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Full Scale Measurement on Structural Dynamic Characteristics and Vortex-induced Vibrations of a Long-span Suspension Bridge

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ABSTRACT

Full scale measurement on the structural dynamic characteristics and Vortex-induced Vibrations (VIV) of a long-span suspension bridge with a central span of 1650m were conducted. Different Finite Element (FE) modeling principles for the separated twin-box girder were compared and evaluated with the field vibration test results, and the double-spine model was determined to be the best simulation model, but certain modification still needs to be made which will affect the basic modeling parameters and the dynamic response prediction values of corresponding wind tunnel tests. Based on the FE modal analysis results, small-scaled and large-scaled sectional model tests were both carried out to investigate the VIV responses and significant Reynolds Number effects on VIV responses were discovered. Based on the observed VIV modes in the field measurement, the VIV results obtained from sectional model tests were converted into those of the three-dimensional (3D) full-scale bridge and subsequently compared with field measurement results. Well matched results verify the effectiveness of large-scaled sectional mode tests on VIV prediction.

1. INTRODUCTION

Structural dynamic characteristics are fundamental parameters when conducting dynamic response analysis or wind-tunnel tests on wind-induced aero-elastic performance of long-span bridges. In the current procedure of wind-resistant design for long-span bridges, structural dynamic characteristics are calculated by Finite Element Methods (FEM) and further used to determine fundament parameters of spring-supported sectional model or full-bridge aero-elastic model wind tunnel tests. Consequently, any deviation of these calculated values from the dynamic characteristics of full-scale bridges may lead to errors in the predictions of structural aero-elastic behaviors.

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Currently, single-spine, double-spine and triple-spine beam element models and shell element model have been developed to model bridge decks with different configurations in FEM analysis, which were discussed in detailed (Zhu et al. 2000). Even there is an optimal deck model for bridges with different deck sections or different structural systems, inevitable deviation of dynamic characteristics exists between FE models and full-scale bridges (Ma & Ge, 2014). For this purpose, field vibration tests, like ambient vibration test or forced vibration test, have been conducted to identify the actual modal properties and check the baseline of FE models. Modal analysis and ambient vibration tests have been both carried out for several suspension bridges with super-long span, such as the Akashi Kaikyo Bridge in Japan (Katsuchi et al. 2004, 2006), the Great Belt Bridge in Denmark (Tanaka, 1998; Brincker et al. 2000) and the Runyang Bridge in China (Wang et al. 2010). About 10% lower natural torsional frequencies compared with field measurement results were discovered in the modal analysis of the Akashi Kaikyo Bridge, which was attributed to the ignorance of the deck stiffness of the composite girder. For Runyang Bridge, the natural frequencies of lateral bending modes and torsional modes calculated with a single-spine beam element model are much lower than those measured from ambient vibration tests. Above mentioned comparative results all indicate that there are more or less differences in the natural frequencies of long-span bridges between FEM analysis and full scale measurement. Moreover, frequently observed fluctuating modal damping ratios on site are quite different from the damping ratio which was set as a certain value in spring-supported wind tunnel tests. These all imply that some modifications should be adopted in the predictions of structural aero-elastic behaviors from the collaborative work of theoretical analysis and wind tunnel test.

As a kind of structural aero-elastic behaviors, Vortex-induced Vibrations (VIV) has attracted more and more attention in wind engineering to achieve better serviceability of bridges. Wind tunnel test is considered to be a reliable way to predict VIV response provided that the experimental situation can adequately simulate full-scale conditions, including structural parameters (modal frequencies, damping ratios) and flow condition around the more or less bluff body, which, however, can hardly be realized perfectly. In the present paper, field vibration tests on structural dynamic characteristics were carried out for a suspension bridge with a separated twin-box girder in China. Modal analysis results were compared with field testing results, and different modeling principles of FE model were compared and evaluated. Small scale and large scale girder sectional model tests and full scale measurements on VIV responses were both conducted. The influence of structural parameters and Reynolds Number effects on VIV response prediction are finally discussed.

