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COMPUTER SIMULATION OF BUFFETING RESPONSES FOR LONG-SPAN CONTINUOUS RIGID-FRAME BRIDGE WITH OVER-HEIGHT PIER

HU JUN

School of Civil Engineering and Architecture, Chongqing Jiaotong University, Chongqing 400074, China

E-mail: hjisincsu@163.com

ABSTRACT

The continuous rigid frame bridges are more sensitive to wind action with the increase of bridge span and pier height. In order to calculate the wind induced buffeting responses, the fluctuating wind field at bridge site are simulated by harmonic synthesis method and the wind loads are also derived based on the simplified quasi-steady theory, the time-history dynamic responses of the bridge are calculated based on the computer software. The Hezhang bridge with 195m height pier is discussed in detail and the results indicate that the bridge with over-height pier is sensitive to wind action, the maximum displacement responses of construction state and the completion stage are 8.09cm and 3.39cm respectively and the fluctuating amplifying coefficient of construction phase is larger than the finished stage, the bridge is more sensitive to wind load during the construction stage.

Key words: *continuous rigid-frame bridge; numerical simulation; time-domain analysis; buffeting*

1 INTRODUCTION

The continuous rigid frame bridges are more sensitive to wind action with the increase of bridge span and pier height, the buffeting responses have gradually become an important issue that can not be ignored especially in the mountainous area. However, the wind-induced buffeting researches are mainly discussed in long-span cable supported bridges, the continuous rigid frame bridges are rarely concerned[1,2].

Chen[3] discussed the buffeting responses of Humen suspension bridge and its fatigue reliability of buffeting is also derived by assuming the fatigue life is in accord with the Weibull distribution; Gu[4] discussed the Yangpu bridge's buffeting responses, however the nonlinear factors can't be considered in his study; Xu[5] analyzed the Tsing Ma suspension bridge's buffeting responses in time domain and discussed the fatigue damage by continuum damage mechanics method. However, these current researches are mainly related to cable supported bridges, the long-span continuous rigid frame bridge with pier height of nearly 200m is rarely concerned.

In this paper, section 2 presents the computer simulation of fluctuating wind field, section 3 presents the buffeting wind load model for time history analysis, section 4 gives an detailed

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calculation example and section 5 gives a conclusion to the whole paper.

2 NUMERICAL SIMULATION OF FLUCTUATING WIND FIELD OF CONTINUOUS RIGID-FRAME BRIDGE

According to the bridge girder, the wind velocities of along-wind and vertical directions(*u*, *w*) are simulated, and the wind speed of cross-wind direction(*v*) is ignored; also only the wind velocity of along-wind direction(*u*) is simulated for the bridge piers. The correlations of wind speeds between piers are ignored because their distances are large, also the correlations of wind speeds between girder and piers are ignored to simplify the analysis.

Thus, the three dimensional correlated random wind field of large span continuous rigid-frame bridge can be simplified as several independent random processes. For the simulation of one dimensional multivariate random process, the harmonic synthesis method is adopted in this paper. The power spectral density function matrix of one dimensional *n* variables with zero mean Gauss random process can be expressed as:

$$S(\omega)=\begin{bmatrix} s_{11}(\omega) & s_{12}(\omega) & \cdots & s_{1n}(\omega) \\ s_{21}(\omega) & s_{22}(\omega) & \cdots & s_{2n}(\omega) \\ \cdots & \cdots & & \cdots \\ s_{n1}(\omega) & s_{n2}(\omega) & \cdots & s_{nn}(\omega) \end{bmatrix} \quad (1)$$

where, $S_{pq}(\omega)$ （ $p=1,2,\cdots n$ ； $q=1,2,\cdots n$ ） is the fourier transform of correlation function, $S_{pp}(\omega)$ is the dower spectral density, $S_{pq}(\omega)$ （ $p \neq q$ ） is the cross spectral density function, ω is the circular frequency.

