Wind-induced vibrations and equivalent static wind loading for cable-stayed bridges

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ABSTRACT: Although cable-stayed bridges generally have higher rigidity and natural frequencies, flutter instability still can pose threats to some aerodynamic bluff girder sections, such as box girder section with cantilevered slab and two-isolated-girder section. Experimental researches on the flutter instability of these girder sections were carried out to find effective ways to improve their flutter performance. While the adoption of twin box girder can be an effective solution to improve the flutter stability of cable-stayed bridges, more attention should be paid to its vortex-induced vibration performance. Based on a long-span cable-stayed bridge, the vortex-induced vibration performance of twin box girder section was evaluated and aerodynamic control measure was proposed to mitigate the vibration response. Since it is more convenient for a bridge engineer to evaluate buffeting responses of a bridge structure by some kind of equivalent static wind loading than comprehensive buffeting analysis, the method to calculate equivalent wind load of cable-stayed bridges caused by buffeting response, which is recommended by the China Code for wind-resistant design of highway bridges, was demonstrated.

KEYWORDS: vortex-induced vibration; flutter; vortex-induced vibration; vibration control measure; wind tunnel test; equivalent static wind load

1 INTRODUCTION

With the rapid increase of span length, bridge structures are becoming more flexible and more vulnerable to wind-induced vibrations. Compared with suspension bridges with similar main span, cable-stayed bridges usually have higher rigidity and natural frequencies, which implies better aerodynamic stability. However, if the shape of a main girder hasn’t been optimized enough to obtain good aerodynamic performance, there still will exist flutter instability problems even when the main span of a cable-stayed bridge isn’t very large. For those girder sections which has been optimized to have better flutter performance, another possible self-excited vibration, vortex-induced vibration, may affect the operating performance of a bridge if happens. Furthermore, buffeting always exists when a bridge is exposed to turbulent flow, which is very common in the Atmospheric Boundary Layer. Different from the way flutter and vortex-induced vibration are dealt with, it is more convenient for a bridge engineer to evaluate the buffeting responses of a bridge structure by some kind of equivalent static wind loading, than comprehensive three-dimensional frequency domain or time domain buffeting analysis. In this paper, problems related to flutter and vortex-induced vibration of cable-stayed bridges and their countermeasures are introduced based on some engineering projects in China, and the method to calculate equivalent wind load caused mainly by buffeting response, which is recommended by the China Code for wind-resistant design of highway bridges (cable-stayed bridges and suspension bridges), will be demonstrated.
2 FLUTTER INSTABILITY

Generally speaking, cable-stayed bridges have better flutter performance than suspension bridges for higher rigidity resulted from their unique structural system. Even for a super long-span cable-stayed bridge which have a main span over 1 kilometer, flutter instability never posed a real threat after a spatial cable system and a reasonable girder section were adopted. However, the span length isn’t the only key factor which determines the flutter performance of a long-span bridge, as the infamous original Tacoma Narrows Bridge, which suffered from flutter-induced collapse, was not the longest bridge at that time. The dynamic performance of the structural system composed of girder, stay-cables and pylon also matters, as well as the aerodynamic configuration of the girder section. For cable-stayed bridges with middle-range main spans, aerodynamically blunter composite main girders are more preferred by designers in most cases to traditional flat single box steel girders for economical reasons.

2.1 STRUCTURAL INFORMATION

The two girder sections (Figure 1) investigated in this research have been widely used as the main girder section of cable-stayed bridges, especially when the bridge span is not very large. Section A, box girder section with cantilevered slab, and Section B, two isolated rectangular box girder, are main girders of the Main Bridge and the Kezhushan Bridge respectively, which are two major crossings of East Sea Bridge over two main navigational channels between the Yangshan International Deep Water Harbor and Shanghai city.

Although the main spans of these two bridge are not very large, the torsional fundamental frequency are lower than 0.6 Hz and the ratios between the torsional and bending fundamental frequencies are no more than 1.65, due to their general structural layout (single cable plane or parallel cable plane) and composite girder sections.

