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Aerodynamic stability of cable-stayed bridges under erection*

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Abstract: In this work, nonlinear multimode aerodynamic analysis of the Jingsha Bridge under erection over the Yangtze River is conducted, and the evolutions of structural dynamic characteristics and the aerodynamic stability with erection are numerically generated. Instead of the simplified method, nonlinear multimode aerodynamic analysis is suggested to predict the aerodynamic stability of cable-stayed bridges under erection. The analysis showed that the aerodynamic stability maximizes at the relatively early stages, and decreases as the erection proceeds. The removal of the temporary piers in side spans and linking of the main girder to the anchor piers have important influence on the dynamic characteristics and aerodynamic stability of cable-stayed bridges under erection.

Key words: Cable-stayed bridge, Erection stage, Aerodynamic stability, Nonlinear multimode aerodynamic analysis
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INTRODUCTION

Long-span cable-stayed bridges under service and particular construction conditions are very susceptible to wind action due to their great flexibility, so the wind stability (aerodynamic stability or flutter stability) is becoming a major concern in the design and construction phrases. As compared with the service condition, although the period of erection is not too long, the structural stiffness of cable-stayed bridges under erection is greatly reduced, and consequently they become very susceptible to the dynamic wind action, particularly at the stages with maximum single and double cantilevers. Many engineering projects across straits or rivers are being planned around the world in the 21st century, and many long and super long span cable-stayed bridges are proposed, for example, the 1088 m center span Sutong Bridge over the Yangtze River in China. The effect of increasing the span length on the wind stability of cable-stayed bridges under erection will be more prominent, and should therefore be comprehensively investigated.

In previous studies, much attention was paid to the wind stability of long-span suspension bridges under erection, and some countermeasures for enhancing the wind stability of long-span suspension bridges under erection were studied (Cobo del Arco, 2001; Ge and Tanaka, 2000b; Larsen, 1995; Tanaka, 1998; Zhang *et al*., 2003b). On the contrary, few investigations have been conducted on the wind stability of cable-stayed bridges under erection. Moreover, in previous studies, the simplified method based on airfoil aerodynamic theory was commonly used to evaluate the aerodynamic stability of cable-stayed bridges under erection. In fact, the simplified method is based on the coupling of two degrees of freedom including the vertical bending and torsion motions, and may not be suitable for application to the multimode coupled flutter of cable-stayed bridges (Zhang *et al*., 2003a). In addition, the effect of structural

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deformation due to the static wind action on the dynamic characteristics and the aerodynamic response (called as the aerostatic effect herein) was also neglected. Zhang *et al*. (2002; 2003a; 2003b)'s studies showed that the aerostatic effect importantly influences the dynamic characteristics and the aerodynamic stability of long-span bridges during erection and after completion. As cable-stayed bridges under erection are very flexible, the aerostatic effect becomes prominent and should be considered in the aerodynamic stability analysis.

In this work, taking the Jingsha Bridge over the Yangtze River as example, the evolution of critical wind speed with erection was numerically investigated by nonlinear multimode aerodynamic analysis that yielded some important conclusions.

METHOD AND SOLUTION PROCEDURE OF NONLINEAR MULTIMODE AERODYNAMIC ANALYSIS

Currently, the multimode method is commonly used in the aerodynamic stability analysis of long-span bridges (Jain *et al*., 1996; Namini, 1992; Scanlan and Jones, 1990; Ge and Tanaka, 2000a). As cable-stayed bridges under erection are very flexible and exhibit strong geometric nonlinearity, how to accurately determine the static equilibrium position of each erection stage becomes an important problem. In this work, the static equilibrium position of each erection stage was determined by the iteration method, in which the tensional forces in the stays are gradually adjusted until the calculated static equilibrium position converges. Regarding the calculated static equilibrium position of each erection stage, the dynamic characteristics were analyzed by the subspace iteration method, after which the aerodynamic stability analysis was finally conducted. Regarding the three-dimensional finite element model of bridge structures, a method of nonlinear multimode aerodynamic analysis was presented, in which the geometric nonlinearity and the aerostatic effect were both considered, and a solution procedure BSNASA was also developed. Based on the solution procedure BSNASA, an approach for predicting the critical wind speeds for the cable-stayed bridges under erection was established. The computational flow was as follows:

1. Input the geometric and physical data of the bridge, the aerostatic coefficients, the aerodynamic derivatives, etc.

2. Following the erection scheme, determine the static equilibrium position of each erection stage by the procedure described below.