The Cholesky decomposition of $S(\omega)$:

$$S(\omega)=H(\omega)H^{T*}(\omega) \quad (2)$$

Where, $H(\omega)$ is lower triangular matrix, $H^{T*}(\omega)$ is the complex conjugate transpose matrix of $H(\omega)$.

Therefore, According to Shinozuka's research, the algorithm for simulating the multivariable random process can be expressed as Equation 3 below:

$$f_j(t)=\sqrt{2}\sum_{k=1}^{j}\sum_{l=1}^{N}\left|H_{jk}(\omega_{kl})\right|\sqrt{\Delta\omega}\cos[\omega_{kl}t+\Phi_{kl}], \quad (3)$$
$$j=1,2,,m$$

Where N is a sufficiently large positive integer; Φ_{kl} is the random phase angle which is a uniformly distributed random number from 0 to 2π, ω_{kl} is the frequency, $\omega_{kl}=(l-1)\Delta\omega+\Delta\omega k/m$; Δω is the frequency increment; H_{jk} is the element of matrix H(ω); H(ω) is the Cholesky decomposition matrix of S(ω); S(ω) is the power spectral density function of fluctuating wind and Kaimal spectra is used here. For the random process simulation of Equation 1, the FFT technology can be used to improve simulation efficiency.

3 BUFFETING WIND LOAD MODEL FOR TIME HISTORY ANALYSIS

The effect of wind load on the bridge section are shown in figure 1.

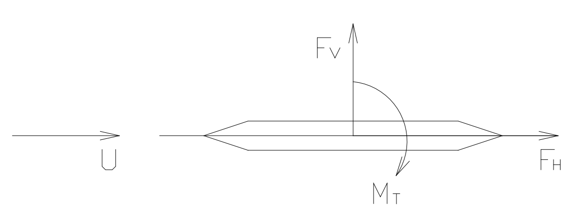

Fig.1 Wind effect on the bridge deck

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Wind load is generally composed of three parts, i.e.:

$$\{R(t)\} = \{F_{st}(t)\} + \{F_b(t)\} + \{F_{se}(t)\} \quad (4)$$

Where, $\{F_{st}(t)\}$ is the static wind load induced by the average wind; $\{F_b(t)\}$ is the buffeting load induced by the fluctuating wind; $\{F_{se}(t)\}$ is the self-excited force induced by average wind.

The static wind loads are generally described by the aerodynamic force coefficient, the static wind loads of the main beam's per unit length can be expressed as:

$$\begin{aligned} F_H &= \frac{1}{2}\rho U^2 C_H(\alpha) D \\ F_V &= \frac{1}{2}\rho U^2 C_V(\alpha) B \\ M_T &= \frac{1}{2}\rho U^2 C_M(\alpha) B^2 \end{aligned} \quad (5)$$

Where, U is the average wind speed, $C_H(\alpha)$, $C_V(\alpha)$ and $C_M(\alpha)$ are the girder section's resistance, lift and pitch moment coefficients under wind attack angle α respectively.

Based on the quasi-steady theory, the buffeting forces of bridge deck can be expressed as:

$$\begin{aligned} L(x,t) &= 0.5\rho U^2 B[2C_V \frac{u(x,t)}{U} + (C_V^{'} + C_H)\frac{w(x,t)}{U}] \\ D(x,t) &= 0.5\rho U^2 B[2C_H \frac{u(x,t)}{U} + C_H^{'} \frac{w(x,t)}{U}] \\ M(x,t) &= 0.5\rho U^2 B^2[2C_M \frac{u(x,t)}{U} + C_M^{'} \frac{w(x,t)}{U}] \end{aligned} \quad (6)$$

Where, $u(x,t)$, $w(x,t)$ are fluctuating wind components of along-wind and vertical directions; $L(x,t)$, $D(x,t)$, $M(x,t)$ are the lift, drag and torque buffeting forces.

