the displacement response envelope of the maximum cantilever construction state under the maximum wind speed.

<table>
<thead>
<tr>
<th>stage</th>
<th>Transverse vibration displacement /mm</th>
<th>Vertical vibration displacement /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>maximum cantilever</td>
<td>numerical calculation</td>
<td>606.6</td>
</tr>
<tr>
<td>construction state</td>
<td>wind tunnel test</td>
<td>581.3</td>
</tr>
<tr>
<td>deviation</td>
<td></td>
<td>±4.2%</td>
</tr>
</tbody>
</table>

It can be seen from the above TABLE II that the numerical results and the wind tunnel test results show that the buffeting response of the main girder is larger at maximum wind speed. The transverse vibration displacement response reaches 606.6mm, and the vertical vibration displacement response to 170.6mm. Vertical and
transverse displacement response are large, seriously affected the construction stage safety of the bridge and worker safety.

**WIND-INDUCED VIBRATION RESPONSE OF THE COMPLETED BRIDGE STATE**

Due to the large span of the bridge and the maximum wind speed reaching to 34.9m/s, the wind-induced response of the completed bridge is also the dominant load of the bridge. According to the wind tunnel test results, when the wind is turbulent wind, the main girder has different degree of buffeting phenomenon, and the buffeting response is largest at the maximum wind speed. The results of wind tunnel test and numerical calculation are listed below. The envelope value is the maximum wind-induced vibration response value of three groups of buffering winds at different angles. TABLE III shows the displacement response envelope value of the completed bridge in the middle of the girder at the maximum wind speed.

<table>
<thead>
<tr>
<th>stage</th>
<th>Transverse vibration displacement /mm</th>
<th>Vertical vibration displacement /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>numerical calculation</td>
<td>472.4</td>
<td>126.1</td>
</tr>
<tr>
<td>the completed bridge</td>
<td>wind tunnel test</td>
<td>364.4</td>
</tr>
<tr>
<td>deviation</td>
<td>±22.9%</td>
<td>±5.5%</td>
</tr>
</tbody>
</table>

It can be seen from TABLE III that the buffeting response envelope value of the main girder is smaller than that of the maximum cantilever construction state under the maximum wind speed, due to the large stiffness, good stability and relatively small deformation of the completed bridge. In completed bridge, the transverse vibration response in the girder middle reaches 472.4mm and the vertical vibration displacement response reaches 133.1mm. It will also affect the safety of the completed bridge. In addition, it can be seen from TABLE III that the numerical results of the transverse vibration displacement are larger than those in the wind tunnel test, which is due to the practical finite elastic stiffness of the transverse wind support in the numerical calculation process.

**VIBRATION CONTROL DESIGN OF THE BRIDGE**

The tune mass damper (TMD) system is an effective device, which can suppress the vibration of the main structure. When the parameters are adjusted properly, the vibration of the bridge can be effectively controlled [2]. Some scholars at home and abroad have proposed a multi-tune mass damper (MTMD) to control the structure of
the method [9] [10]. The MTMD is composed of multiple TMDs. The natural frequency of each TMD is distributed within a certain range of the controlled modal frequency. The mechanical model of the wind-induced vibration control of the multi-tune mass damper is shown in Figure 2, where $M_1, M_2 ... M_N$ represent the mass quantities of MTMD, $k_1, k_2 ... k_N$ represent the stiffness quantities of MTMD, $c_1, c_2 ... c_N$ represents the damping quantities of MTMD, $M_S, c_s, k_s$ represent the quality, damping and stiffness of the bridge.