2. TWIN-BOX GIRDER SUSPENSION BRIDGE

Connecting two large cities, Ningbo and Zhoushan, the Zhoushan Island-Mainland Connection Project has provided a major traffic line between China mainland and the archipelago city, and Xihoumen Bridge is the key part of this project, crossing the Xihoumen Channel, one of the most important deep waterways. A very long span is required in order to minimize bridge and environmental risks due to ship collision, technical complexity and unpredictable costs in constructing deep-water foundation.

These requirements result in a suspension bridge with a 1650 m main span with steel box girder (Fig. 1), which has created a world record for box-girder suspension bridges.

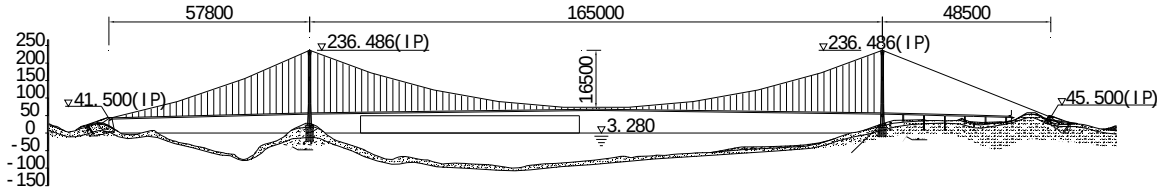


Fig. 1 Elevation of Xihoumen Bridge

Based on the experience gained from the 1624 m Great Belt Bridge in Denmark with the critical flutter speed of 62 m/s and the 1490 m Runyang Bridge in China with the flutter speed of 65 m/s, the span length of 1500 m seems to be its intrinsic limit in the aspect of aerodynamic instability for box-girder suspension bridges, even with the more strict stability requirement of 78.7 m/s for Xihoumen Bridge. After comprehensive evaluation through theoretical analysis and wind tunnel tests, a twin box girder with 6m central vent was selected as the final scheme, as shown in Fig. 2.

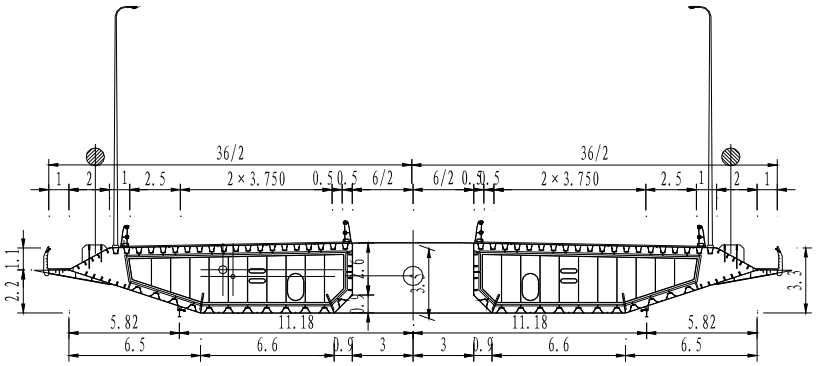


Fig. 2 Twin-box girder section of Xihoumen Bridge

While the adoption of twin box girder can be an effective solution to improve the flutter stability of box-girder suspension bridges, more attention should be paid to the VIV performance of this girder section, since the existence of central vent will make vortex shedding process and its effect on structures more complicated. Besides, it is more challenging to acquire accurate modal analysis results through FEM analysis due to the special layout of separated twin box girder.

3. STRUCTURAL DYNAMIC CHARACTERISTICS

3.1 FE model

In the three-dimensional FE model of Xihoumen Bridge, pylons and piers are modelled by beam elements with six degrees of freedom (DOFs) at each node. The main cables and the suspenders are simulated by three-dimensional linear elastic truss

elements with three DOFs at each node. Both the back cables and the towers are fixed at the base.