2.2 FLUTTER PERFORMANCE

Sectional model tests and aeroelastic mode tests were carried out to examine the flutter performance of these two cable-stayed bridges in TJ-1 and TJ-3 Boundary Layer Wind Tunnel in the State Key Laboratory for Disaster Reduction in Civil Engineering (SKLDRCE) of Tongji University as shown in Figure 2.
Figure 2 Sectional model and aeroelastic model wind tunnel test for flutter performance

The measured flutter critical speeds for both girder sections of these two cable-stayed bridges are listed in table 1. For Section A, the structure is very aerodynamically stable even after wind speed exceeds 100 m/s when the wind angle of attack is zero or negative. However, when the wind angle of attack turns positive, the flutter onset speed decreases dramatically and is below the flutter checking wind speed. This is the characteristics of the aerodynamic performance for this type of girder section. The flutter performance of Section B is worse than Section A since this is a blunter girder section. The results of aeroelastic model tests generally have good agreement with sectional model tests.

Table 1 Flutter critical speed of the original section (m/s)

<table>
<thead>
<tr>
<th>Girder</th>
<th>Test</th>
<th>$-3^\circ$</th>
<th>$0^\circ$</th>
<th>$+3^\circ$</th>
<th>$[U_{cr}]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>Sectional model</td>
<td>&gt;176</td>
<td>145.0</td>
<td>81.4</td>
<td>84.6</td>
</tr>
<tr>
<td></td>
<td>Aeroelastic model</td>
<td>&gt;100</td>
<td>82.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section B</td>
<td>Sectional model</td>
<td>96.2</td>
<td>74.8</td>
<td>69.0</td>
<td>79.9</td>
</tr>
<tr>
<td></td>
<td>Aeroelastic model</td>
<td>72.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Based on the testing results, the flutter critical speeds for both original structures cannot meet the requirement of flutter checking speeds of 84.6 m/s for the main navigational channel bridge and 79.9 m/s for Kezhushan Bridge, so effective preventive measures had to be considered to stabilize the original bridges.

2.3 FLUTTER CONTROL

In order to improve the aerodynamic stability of the main navigational bridge, central barrier was installed on the bridge deck (Section A) as shown in Figure 3. Three different heights of central barrier were chosen to investigate the controlling effect of this aerodynamic measure, which are 0.8m, 1.0m and 1.2m respectively, or 20%, 25% and 30% of the girder depth respectively.

Figure 3 Girder section A with central barrier

The measured flutter critical speeds are listed in table 2. As the results shown, the flutter controlling effect of central barrier is excellent. For three types of central barrier the flutter onset speeds are all exceed the wind speed requirement, and the section with 1.2 m high central barrier has the best aerodynamic performance which increases the flutter onset speed by approximately 11%.

Table 2 Flutter critical speed of Section A with central barriers (m/s)

<table>
<thead>
<tr>
<th>Angle of attack</th>
<th>$-3^\circ$</th>
<th>$0^\circ$</th>
<th>$+3^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original section</td>
<td>&gt;176</td>
<td>145.0</td>
<td>81.4</td>
</tr>
<tr>
<td>0.8m central barrier</td>
<td>&gt;176</td>
<td>151.8</td>
<td>85.8</td>
</tr>
</tbody>
</table>
As mentioned previously, when at positive angle of attack the aerodynamic stability of the original girder section A is rather low and different from those at negative and zero angle of attack. It can be inferred that adjusting aerodynamic configuration of the windward inclined web and the bottom slab will affect the aerodynamic stability of the girder section dramatically. So several measures concerning various positions of the inspection rails were investigated in wind tunnel tests. These testing sections are shown in Figure 4.