In time domain buffeting analysis, the self-excited forces mainly use Scanlan's expression based on step function or Lin's expression based on impulse response function. However, the two kinds of methods involve the fitting of aerodynamic derivatives and the convolution form solving of self-excited forces, both are very complicated and time-consuming. NAMINI [6] proposed a quasi-steady self-excited aerodynamic force model, and gain the two-variable Taylor expansion of the model, based on the finite element theory, the introduction of space beam element displacement interpolation function and the principle of virtual work, the self-excited aerodynamic stiffness matrix and aerodynamic damping matrix can be got. Su [7], Han [8] all adopt this aerodynamic model, and verified its practicality and accuracy. Based on this model, the element aerodynamic stiffness matrix and element aerodynamic damping matrix can input via the Matrix27 element of ANSYS software.

4 CALCULATION EXAMPLE ANALYSIS

4.1 Project overview

The Hezhang bridge in Guizhou province of China is a large span continuous rigid-frame bridge with over-height pier, the arrangement of bridge spans is 96+180+180+96m and the height of the middle pier is up to 195m. The arrangements of Hezhang bridge are shown in Figure 2.

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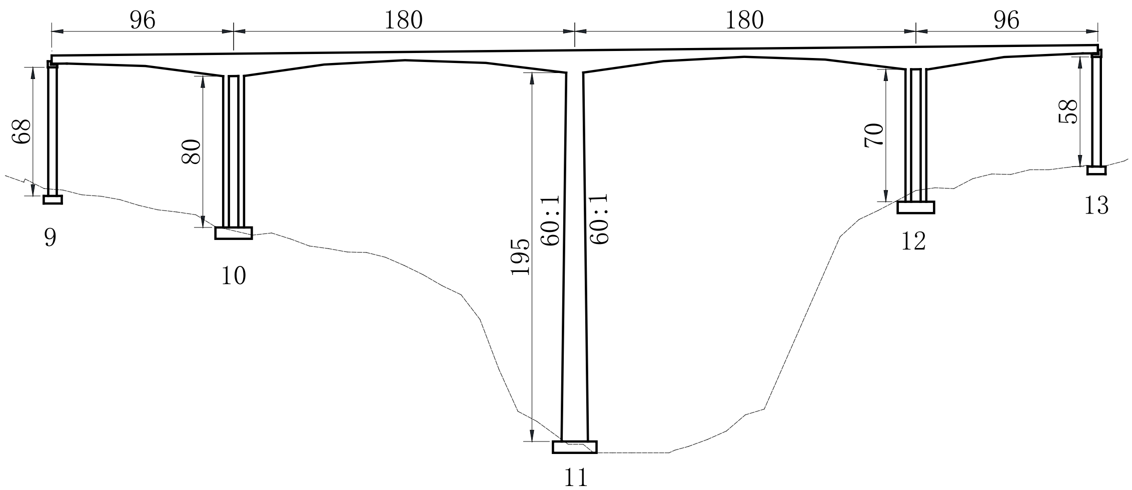
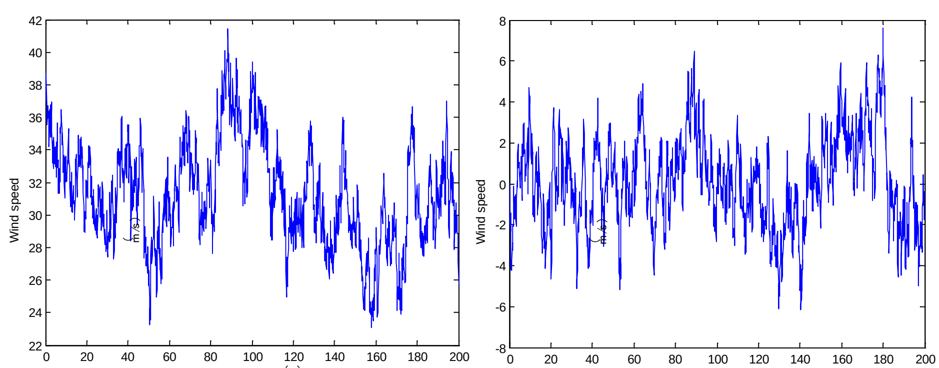


Fig 2. The layout of Hezhang bridge(unit:m)

4.2 Computer simulation results of bridge buffeting responses

It can be assumed that the average wind velocities along the main beam are same and the three dimensional stochastic wind field can be simplified as several one-dimensional stochastic wind field.