The modelling of bridge deck is more challenging for its typical separated twin-box section. The traditional single-spine model best suited for single box girder may present complexities in the calculation of equivalent stiffness. The deviation of the mass centroid of the whole girder section from the shear center of each single box will cause another simulation problem concerning axis positioning. Special attention was therefore paid to the establishment of a better FE deck model. Totally four kinds of deck models, including single-spine model (SG) with axis located at the mass centroid of the whole girder section, double-spine model with axes located at the mass centroid of each separated box (DG1), double-spine model with axes located at the shear center of each separated box (DG2) and four-spine model with axes both at the mass centroid and the shear center of each separated box (FG), are compared and evaluated. Fig. 3 illustrates the details of different FE deck models. The crossbeams are simulated by traditional 3D beam elements.

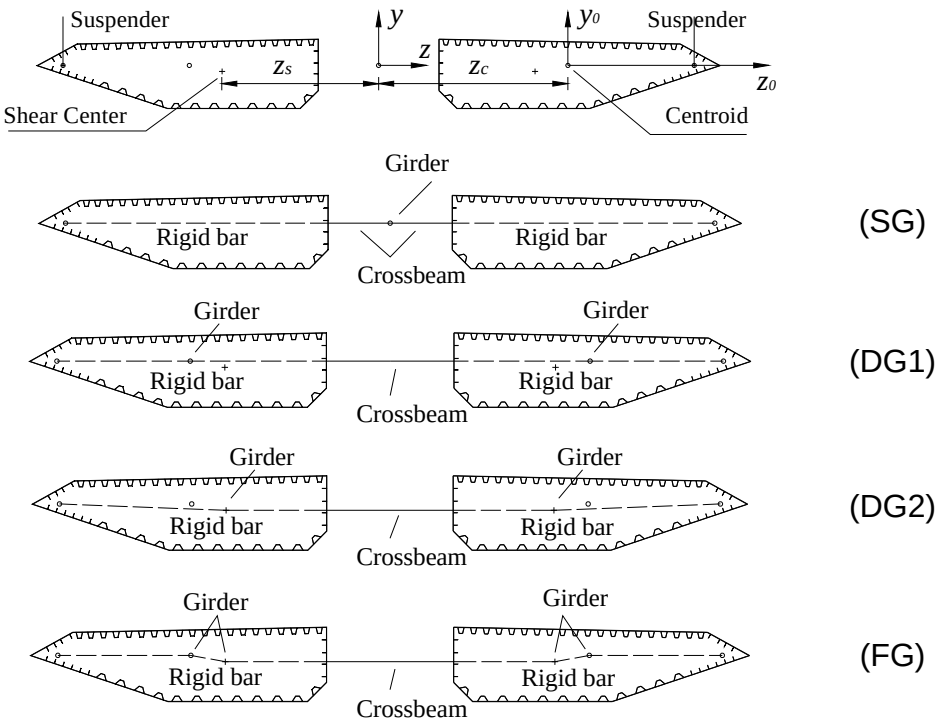


Fig. 3 Modeling details of different deck models

The principal parameters of different deck models are listed in Table 1. Parameters A, I_{z0}, I_{y0}, J_d represent the area, the vertical bending moment of inertia, the lateral bending moment of inertia, the free torsional moment of inertia of each separated single box girder, respectively. m and I_m denote the mass and the mass moment of inertia per unit length (along the bridge longitudinal axis) around its mass centroidal axis for each separated box section, respectively.