<table>
<thead>
<tr>
<th>Angle of attack</th>
<th>0°</th>
<th>+3°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original section</td>
<td>&gt;176</td>
<td>145.0</td>
</tr>
<tr>
<td>Section A1</td>
<td>&gt;176</td>
<td>162.8</td>
</tr>
<tr>
<td>Section A2</td>
<td>&gt;176</td>
<td>151.8</td>
</tr>
<tr>
<td>Section A3</td>
<td>&gt;176</td>
<td>121.0</td>
</tr>
<tr>
<td>Section A4</td>
<td>&gt;176</td>
<td>154.0</td>
</tr>
<tr>
<td>Section A5</td>
<td>&gt;176</td>
<td>154.0</td>
</tr>
</tbody>
</table>

From the testing results we can see that the aerodynamic stabilities of sections with central barrier and inspection rails being appropriately positioned have been improved remarkably except section A3. Section A1 which has 1.2m central barrier and inspection rails at the bottom of the inclined web has the best aerodynamic performance considering flutter instability. For section A5 which has inspection rails at the same position as section a and no central barrier, the aerodynamic stability also has been improved by 11% and exceeds the flutter verification speed. As section A5 also has economic advantage, this flutter control measure is finally recommended to bridge designer.

The girder section B of the Kezhunshan bridge is a more bluff body compared to the original girder section A of the main navigational bridge. So the first flutter control measure taken into account is the installation of fairings. The girder section with fairings is shown in Figure 5 a).

<table>
<thead>
<tr>
<th>Angle of attack</th>
<th>0°</th>
<th>+3°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original section</td>
<td>&gt;176</td>
<td>145.0</td>
</tr>
<tr>
<td>Section A1</td>
<td>&gt;176</td>
<td>162.8</td>
</tr>
<tr>
<td>Section A2</td>
<td>&gt;176</td>
<td>151.8</td>
</tr>
<tr>
<td>Section A3</td>
<td>&gt;176</td>
<td>121.0</td>
</tr>
<tr>
<td>Section A4</td>
<td>&gt;176</td>
<td>154.0</td>
</tr>
<tr>
<td>Section A5</td>
<td>&gt;176</td>
<td>154.0</td>
</tr>
</tbody>
</table>

Figure 5 Girder section B with fairings
The flutter performance of girder section B with fairings was tested in aeroelastic model wind tunnel test, and the results are listed in table 4. For each angle of attack the flutter onset speed is over the flutter verification speed 79.9 m/s, which means it is a very effective measure to improve the aerodynamic stability for such a two-isolated-girder section.

Table 4 Flutter critical speed of section B with fairings and central barrier (m/s)

<table>
<thead>
<tr>
<th>Angle of attack</th>
<th>Original section B</th>
<th>Section with fairings</th>
<th>Section with central barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>72.5</td>
<td>95.0</td>
<td>87.5</td>
</tr>
<tr>
<td>-3°</td>
<td></td>
<td>&gt;100</td>
<td>&gt;100</td>
</tr>
<tr>
<td>0°</td>
<td></td>
<td>87.5</td>
<td>82.5</td>
</tr>
<tr>
<td>+3°</td>
<td></td>
<td>95.0</td>
<td>&gt;100</td>
</tr>
</tbody>
</table>

For the interest of research on the effectiveness of flutter control measures, an alternative control measure which has a central barrier installed under the bridge deck and with the same height as the girder section is investigated in this research. The girder section B with central barrier is shown in Figure 5 b). Aeroelastic model tests were carried out to verify the effectiveness of the installation of central barrier. The corresponding testing results are also shown in table 4. Although the flutter controlling effectiveness of the central barrier on this type of girder section is not as remarkable as that of the application of fairings, this measure does improve the aerodynamic stability of the original section to meet the design requirement.

3 VORTEX-INDUCED VIBRATION

Even for aerodynamic stable girder sections which have been optimized to obtain better flutter performance, another kind of self-excited vibration, vortex-induced vibration, may happen at lower wind speed range. Section C is a typical twin box girder with a central vent (Figure 6), which is the main girder of Shanghai Yangtze Bridge, a three-span cable-stayed bridge with a main span of 730m.

Figure 6 Girder section C

Although twin box girder has been proven to be very effective to enhance the flutter performance of super-long span bridges, more attention should be paid to the vortex-induced vibration performance of this innovative girder section, since the existence of central vent will make the vortex-shedding process and its interaction with the movement of the bridge girder more complicated.