![Mechanical model of MTMD](image)

Under buffeting action, displacement of bridge deck is $y_s(x, t)$, the displacement of the $i$-th TMD is $y_i(x, t)$, and the wind load is $f(t)$. Dynamic equilibrium equation of the main girder is:

$$m_s \ddot{y}_s + c_s \dot{y}_s + k_s y_s - \sum_{i=1}^{N} c_i (\ddot{y}_i - \dot{y}_s) - \sum_{i=1}^{N} k_i (y_i - y_s) = f(t)$$  \hspace{1cm} (13)

The dynamic balance equation of the $i$-th tune mass damper is:

$$m_i \ddot{y}_i + c_i (\ddot{y}_i - \dot{y}_s) + k_i (y_i - y_s) = 0 \quad (i=1, 2, \ldots, N)$$  \hspace{1cm} (14)

Adding the bridge to the dynamic balance equation of the multi-tune mass dampers:

$$(m_s + \sum_{i=1}^{N} m_i) \ddot{y}_s + c_s \dot{y}_s + k_s y_s = f(t) - \sum_{i=1}^{N} m_i (\dddot{y}_i - \dot{y}_s)$$  \hspace{1cm} (15)

According to the optimal principle of MTMD and the optimization of MTMD parameters, four TMDs whose weight is 7.5 tons are needed to be set up at the maximum cantilever construction stage of the transverse of the bridge, where the size of single TMD is $3m \times 2m \times 1m$. And one TMD which has the same weight is needed to be installed on the vertical of the bridge, where the single TMD size of $1.8m \times 2.1m \times 1.5m$. The completed bridge phase has the same weight and size of TMD as the maximum cantilever construction stage in the transverse; the vertical of
the bridge is needed to install three TMDs where the weight is 7.5 tons, and single TMD has the same size. The envelope response of the bridge is shown in TABLE IV and TABLE V.

**TABLE IV. THE ENVELOPE RESPONSE OF THE MAXIMUM CANTILEVER CONSTRUCTION STATE.**

<table>
<thead>
<tr>
<th>stage</th>
<th>Transverse vibration displacement /mm</th>
<th>Vertical vibration displacement /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>maximum cantilever construction state comparison</td>
<td>Before damping</td>
<td>After damping</td>
</tr>
<tr>
<td>numerical calculation</td>
<td>606.6</td>
<td>242.5</td>
</tr>
<tr>
<td>wind tunnel test</td>
<td>581.3</td>
<td>230.2</td>
</tr>
</tbody>
</table>

**TABLE V. THE ENVELOPE RESPONSE OF THE COMPLETED BRIDGE IN THE MIDDLE OF THE GIRDER.**

<table>
<thead>
<tr>
<th>stage</th>
<th>Transverse vibration displacement /mm</th>
<th>Vertical vibration displacement /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>completed bridge comparison</td>
<td>Before damping</td>
<td>After damping</td>
</tr>
<tr>
<td>numerical calculation</td>
<td>472.4</td>
<td>147.8</td>
</tr>
<tr>
<td>wind tunnel test</td>
<td>364.4</td>
<td>93.6</td>
</tr>
</tbody>
</table>

It can be seen from TABLE IV and TABLE V that the vibration damping rates of the maximum cantilever construction state in the transverse of the bridge are respectively 60% and 60.4%, and the vertical vibration displacement damping rates is 80.4% and 62.7%. The transverse vibration damping rates of the completed bridge are 68.7% and 74.3%, respectively and the vertical vibration damping rates are 72.9% and 65.9% respectively.

**CONCLUSION**

(1) The results of numerical calculation and wind tunnel test show that the buffeting behavior of the bridge occurs, at the maximum cantilever construction and the completed bridge. And the buffeting response of the bridge girder is largest at the maximum wind speed. The transverse displacement of the girder is 606.6mm and the vertical displacement reaches 170.6mm at the maximum cantilever construction. The transverse displacement of the girder reaches 472.4mm and the vertical displacement reaches 133.1mm at the completed bridge.
The vibration damping control design of the bridge is carried out. The maximum vibration damping rate of the transverse of the girder is 60.4% at the maximum cantilever construction, the maximum vibration damping rate of the vertical of the girder is 80.4%. The transverse vibration damping rate is 74.3% and vertical vibration damping rate is up to 72.9%, at the completed bridge.

REFERENCES

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