Table 1 Structural parameters of different deck models

Parameter	SG	DG1	DG2	FG
Axis Location	($y=y_c$, $z=0$)	($y=y_c$, $z=z_c$) ($y=y_c$, $z=-z_c$)	($y=y_s$, $z=z_s$) ($y=y_s$, $z=-z_s$)	($y=y_c$, $z=z_c$, $z=-z_c$) ($y=y_s$, $z=z_s$, $z=-z_s$)
Tensile Rigidity	$2EA$	EA	EA	EA(Centroid)
Vertical Bending Stiffness	$2EI_{z0}$	EI_{z0}	EI_{z0}	EI_{z0} (Centroid)
Lateral Bending Stiffness	$2(EI_{y0}+EAz_c^2)$	EI_{y0}	EI_{y0}	EI_{y0} (Centroid)
Torsional Stiffness	$2GJ_d$	GJ_d	GJ_d	GJ_d(Shear center)
Distributed Mass	$2m$	m	m	m(Centroid)
Distributed Mass Moment of Inertia	$2(I_m+mz_c^2)$	I_m	$I_m+m(z_c^2-z_s^2)$	I_m (Shear center)

Note: subscripts *c* and *s* represent mass centroid and shear center, respectively.

Based on the FE models developed above, modal analysis was performed and principal results of natural frequencies are shown in Table 2. These results indicate that DG1 is equivalent to FG. For twin-box section with boxes connected by crossbeams, the real lateral bending stiffness ranges between $2EI_{y0}$ and $2(EI_{y0}+EAz_c^2)$. It can be observed in Table 1 that SG overestimates the lateral bending stiffness, which results in overestimated lateral vibration frequencies. But for DG2, underestimated lateral bending frequencies are obtained compared with DG1. As shown in Fig. 3, compared with the mass centroid of each single box, the shear center is closer to the mass centroidal axis of the whole girder section ($z_s<z_c$), which results in a smaller lateral bending stiffness and hence a lower frequency of each lateral bending mode. It can be concluded that SG and DG2 are inappropriate in lateral stiffness modelling for twin box girders. On the other hand, vertical bending frequencies show little differences between different deck models. The difference in torsional frequencies between these deck models can be attributed to the different simulation of section warping effects since the mass moment of inertia of the whole twin-box girder section for different deck models are the same, as shown in Table 1. The warping effects have not been considered in SG, while for either DG1 or DG2, the warping stiffness is represented by the vertical bending stiffness of each single box times the square of the horizontal distance between the axis of each spine and the center of the whole girder section, which lead to higher torsional frequencies. Compared with DG2, the contribution of section warping effect is higher for DG1 model due to the fact that $z_s<z_c$. In summary, DG1 model can be recommended to model separated twin-box girder for its reasonability and simplicity in FE modelling.

Table2 Modal analysis results of Xihoumen Bridge (Units: Hz)

Mode Shape	SG	DG1	DG2	FG
1-S-L	0.0487	0.0485	0.0469	0.0485
1-AS-L	0.1106	0.1094	0.0998	0.1094
1-AS-V	0.0787	0.0791	0.0791	0.0791
1-S-V	0.1005	0.1005	0.1005	0.1005
1-S-T	0.2145	0.2321	0.2284	0.2321
1-AS-T	0.2132	0.2371	0.2312	0.2371

Note: S-symmetric, AS-asymmetric, L-lateral bending, V-vertical bending, T-torsion

3.2 Field vibration test

The modal testing and measurement of a bridge on site provide accurate and reliable predictions of its global modal parameters, including natural vibration modes, natural frequencies and modal damping ratios. In order to verify the FE modal analysis results and to identify the structural damping, field vibration tests consisting of both forced vibration test and ambient vibration measurement were carried out for the Xihoumen Bridge.

3.2.1 Forced vibration test

The forced vibration test was conducted particularly for several vertical bending modes since heaving VIV was observed in previous wind tunnel investigation. The excitation of the forced vibration test was generated by a group of 200 workers, who located at mid-span, 1/4 span, 1/8 span and jumped at a specific frequency with the help of metronomes. The modal damping ratios were identified with Logarithmic Decrement (LD) method. The measured frequencies, damping ratios of totally four vertical bending modes are listed in Table 3.

