Wind-induced fatigue analysis of Yueyang Dongting Lake Suspension Bridge

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ABSTRACT: Recent demand for a longer span has complicated the dynamic behaviours in the strong wind. The limited amplitude but frequent occurred buffeting response of bridge affect fatigue resistance of the bridge, which should be considered. The time-domain buffeting response and fatigue performance of Yue yang Dongting Lake Suspension Bridge are realized by mixed programming with MATLAB and ANSYS. The gusty wind was simulated by the WAWS method, while the self-excited force was modelled as the elemental aeroelastic stiffness and damp matrices by the element Matrix 27. The mode superposition method was then adapted to calculate the buffeting response of the bridge in ANSYS, and its accuracy is verified by the results of the RMS and PSD using the traditional frequency-domain method. It was found that the fatigue life reliability of the suspender neat tower of the leeward side is 97.54% during the structure's service in the design wind speed.

1. INTRODUCTION:
Suspension bridges are taken as the priority bridge-type spanning significant obstacles because of their large span capacity. The bridge stiffness decreases with the increase of span length, and hence the long-span suspension bridge is sensitive to wind loading. The buffeting responses are related to the wind field and dynamic structural properties and the bridge size, and the interaction between wind and structural motions. The wind-induced fatigue damage problem of critical bridge components, such as bridge deck and hanger rope, may have to be considered in the bridge's design. It has carried out many research on the fatigue problem for long-span bridges focusing principally on the bridge girder [1-3]. Also, a specific fatigue analysis on the hanger ropes, which experience high-stress variations during the lifetime, was required in recent years [4-6].

This paper investigates the buffeting-induced fatigue of hanger ropes in the Yueyang Dongting Lake Suspension Bridge (YDLSB) with a main span of 1480 m. The specific objectives of this study are to (1) perform the time-domain buffeting responses implemented in ANSYS with the aeroelastic effect included; (2) investigate the distribution characteristics of buffeting displacement of the bridge deck and stress responses of bridge suspender; and (3) evaluated the accumulative fatigue damage to the critical steel members during the bridge design life.
2. BRIDGE DESCRIPTION
The subject of this study is YDLSB (as shown in Figure 1), a double tower and double span steel suspension bridge that crosses Dongting Lake in Yueyang, China. The total length of the bridge is 1933.6m with the span arrangement of 1480 m+453.6 m. Figure 2 shows the schematic elevation view of the bridge. The stiffening girder of the main bridge adopts a plate girder combined stiffening girder beam, with the girder height of 9 m, girder width of 35.4 m and internode length of 8.4m. In order to solve the problem that the orthogonal special-shaped plate bridge system is prone to be fatigue, it adopts STC high-performance ultra-thin concrete layer instead of the traditional bridge deck pavement so that the thickness of the steel bridge deck is reduced to 12mm. At the same time, the bridge deck is combined with the upper chord and the crossbeam so that the bridge deck and truss can bear the force together to reduce the stress level. The span ratio of the main cable is F/L=1/10. The centre distance of the main cable is 35.4 m, and the cable tower adopts the concrete bridge tower with a portal frame.

![Figure 1 General arrangement of bridge (unit: m)](image1)

![Figure 2 Steel truss girder (unit: m)](image2)

3. DYNAMIC FINITE ELEMENT MODEL
A three-dimensional finite element model of the bridge has been built, which is shown in Figure 3. In this model, the two truss girders are modelled as Timo-shenko's beam elements with 6 degrees of freedom (DOFs) at each node, which account for transverse shear deformation. The bridge towers and piers are also modelled as Timoshenko's beam elements. A 2-node link element simulates the cable and suspender, accounting for only tension and no compression based on the actual condition. An iterative method calculates the cable configuration under the final dead load to consider the geometric stiffness of cables under dead load.

![Figure 3 Finite element model of the bridge](image3)

The suspension bridge is a very geometrically nonlinear structure. For the long-span suspension bridge, the initial stress generated by its weight and dead load will significantly affect the structure's
stiffness. The suspension bridge is entirely dependent on the shape change to obtain the stiffness of the structure. Before performing modal analysis, the static analysis must be carried out to analyze the reasonable initial line shape and initial internal force under the action of dead load by nonlinear iteration. The dynamic analysis considers the stress stiffness, the stiffness caused by large deformation, and the structure's stiffness to obtain the dynamic characteristics. The mesh size of the finite element (FE) modal is determined by the convergence property of the nonlinear iteration calculation. The main dynamic properties modes are listed in Table 1. In this paper, we compare my results with the wind resistance report of the bridge.