The mean wind speeds within the atmospheric boundary layer follows the power exponent law along the vertical height and the surface roughness coefficient takes 0.15. Fluctuating wind power spectral density functions take Kaimal functions, fluctuating wind coherence functions take Davenport form. The mean wind speed at height takes 34.12m/s, 120 points' fluctuating wind speeds are simulated; the results of wind speeds at middle main span of main girder are shown in figure 3, also the results of wind speeds at bridge pier are shown in figure 4. To verify the reliability of the simulated results, the simulated wind velocity's power spectrum are compared with the target spectrum in order to see whether they are consistent with each other, the comparisons are shown in figure 5. The results indicate that the simulated wind speed spectrum is consistent with the target spectrum, the spectrum is very close to the target spectrum, the simulated results are reasonable.



(a) along-wind direction *(b) vertical direction*

Fig 3. Simulated fluctuating wind speeds at middle span of main girder

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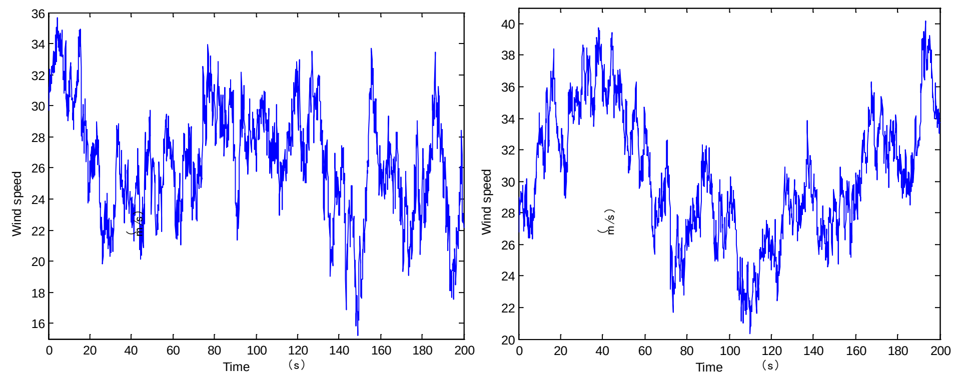

(a) *middle point of the pier* (b) *top of the pier*

Fig 4. Simulated along-wind fluctuating wind speeds of bridge pier

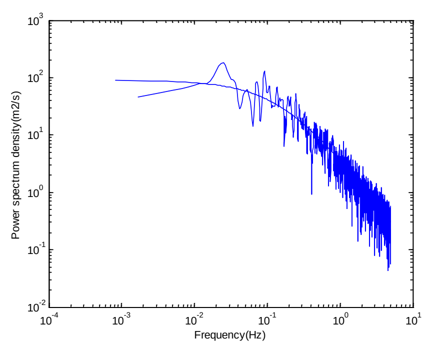

Fig 5. Comparisons of power spectrum of fluctuating wind

The finite element model of the continuous rigid-frame bridge is established in ANSYS software, where the girder and the bridge pier are established by space beam element, see figure 6. Considering the large deformation and stress stiffening effect, the full method is used to do the transient analysis, the simulated buffeting loads are shown in figure 7.