3.2.2 Ambient vibration test

The ambient vibration test of the Xihoumen Bridge was performed right after the completion of bridge deck pavement and guard rail installation (Ge & Yang 2011). The wind speed during the test ranged from 2m/s to 8m/s, while the temperature varied between 25℃ and 30℃. A wireless vibration testing and data acquisition system was employed for vibration identification of the deck and the main cables. A rather dense measurement layout on the deck in the vertical and lateral direction was employed to identify better vibration mode shapes of the bridge. As a result, there are totally 50 vertical and 25 lateral accelerometers set on the bridge deck. Besides, 14 vertical and 7 lateral accelerometers are located along the main cables, and 18 measuring points on the bridge pylon. The detailed layout of the measurement points on the deck and the main cables are shown in Fig. 4.

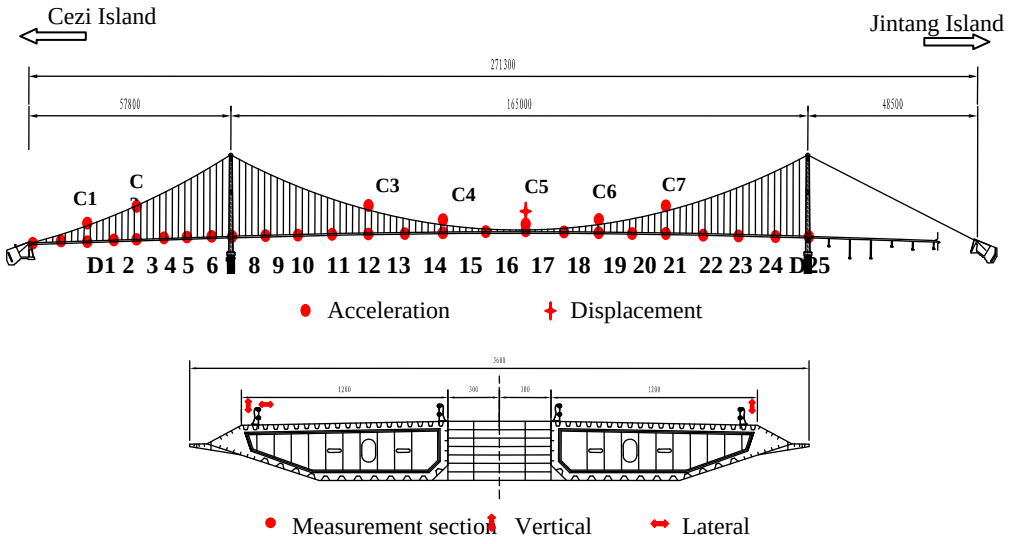


Fig. 4 Layout of measurement points

The Enhanced Frequency Domain Decomposition (EFDD) technique and the Stochastic Subspace Identification (SSI) method in time domain were used to identify modal parameters. The frequencies, mode shapes and damping ratios of totally 32 natural vibration modes were identified in the ambient vibration tests. The measured frequencies and damping ratios of the first modes are listed and compared with the FE analysis results (double-spine model DG1), as shown in Table 3.