(1) Establish a three-dimensional finite element model of the bridge at the current stage.

(2) Assume an initial tensile force of each stay (usually a very small value).

(3) Apply the corresponding dead load of the current stage to the bridge, and determine the calculated static equilibrium position by three-dimensional geometric nonlinear analysis.

(4) Check if the calculated static equilibrium position is convergent. If not, the tensional forces in the stays are adjusted according to the values obtained in Step (3), and then repeat Step (3), until the calculated static equilibrium position converges.

3. Regarding the calculated static equilibrium position at the current stage, aerodynamic stability analysis was conducted by BSNASA. The computing flow is described as follows:

(1) Compute the current wind speed U_{cur}, starting with the initial wind speed U_{low} and incrementing by U_{inc}.

(2) Predict the aerostatic equilibrium position under current wind speed by three-dimensional nonlinear aerostatic analysis (Zhang *et al*., 2002).

(3) Regarding the deformed aerostatic equilibrium position, structural dynamic characteristics were analyzed by the subspace iteration method, and determine the modes to be subjected to aerodynamic stability analysis.

(4) Calculate the effective wind attack angle according to the deformation obtained in Step (2), then recalculate the aerodynamic stiffness matrix and the aerodynamic damping matrix to account for the nonlinear and three-dimensional effects of the aerodynamic force. Using the changed dynamic characteristics in Step (3), establish the aerodynamic modal equation and solve it to predict the aerodynamic response under the current wind speed.

(5) Check if the aerodynamic response is divergent. If not, repeat Steps (1)–(4) until the aerodynamic critical condition is reached.

4. Repeat Steps 2–3 for all the erection stages.

DESCRIPTIONS OF THE JINGSHA BRIDGE AND ITS ERECTION STAGES

The Jingsha Bridge is a three-span cable-stayed bridge with a 500 m center span and two 200 m side spans as shown in Fig.1. The deck is Π-shaped, 27.0 m wide and 2.0 m high. The H-shaped towers are 137 m high. Two cable planes are inclined and fan-shaped. In the following analysis, the bridge is idealized to a three-dimensional finite element model, in which the columns and transverse beams of towers and the girder are modeled by 3D beam elements, and the stays are modeled by 3D bar elements. The deck is idealized to a three-girder finite element model. The connections between the bridge components and the supports are properly modeled. Fig.2 shows the three-dimensional finite element model of the completed bridge. The flutter derivatives and the aerostatic coefficients during erection and after completion were obtained from the sectional model test of the bridge (Song, 1998). Structural damping was taken as 1.0%.

In the following sections, seven erection stages as shown in Fig.3 were selected for analysis, and described as follows:

Stage A: Double 100 m long cantilevers, and 24 stay cables in each cable plane.

Stage B: Single 138 m long cantilever, and 32 stay cables in each cable plane; the side span ends are linked to the temporary piers.

Stage C: Single 250 m long cantilever, and 62 stay cables in each cable plane, with the side span ends not being linked to the anchor piers.

Stage D: Single 250 m long cantilever, with the side span ends being linked to the anchor piers.

Stage E: The center span is closed, and the temporary piers are retained.