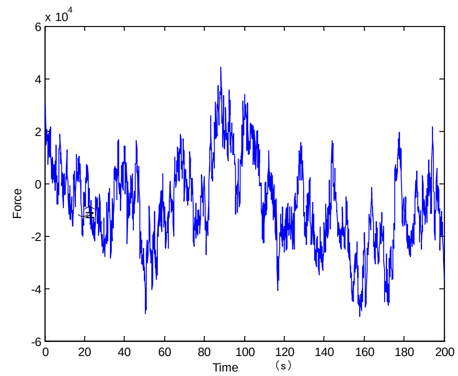

Fig.7 Simulated buffeting load

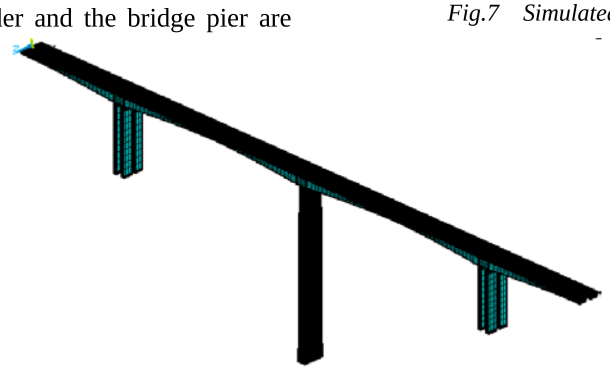

Fig 6. Finite element model

Based on these finite element model, the wind-induced responses results are derived, and the buffeting responses of construction state are shown in Table 1, the results of finished state are shown in Table 2.

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Table 1. Wind-induced responses results of construction state

Type of response	Static wind response	Buffeting response	Total response	Amplifying coefficient of buffeting
Horizontal displacement of main girder(cm)	5.63	2.46	8.09	1.44
Vertical displacement of main girder(cm)	0.3550	0.4036	0.7586	2.14
Torsional displacement of main girder(rad)	4.08×10^{-4}	1.61×10^{-4}	5.69×10^{-4}	1.39
Bending moment of pier's bottom (kN-m)	405414	160036	565450	1.39

Table 2. Wind-induced responses results of finished state

Type of response	Static wind response	Buffeting response	Total response	Amplifying coefficient of buffeting
Horizontal displacement of main girder(cm)	1. 89	1.50	3.39	1.79
Vertical displacement of main girder(cm)	0.15890	0.1016	0.2605	1.64
Torsional displacement of main girder(rad)	3.89×10^{-4}	1.80×10^{-4}	5.69×10^{-4}	1.46
Bending moment of pier's bottom (kN-m)	160120	123497	283617	1.77

The results indicate that the fluctuating amplifying coefficient is between 1.39 and 2.14 under the maximum cantilever construction stage and between 1.46 and 1.79 under finished stage. Thus, the fluctuating amplifying coefficient of construction phase is larger than the completion stage, the bridge is more sensitive to wind load during construction stage.

5. CONCLUSIONS:

Based on computer simulation method, the buffeting responses of a long-span continuous rigid-frame bridge with over-height pier are discussed in detail, the conclusions are as follows:

1) The present bridge buffeting time-domain analysis theory is introduced, and the simplified quasi-steady buffeting force formula is given based on the quasi-steady theory.

2) According to the harmonic synthesis method, the random processes of fluctuating wind speeds are simulated, and the wind loads are also derived based on the simplified quasi-steady theory, the finite element of the bridge is established in ANSYS software and the full method is used to do the transient analysis.

3) The maximum displacement responses of construction state and the finished stage are 8.09cm and 3.39cm respectively, and the maximum bending moment at pier bottom of construction state and the finished stage are 565450kN-m and 283617kN-m

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respectively.

4) The fluctuating amplifying coefficient of the maximum cantilever construction stage is between 1.39 and 2.14, and the average is 1.77; the fluctuating amplifying coefficient of the bridge completion phase is between 1.46 and 1.79, and the average is 1.62. Thus, the fluctuating amplifying coefficient of construction phase is larger than the completion stage, the bridge is more sensitive to wind load during construction stage.

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