Table 3 Comparison of modal parameters of Xihoumen Bridge

No.	Ambient vibration tests			Forced vibration tests		Calculated	Mode shape
	Frequency (Hz)	Damping ratio (%)		Frequency (Hz)	Damping ratio (%)	Frequency (Hz)	
		EFDD Method	SSI Method		LD Method		
1	0.054	1.42~2.60	1.95~2.25	-	-	0.0485	1-S-L
2	0.095	1.12~2.64	1.80~2.18	-	-	0.0791	1-AS-V
3	0.103	1.39~2.33	1.02~1.62	-	-	0.1005	1-S-V
4	0.123	0.82~2.54	1.54~2.02	-	-	0.1094	1-AS-L
5	0.133	0.84~2.32	0.90~1.46	0.135	0.47~0.85	0.1328	2-S-V
6	0.183	0.18~1.02	0.37~0.61	0.180	0.49~0.92	0.1787	2-AS-V
7	0.202	0.38~0.64	0.28~0.48	-	-	0.1868	Cable
8	0.208	0.23~0.57	0.43~0.67	-	-	0.1872	2-S-L
9	0.209	0.22~0.68	0.18~0.60	-	-	0.1955	Cable
10	0.229	0.27~0.65	0.48~0.94	0.230	0.41~1.17	0.2293	3-S-V
11	0.229	0.21~0.59	0.23~0.62	-	-	0.2321	1-S-T
12	0.233	0.25~0.41	0.77~0.95	-	-	0.2371	1-AS-T
13	0.268	0.16~0.94	0.21~0.43	-	-	0.2610	1-AS-V (Side span)
14	0.276	0.34~1.14	0.43~0.83	0.275	0.39~0.46	0.2739	3-AS-V
15	0.327	0.35~0.63	0.36~0.68	-	-	0.3245	4-S-V
16	0.349	0.40~0.82	0.42~0.88	-	-	0.3487	2-S-T
17	0.379	0.09~0.75	0.44~0.72	-	-	0.3751	4-AS-V
18	0.380	0.16~0.60	0.17~0.51	-	-	0.3755	2-AS-L
19	0.418	0.15~0.65	0.28~0.84	-	-	0.4031	1-S-V (Side span)
20	0.435	0.25~0.79	0.21~0.37	-	-	0.4295	5-S-V

Note: S-symmetric, AS-asymmetric, L-lateral bending, V-vertical bending, T-torsion

Generally, the calculated modal frequencies agree well with the measured results except for the lateral bending modes and the first asymmetric vertical mode. The obviously lower calculated frequencies of the lateral bending modes can be probably attributed to the released longitudinal constraints at the sliding bearings, which connect the deck to the tower cross-beam and the abutment. These bearings allow movements along the longitudinal direction of the bridge and rotation around the vertical axis. However, for low-level (ambient) dynamic response, the behavior of these connections can be different (Magalhães et al. 2008). It is always assumed that relative translational motions between deck and tower would not occur and that the only relative motion possible was a free rotation of the deck with respect to tower cross-beam (longitudinal bending) (Wilson and Gravelle, 1991). Furthermore, the longitudinal dampers may possibly contribute to the constraint for the longitudinal movements of the girder.

Therefore, modal analysis was conducted by the modified FE model with longitudinal constraints at each of those bearings. The frequency of the first asymmetric vertical bending mode increased to 0.0939Hz, while the frequencies of the first symmetric and asymmetric lateral bending modes changed to 0.0562Hz and 0.1287Hz, respectively. The refined FE model presents better agreement with the field measurement results.

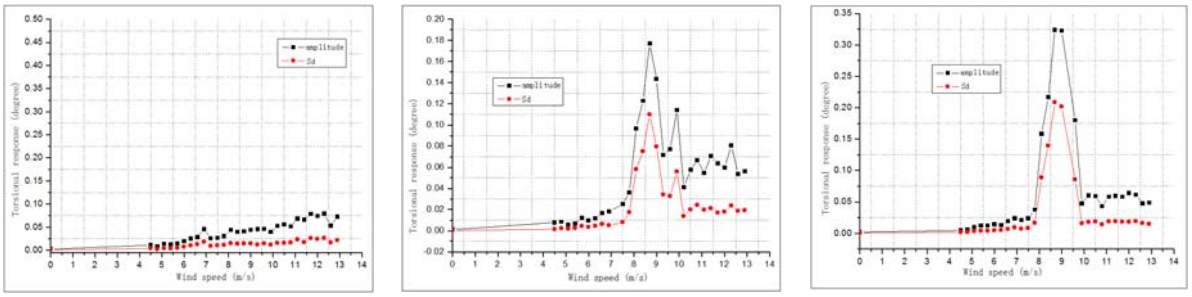
4. VORTEX-INDUCED VIBRATIONS

4.1 Small Scaled Sectional Model Testing

Traditional small scaled girder sectional model wind tunnel test was firstly conducted to evaluate the VIV performance of the twin box girder for the Xihoumen Bridge, as shown in Fig. 5. The geometrical scale is 1:40 and a 1:1.5 wind velocity scale was selected. The equivalent mass and the frequencies (as shown in Table 3) of the first symmetric heaving and torsional modes resulted from FE modal analysis (DG1) were scaled and applied to the sectional model. The damping ratio of the sectional model is 0.45% for the heaving mode and 0.3% for the torsional mode.