| Mode num. | Frequency(Hz) Report[3] | Frequency(Hz) This paper | Mode description | Diff. (%) |
|-----------|--------------------------|--------------------------|------------------|-----------|
| 1         | 0.056                    | 0.055                    | L-S-1            | -1.79%    |
| 2         | 0.093                    | 0.109                    | V-S-1            | 17.20%    |
| 3         | 0.115                    | 0.118                    | V-A-1            | 2.61%     |
| 4         | 0.142                    | 0.138                    | L-A-1            | -2.82%    |
| 5         | 0.152                    | 0.152                    | V-S-2            | 0.00%     |
| 6         | 0.198                    | 0.196                    | V-S-3            | -1.01%    |
| 7         | 0.206                    | 0.210                    | V-A-2            | 1.94%     |
| 9         | 0.229                    | 0.225                    | T-S-1            | -1.75%    |
| 13        | 0.266                    | 0.258                    | L-S-2            | -3.01%    |
| 14        | 0.275                    | 0.270                    | V-S-4            | -1.82%    |
| 20        | 0.305                    | 0.306                    | T-A-1            | 0.33%     |

L-Lateral V-vertical T-Torsional S-symmetrical A-asymmetrical
The modal analysis results reveal the following dynamic properties of the bridge: (1) the first mode is symmetric lateral bending, which will significantly affect the lateral displacement of the structure under wind load. The lateral bending stiffness of suspension bridge is still its weak term. (2) the vertical bending mode of the main beam appeared, and the distribution of the vertical bending mode repeatedly appeared. (3) The first six vibration modes are mainly based on the vibration of the stiffener beam, which shows that the stiffness of the stiffener beam in the entire cable system is relatively weak. (4) The first-order torsional modes of stiffening beams appear in the 9th and 12th order, respectively, and appear later than the lateral and vertical bending. The main transverse truss of stiffening beams is dense, and the ratio of width B to height H of the structure is close to 4, so the structure has strong torsional stiffness. (5) The structural modal participation factor shows that the structure of the modal coupling effect is stronger. Structural lateral and torsional vibration modes often coincide, proving the model of shear and torsion hearts do not overlap.

4. NONLINEAR BUFFETING RESPONSE OF THE LONG-SPAN BRIDGE

4.1 Simulation of the Wind Field
The multi-dimensional multivariable stochastic process of natural wind is treated as the ergodic stochastic process, and the time history of fluctuating wind is simulated by the spectral decomposition theory based on the superposition of trigonometric series. The design wind speed of the bridge, the turbulence degree, the power spectral density function, and the fluctuating wind correlation function are all taken according to the specifications. Kaimal spectrum is adopted for horizontal direction and Panofsky spectrum for vertical fluctuating wind direction. Davenport coherence function is used to describe the correlation of points in different spatial positions. The test of power spectral density and the coherence function on the simulated wind field is shown below.
4.2 Buffeting-Induced Buffeting Response

4.2.1 Buffeting Analysis Method
The simulated fluctuating wind speed time histories are input to obtain the buffeting responses of the bridge in the time domain. The buffeting analysis is carried out in the finite element model with the following steps: (1) A 3D FE model is established. (2) The steady aerodynamic forces and buffeting forces are obtained by the measured three-component force coefficient obtained by the wind tunnel experiments in Hunan university. The force coefficient and its derivative are shown in Figure 7. Buffeting force is determined by Scanlan's quasi-steady aerodynamic theory. It should be noted that the effect of the aerodynamic admittance is disregarded because it was not available from wind tunnel tests. (3) The matrix 27 elements are incorporated to simulate the self-excited aerodynamic forces, which depend on the flutter derivative of the main girder [3]. (4) The buffeting forces and steady aerodynamic forces are applied as external loadings to the structural model to analyze the buffeting responses of the bridge in the time domain.

![Figure 7 Static force coefficient and its derivative of main girder](image)

4.2.2 Buffeting Responses of the Bridge Deck
The buffeting responses of the YDLSB were calculated at the design wind velocity of 39.9 m/s at the bridge deck for the return period of 100 years with the initial wind attack of 0°. The power spectra of vertical, horizontal, and torsional buffeting displacements are also calculated to investigate the frequencies of buffeting response. Figure 8 - Figure 13 shows the time histories and power spectra of vertical, horizontal, and torsional buffeting dis-placements of the midspan and quarter span point at the bridge deck. The following can be seen:

a) In PSD of torsional buffeting displacement, the first and second peaks correspond to the first symmetric lateral bending modes and the first symmetric torsional mode of the bridge deck. The torsional buffeting displacement at the girder midspan mainly depends on the coupling effect of the bridge deck’s lateral bending and torsional modes.

b) In PSD of bending buffeting displacement, the first and second peaks correspond to the first symmetric and asymmetric bending modes, respectively. Thus, the bending buffeting displacement at the girder midspan mainly depends on bending modes.
4.2.3 Check calculation in the frequency domain

The buffeting analysis program for the suspension bridge in the frequency domain was downloaded from the MATLAB website by a toolkit written by Professor E. Cheynet. This program is a typical SRSS method to calculate the buffeting response of a suspension bridge. This program ignores the aerodynamic and modal coupling effects.
It can be seen from the comparison of the time-frequency domain PSD that the peak of the PSD in the time-frequency domain corresponds, but the time-domain method is superior to the frequency-domain method.