Stage F: The center span is closed, and the temp-

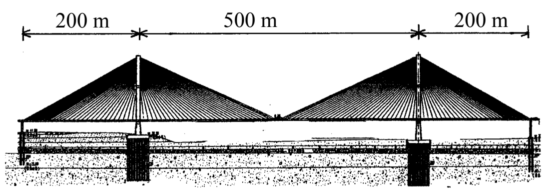

Fig.1 General view of the Jingsha Bridge over the Yangtze River

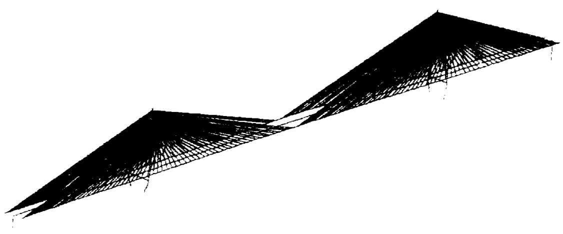

Fig.2 Three-dimensional finite element model of the completed Jingsha Bridge

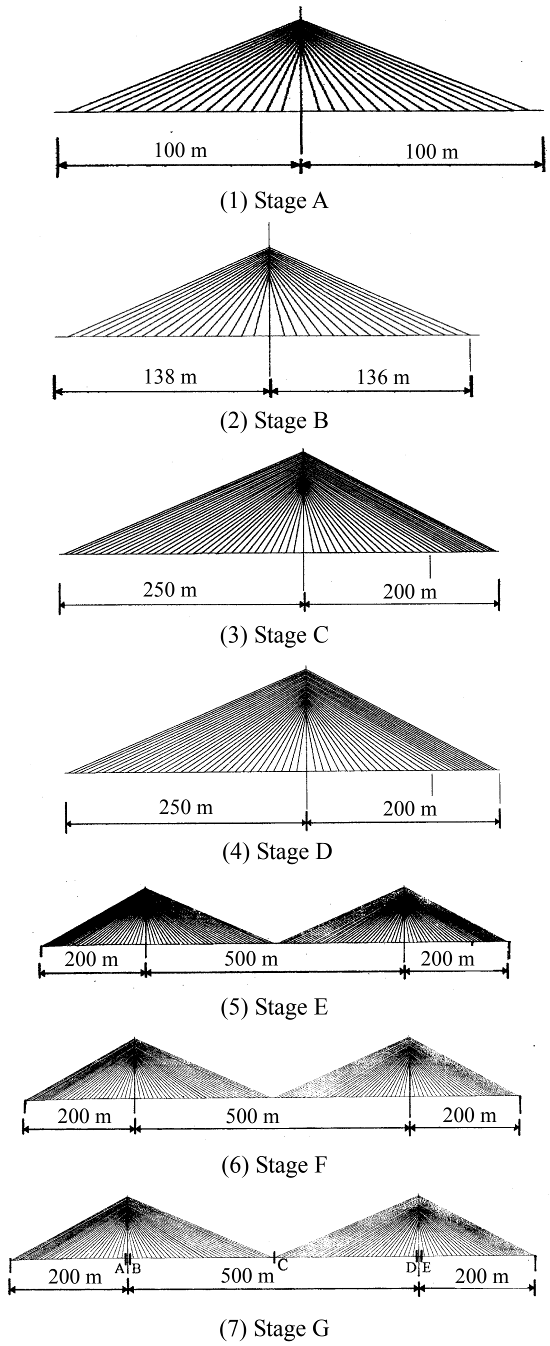

Fig.3 Seven erection stages of the bridge

orary piers are removed, but the deck surfacing has not been paved.

Stage G: The completed bridge.

The constraint condition of the bridge during erection is set as follows: the girder is fixed to the towers; the lateral and vertical motion, and the rotations about the longitudinal and vertical axis of the girder are subordinated to the temporary piers and the anchor piers; but the longitudinal motion and the rotation about the lateral axis are left free. After completion of the bridge, the girder functions as a floating system, and within the tower sections the lateral motion is subordinated to the towers.

DYNAMIC CHARACTERISTICS ANALYSIS

Regarding the calculated static equilibrium position of each erection stage, structural dynamic characteristics were analyzed by BSNASA based on the subspace iteration method, and the evolutions of the modal frequencies with erection were numerically generated.

Table 1 shows the frequencies of the fundamental symmetrical vertical bending and torsional modes obtained by BSNASA, which are compared with those obtained by the computer program SAP5 (Song, 1998).

Table 1 shows that the results obtained by the computer program BSNASA are very close to those from SAP5, thus verifying the accuracy of structural dynamic characteristics analysis.

The evolution of the fundamental vertical bending frequency tended to be irregular. When the temporary piers in the side spans were removed (from stage E to stage F), the frequency decreased remarkably from 0.3066 to 0.1713, and the decrement amplitude reached 44.2%. Similarly, from stage C to stage D, when the main girder was linked to the anchor piers, the frequency increased greatly from 0.227 to 0.3033, and the increment amplitude reached 33.6%. Therefore, the removal of the temporary piers in the side spans and the linking of the main girder to the anchor piers had significant influence on the fundamental vertical bending frequency.

The fundamental torsional frequency, however, decreased gradually as the erection proceeded. The removal of the temporary piers in the side spans and linking of the main girder to the anchor piers also influenced the fundamental torsional frequency, although the influence was not remarkable.