Fig. 5 Small scaled sectional model test

No heaving VIV response was detected, while torsional VIV responses were observed at 0° and +3° wind angle of attack, which is shown in Figure 6. The Strouhal Number and Reynolds Number at the VIV condition are 0.1083 and 25,418 respectively.



a) -3° b) 0° C) +3°

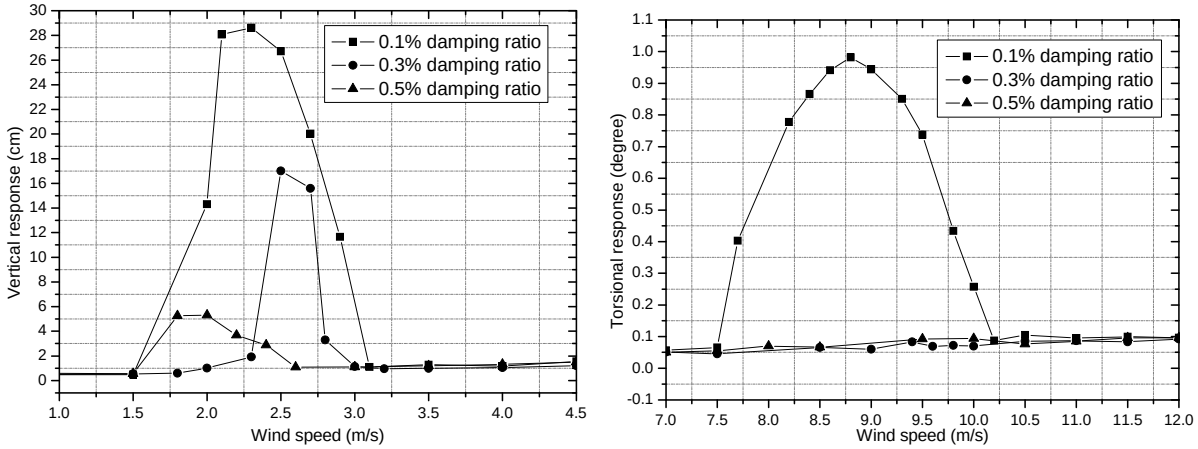
Fig. 6 VIV responses in small scaled sectional model testing

4.2 Large Scaled Sectional Model Testing

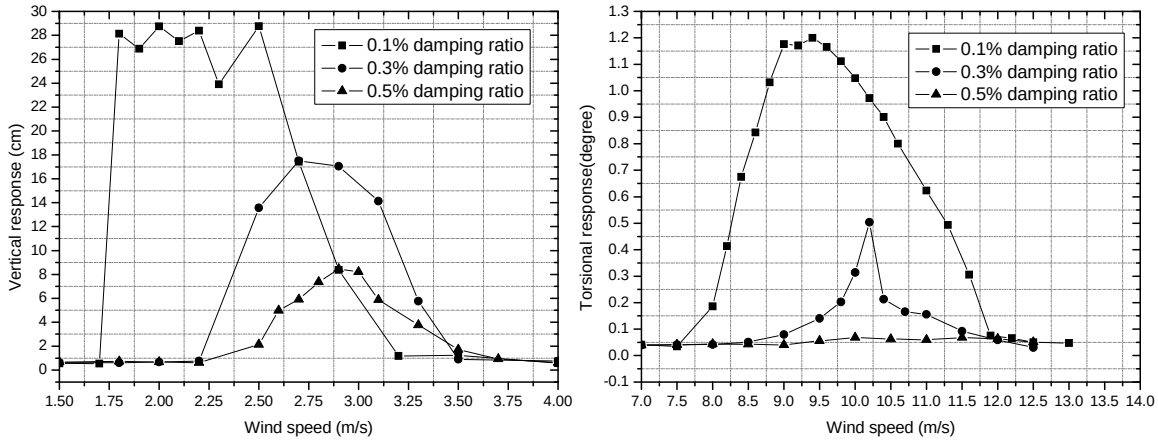
For further detailed investigation on VIV responses, a combination of geometry, mass, stiffness and Reynolds number considerations resulted in the selection of a 1:20 geometrical scale for the sectional model, which has a total length of 3.6m and a width of 1.8m (Yang & Ge 2012). The wind velocity scale of 1:1 was selected. The wind tunnel testing was 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 (Fig. 7), and the suspension system including 8 springs and supporting frames was built in these two end walls together with 4 optical laser displacement transducers for the measurement of the response of the sectional model to vortex-shedding excitation.