Because the frequency-domain method only selects a few fundamental frequencies of the structure to calculate the response of the structure, so the high-order mode is directly ignored. The time-domain method reflects the influence of all modes on the structural response and considers the modal coupling effect, which leads to the conclusion that the response of the frequency domain method is smaller than that of the time domain method.

4.3 Prediction of wind-induced fatigue

The wind pressure data is analyzed using rainflow technique to establish the fatigue characteristics of the suspender. Then, the wind-induced fatigue damage is calculated utilizing Miner’s rule and Goodman’s method. Goodman’s method can be employed to approximately account for the effect of mean load. Wind-induced fatigue damage, \( D \), can be estimated using the well-known damage accumulation hypothesis of Miner’s rule:

\[
D = \sum_{i=1}^{k} \frac{n_i}{N_i} \tag{1}
\]

where, \( n_i \) is the total number of wind load cycles in the ith block of constant pressure range, \( S_{ri} \); \( N_i \) is the number of cycles to failure under \( S_{ri} \), and \( k \) is the total number of blocks. Failure occurs when \( D \) reaches 1.

The number of cycles to failure \( N_i \) can be obtained from the constant amplitude S-N curve of the corresponding material. By applying Goodman’s simplification to estimate the pressure range equivalent to nonzero mean pressure, the conventional S-N relationship becomes

\[
S_e = S \frac{1}{1 - S_m / S_b} \tag{2}
\]

where \( S_m \) is the mean value of the stress time history, \( S_b \) is the material’s ultimate tensile strength. When \( S_m > 0 \), it is equivalent to the larger equivalent stress amplitude, which is adverse to the fatigue life of the structure.

The buffeting response is a random load spectrum that can be transformed into load cycles by the rainflow counting method. The constant amplitude stress amplitude and the average stress amplitude are obtained to calculate the fatigue damage.

\[
D = \sum D_i = \sum \frac{n_i}{N_i} = \frac{1}{10^6} \sum n_i (S_e)^m \tag{3}
\]

Assume that the fatigue life of the bridge is consistent with the Weibull distribution. The fatigue life reliability function of the structure derived from 27 types of structure details experiment.
\[ R(N) = \exp \left\{ -\frac{1.6628 \times 10^6}{A S_r^{m}} \right\}^{1.801} \]

It can be seen from the stress time history diagram that the farther away from the mid-span, the greater the average stress of the suspender, but the maximum stress amplitude is located in the mid-span. Therefore, it is impossible to determine the critical factor of fatigue failure between the average stress and the stress amplitude.

The stress time histories of the suspender on the leeward side and the windward side are symmetrical, and the peak and valley values correspond to each other. From comparing the stress response of the suspender under different wind attack angles, the fatigue of the suspender is the most unfavorable when the wind attack angle is \(-3^\circ\) because the stress range is more significant than others.
The conclusion is that the suspender near the tower of the leeward side is the critical point of fatigue damage. When the structure reaches the design wind speed, the fatigue life reliability of the suspender can reach 97.2%.

Because of the limited time, this paper only completed the fatigue damage estimation when the structure reaches the design wind speed. A more comprehensive method of assessing the fatigue life of a structure requires the joint probability distribution function of wind speed and wind direction, which requires structure health monitoring systems to get wind property at the bridge site. Because of the
limitation of the finite element model, the research object of this paper is the suspender. The subsequent research will use the multi-scale model to study the fatigue life of each component of the steel truss beam.

5. CONCLUSIONS
Based on the results and discussions presented above, the conclusions are obtained as below:

(1) The first mode of YDLSB is symmetric lateral bending, which will significantly affect the lateral displacement of the structure under wind load. The lateral response RMS of the main beam is in the shape of a 1st asymmetric lateral bending, which also has the most significant influence on the lateral response of the main beam. The peaks correspond to the lateral bending modes in PSD of torsional buffeting displacement, which shows the bridge's strong coupling effect.

(2) In this paper, the buffeting response of the bridge is calculated by time domain and frequency domain. The PSD peak of the two methods is consistent with the natural vibration frequency of the bridge. However, the response of the frequency-domain method is smaller than that of the time-domain method. Because the time-domain method reflects the influence of all modes on the structural response and considers the modal coupling effect.

(3) The wind pressure data is analyzed using rainflow technique to establish the fatigue characteristics of the suspender. The fatigue of the suspender is the most unfavourable when the wind attack angle is -3°. The tower of the leeward side is the critical point of fatigue damage. When the structure reaches the design wind speed, the fatigue life reliability of the suspender can reach 97.2%.

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