Table 1 Frequencies of the fundamental symmetrical vertical bending and torsional modes

Erection stage	The fundamental vertical bending frequency (Hz)		The fundamental torsional frequency (Hz)	
	BSNASA	SAP5	BSNASA	SAP5
A	0.2476	0.2450	0.6210	0.6112
B	0.3161	0.2917	0.6049	0.5961
C	0.2270	0.2075	0.4160	0.4160
D	0.3033	0.2801	0.4509	0.4540
E	0.3066	0.3032	0.4453	0.4404
F	0.1713	0.1987	0.3738	0.3996
G	0.1817	0.1819	0.3911	0.3851

AERODYNAMIC STABILITY ANALYSIS

The results (Table 2) of aerodynamic stability analysis of the bridge under the wind attack angle of +3° during erection were compared with those obtained by the simplified method where the formula of critical wind speed is expressed as

$$U_f = \eta_s \eta_\alpha \left[1 + (\varepsilon - 0.5)\sqrt{r / 0.72\mu b} \right] \omega_b b$$

where η_s is the modified coefficient of cross section shape; η_α is the modified coefficient of wind attack angle; ε is the fundamental torsion to vertical bending frequency ratio; r is the rotational radius of the deck cross section; b is the half width of the deck; μ is the density ratio of the bridge to air; ω_b is the angular frequency of the fundamental vertical bending mode. η_s and η_α of the bridge were taken as 0.39 and 0.91

Table 2 The critical wind speeds during erection (m/s)

Erection stage	BSNASA	The simplified method
A	139.9	98.9
B	114.6	96.4
C	74.3	66.3
D	79.1	71.8
E	78.5	70.9
F	67.6 (77.0)	59.5
G	74.5 (82.0)	57.0

Note: The results in the bracket were obtained from the sectional model test (Song, 1998) respectively (Xiang, 1999).

Table 2 shows that the critical wind speeds obtained from BSNAFA were all greater than those from the simplified method, and that there were large differences between the two cases. The fact could have resulted from the multimode coupling effect, which may play an important role in the aerodynamic stability of cable-stayed bridges, but was ignored in the simplified case. As found from the analysis, the modal shapes of the bridge under erection became so complex that the similarity among the modes was greatly reduced. Besides the fundamental vertical bending and torsional modes, more modes such as the fundamental lateral bending and the 2nd vertical bending modes participate in flutter as shown in Table 3. The positive effect of multimode coupling improves the aerodynamic stability of the bridge under erection. Therefore, the multimode coupling effect should be exactly considered in the aerodynamic stability analysis of cable-stayed bridges under erection.

Table 3 Main modes participating in aerodynamic analysis

Erection stage	Main modes participating in aerodynamic analysis
A	1-V, 1-T, 1-L, 2-V
B	1-V, 1-T, 1-L, 2-V
C	1-V, 1-T, 1-L, 2-V
D	1-V, 1-T, 1-L, 2-V
E	1-S-V, 1-S-T, 1-S-L, 2-S-V
F	1-S-V, 1-S-T, 1-S-L, 2-S-V
G	1-S-V, 1-S-T, 1-S-L, 2-S-V

Note: V–vertical bending; T–torsion; L–lateral bending; S–symmetric

Table 2 shows that the aerodynamic stability maximizes at the relatively early stages, and decreases gradually as the erection proceeds. The critical wind speed was all higher than the design flutter speed of 36.12 m/s during erection and 43 m/s after completion, and thus the bridge was aerodynamically stable. The way the critical wind speed evolved was very similar to that of the torsional frequency in Table 1. Similarly, the removal of the temporary piers in the side spans and linking of the main girder to the anchor piers also had important influence on the critical wind speed. For example, when the temporary piers in the side spans were removed (from stage E to stage F), the critical wind speed decreased from 78.5 m/s to 67.6 m/s, and the decrement amplitude reached 13.9%. In the same way, from stage C to stage D, when the main girder was linked with the anchor piers, the critical wind speed increased from 74.3 m/s to 79.1 m/s, and the increment amplitude reached 6.5%.

CONCLUSION

1. In contrast to the simplified method, the nonlinear multimode aerodynamic analysis was confirmed to be effective for predicting the aerodynamic stability of cable-stayed bridges under erection.

2. As the erection proceeds, the fundamental torsional frequency decreased gradually. The removal of the temporary piers in side spans and linking of the main girder to the anchor piers had importance influence on the fundamental vertical bending frequency. But the influence on the fundamental torsional frequency was not remarkable.

3. The aerodynamic stability of cable-stayed bridges maximizes at the relatively early stages, and decreases gradually as the erection proceeded. The removal of the temporary piers in the side spans and the linking of the main girder to the anchor piers also had important influence on the critical wind speed.

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