3.1 VIV PERFORMANCE

For detailed investigation on VIV responses, a combination of geometrical, mass, stiffness and Reynolds number considerations resulted in the selection of a 1:25 scale for the sectional model. These large-scale sectional models have the total length of about 3.6 m and the width of about 2.06m. The wind tunnel tests were conducted in smooth flow at Tongji University’s TJ-3 Boundary Layer Wind Tunnel with the working section of the 15m width, the 2m height and the 14m length. Two rigid and streamlined temporary end walls were designed and built in the working section to accommodate the large sectional model (Figure 7), and the suspension system including eight springs and supporting frames was built in these two end walls together with four laser displacement transducers for the measurement of the response of the sectional model to vortex-shedding excitation.
Both heaving and torsional VIV responses were observed in the testing, and the Strouhal Number and Reynolds Number at the VIV condition are 0.1230 and 90,768 respectively. Table 1 list the maximum VIV amplitudes and some important parameters in VIV conditions.

<table>
<thead>
<tr>
<th>VIV in heaving (h/D)</th>
<th>VIV in torsion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ampli-</td>
<td>S₁</td>
</tr>
<tr>
<td>0.054</td>
<td>0.1620</td>
</tr>
</tbody>
</table>

3.2 **VIV CONTROL**

Guide vane is a possible VIV control solution, which can smooth air flow around the corner of girder section. After comparisons of the VIV mitigating effects between different locations of guide vanes, the optimum location for Section C was found to be under the bottom slab near central vent, which is shown in Figure 8. The responses of the model with and without guide vanes at -3° wind angle of attack are shown in Figure 9. It can be seen that the torsional VIV of the original twin box girder can be mitigated effectively by the installation of guide vanes. No significant heaving VIV of the girder with guide vanes was observed except for the case at -3° wind angle of attack. In this case the peak amplitude of heaving VIV was reduced from 21.7 cm to 13.6 cm for the prototype bridge structure with 0.3% structural damping ratio. Guide vane seems to have better mitigating effect on torsional VIV than heaving VIV for twin box girder sections. Comparisons of maximum VIV amplitudes among +3°, 0°, and -3° wind angles of attack for girder section C and another two long-span bridges having twin-box girders with and without guide vanes at the condition of 0.3% structural damping ratio are listed in Table 6. The results also imply that the VIV control effect of guide vane is rather sensitive to the outboard and inboard shape of a twin box girder section.

Figure 8 Section C with guide vanes
4 EQUIVALENT WIND LOAD

Buffeting always exists when a bridge is exposed to turbulent flow, which is very common in the Atmospheric Boundary Layer. For a bridge engineer, it is more convenient to evaluate the buffeting responses of a bridge by some kind of equivalent static wind load. Based on the China Code, the total wind load on a bridge structure can be calculated by the following formula:

\[ L_{\text{Total}} = L_{SG} + L_R \]

Where \( L_{\text{Total}} \) is the total wind load, \( L_{SG} \) is called static gust wind load, which is actually the summation of the static wind load and the equivalent static wind load related to background buffeting responses, and \( L_R \) is the equivalent static wind load related to resonant buffeting responses, which can be calculated through Inertia Wind Loading Method.

4.1 STATIC GUST WIND LOAD

According to the China Code for the wind-resistant design of highway bridges, the method to calculate the static gust wind load is the same as static wind load. The only difference is to replace design wind speed with static gust wind speed in static gust wind load evaluation. The conversion factor from design wind speed to static gust wind speed is called static gust wind coefficient, which varies with terrain types at bridge site and horizontal wind-load-bearing length. For the Main Bridge of East Sea Bridge, this coefficient is 1.22 at the service state.

In order to get the static gust wind load, the aerodynamic force coefficients of the girder section were measured through sectional model wind tunnel tests and listed in Table 7. Coefficients of the main pylon were suggested by the code.

<table>
<thead>
<tr>
<th>Components</th>
<th>Drag force ( (C_d) )</th>
<th>Lift force ( (C_l) )</th>
<th>Pitch moment ( (C_m) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>Service 0.931</td>
<td>Construction 0.661</td>
<td>Service -0.043</td>
</tr>
<tr>
<td>Pylon</td>
<td>Service 1.5</td>
<td>Construction 2.0</td>
<td>Service 1.5</td>
</tr>
</tbody>
</table>

Table 7 Aerodynamic force coefficients
Then the static gust wind load for the main girder at the service stage can be calculated as follows.

\[ W_y^d(x) = \overline{W}_{y0}^d \alpha_{y0}^d(x) = \frac{1}{2} \rho U_c^2 BC_y = -2.814 \text{kN/m} \]  
\[ W_z^d(x) = \overline{W}_{z0}^d \alpha_{z0}^d(x) = \frac{1}{2} \rho U_c^2 HC_z = 7.385 \text{kN/m} \]  
\[ W_\theta^d(x) = W_{\theta0}^d \alpha_{\theta0}^d(x) = \frac{1}{2} \rho U_c^2 B^2 C_M = 116.614 \text{kN \cdot m/m} \]

Where, \( \overline{W}_{y0}^d \), \( \overline{W}_{z0}^d \), \( \overline{W}_{\theta0}^d \) are maximum values of the vertical, horizontal and torsional components of the static gust wind load for the main girder, \( \alpha_{y0}^d \), \( \alpha_{z0}^d \), \( \alpha_{\theta0}^d \) are distribution function for the three components of the static gust wind load for the main girder, and \( \alpha_{y0}^d = \alpha_{z0}^d = \alpha_{\theta0}^d = 1 \).

The static gust wind load for the main pylon at the service stage can be calculated in the similar way.

\[ W_y^p(z) = \overline{W}_{y0}^p \alpha_{y0}^p(z) = \frac{1}{2} \rho U_c^2 B_D C_D \left( \frac{z}{z_i} \right)^{2u} \frac{B_D(z)}{B_D} = 29.45 \left( \frac{z}{z_i} \right)^{0.2} \frac{B_D(z)}{B_D} \text{(kN/m)} \]  
\[ W_z^p(z) = \overline{W}_{z0}^p \alpha_{z0}^p(z) = \frac{1}{2} \rho U_c^2 B_L C_L \left( \frac{z}{z_i} \right)^{2u} \frac{B_L(z)}{B_L} = 25.77 \left( \frac{z}{z_i} \right)^{0.2} \frac{B_L(z)}{B_L} \text{(kN/m)} \]

Where, \( \overline{W}_{z0}^p \) and \( \overline{W}_{z0}^p \) are in-plane and out-of-plane static gust wind load for the main pylon at the top of the pylon, while \( \alpha_{z0}^p \) and \( \alpha_{z0}^p \) are distribution function for the static gust wind load for the main pylon.

\[ \alpha_{y0}^p(z) = \left( \frac{z}{z_i} \right)^{0.2} \frac{B_D(z)}{B_D}, \quad \alpha_{z0}^p(z) = \left( \frac{z}{z_i} \right)^{0.2} \frac{B_L(z)}{B_L} \]

\( C_D \) and \( C_L \) are in-plane and out-of-plane aerodynamic force coefficients of the main pylon.

The results of the static gust wind load for the main girder and main pylon of the Main Bridge of the East Sea Bridge is shown in Figure 10.
4.2 EQUIVALENT STATIC WIND LOAD

The equivalent static wind load related to resonant buffeting responses can be calculated through Inertia Wind Loading Method. So the equivalent wind load can be evaluated by the following formulas.

\[
W_y(s) = \tilde{W}_y(s) + \tilde{W}_y(s) \approx \tilde{W}_{y_1}(s) + \tilde{W}_{y_2}(s)
\]

\[
W_z(s) = \tilde{W}_z(s) + \tilde{W}_z(s) \approx \tilde{W}_{z_1}(s) + \tilde{W}_{z_2}(s)
\]

\[
W_\theta(s) = \tilde{W}_\theta(s) + \tilde{W}_\theta(s) \approx \tilde{W}_{\theta_1}(s) + \tilde{W}_{\theta_2}(s)
\]

Where, \(\tilde{W}_y\), \(\tilde{W}_z\), \(\tilde{W}_\theta\) are static gust wind load which can be calculated through equations (1) and (2), \(\tilde{W}_{y_1}\), \(\tilde{W}_{z_1}\), \(\tilde{W}_{\theta_1}\) are vertical, horizontal and torsional components of buffeting inertial forces, \(\tilde{W}_{y_2}\), \(\tilde{W}_{z_2}\), \(\tilde{W}_{\theta_2}\) are peak values of three components of buffeting inertial forces for the first natural vibration mode, \(\tilde{W}_{y_1}\), \(\tilde{W}_{z_1}\), \(\tilde{W}_{\theta_1}\) are peak values of three components of buffeting inertial forces for the second natural vibration mode, \(\alpha_{y_1}(s)\), \(\alpha_{z_1}(s)\), \(\alpha_{\theta_1}(s)\) are distribution functions for the three components of buffeting inertial forces for the first natural vibration mode, \(\alpha_{y_2}(s)\), \(\alpha_{z_2}(s)\), \(\alpha_{\theta_2}(s)\) are distribution functions for the three components of buffeting inertial forces for the second natural vibration mode.

Based on the buffeting responses measured in aeroelastic model wind tunnel tests, the buffeting inertial forces at any measuring points on the main girder or the main pylon can be calculated through the following formula.

\[
\tilde{W}_r\alpha_r(s_k) = (2\pi f_i)^2 \sigma_{r_m}(s_k) m_r(s_k)
\]

Where, \(f_i\) is the frequency of the i\(^{th}\) mode, \(\sigma_{r_m}\) is the peak factor of the buffeting response of the i\(^{th}\) mode, \(\sigma_{r_m}(s_k)\) is the root-mean-square (RMS) value of the buffeting response at measuring point \(s_k\) for the i\(^{th}\) mode, \(m_r(s_k)\) is the mass or mass moment of inertia at measuring point \(s_k\). For locations other than those measuring points, the buffeting inertial forces can be evaluated by

\[
\tilde{W}_r\alpha_r(s) = (2\pi f_i)^2 \sigma_{r_m}(s) m_r(s) \varphi_{r_i}(s_j)
\]

Where, \(\sigma_{r_m}\) is the RMS value of the buffeting response at the location corresponding to the maximum value of the modal function for the i\(^{th}\) mode, \(m_r(s_j)\) is the mass or mass moment of inertia at any location \(s_j\) on the main girder or the main pylon, \(\varphi_{r_i}(s_j)\) is the value of the normalized i\(^{th}\) modal function at the location \(s_j\).

The results of the equivalent static wind load for the main girder and main pylon of the Main Bridge of the East Sea Bridge at the design wind speed is shown in Figure 11.

5 CONCLUSIONS

Experimental researches on the flutter instability of two aerodynamically bluff girder sections, box girder section with cantilevered slab and two-isolated-girder section, which have been widely adopted in the constructions of cable-stayed bridges, were carried out to find effective ways to improve their flutter performance. For single box girder section, the aerodynamic stability under positive wind angle of attack is critical, and adding central barrier on the deck and position adjustment of inspection rails to the bottom of the inclined web are effective aerodynamic measures to improve flutter performance. For two-
isolated-girder section, installing central barrier under the bridge deck and adding fairing can both increase flutter onset speed significantly.

Based on the results of large scaled model wind tunnel tests for a long-span cable-stayed bridge, twin box girders are prone to have vortex-induced vibrations with large amplitude. Installing guide vanes at the bottom of the girder section near central vent can mitigate vortex-induced vibrations, especially in torsional motion.

According to the China Code for wind-resistant design of highway bridges, the equivalent wind load for a cable-stayed bridge is expressed by the summation of static gust wind load and equivalent static wind load. The static gust wind load can be calculated in the similar way as static wind load, and the equivalent static wind load related to resonant buffeting responses can be evaluated by Inertia Wind Loading Method.

![Figure 11 Equivalent static wind load](image)

6 ACKNOWLEDGEMENT

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