Fig. 7 The 1:20 girder sectional model

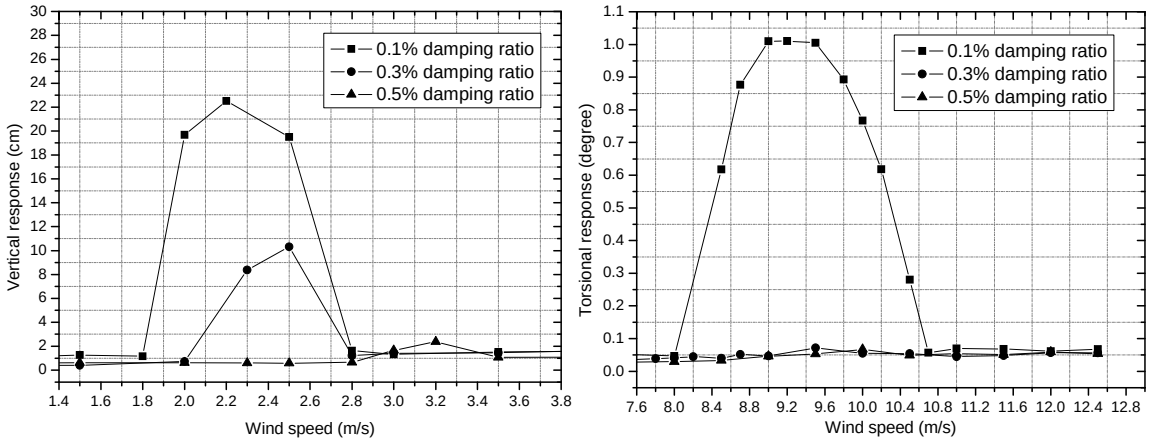
Different from the results obtained from small scaled sectional model testing, the VIV responses in both heaving and torsional DOF were observed in the testing, and the Strouhal Number and Reynolds Number at the VIV condition are 0.1083 and 91,035 respectively. The comparisons between girders with different structural damping ratios at +3°, 0°, -3° wind angle of attack are illustrated in Fig. 8, which indicate that the structural damping has great influence on the amplitude of VIV response.



(a) +3°



(b) 0°



(c) -3°

Fig. 8 VIV responses of girders with different damping ratios

4.3 Field Measurement on VIV responses

Heaving VIV response of twin box girder was observed on site after guard rails were erected on the deck for the first time (Ge & Yang 2011). The vibration process was recorded, and Fig. 9 presents typical screenshots taken from the recorded video.

From the recorded VIV video, the VIV frequency and the oscillation amplitude of the deck near the north pylon can be detected as 0.32Hz and 5.0cm respectively. The VIV mode is probably the 4th symmetric heaving mode according to Table 3, with corresponding damping ratio ranging between 0.35% and 0.63%. Based on the vibration mode shapes, the maximum VIV amplitude of 16.3cm can be estimated as illustrated in Fig. 10.

4006th Frame　　4046th Frame

Fig. 9 Screenshots from videos recording VIV response (near the north pylon)

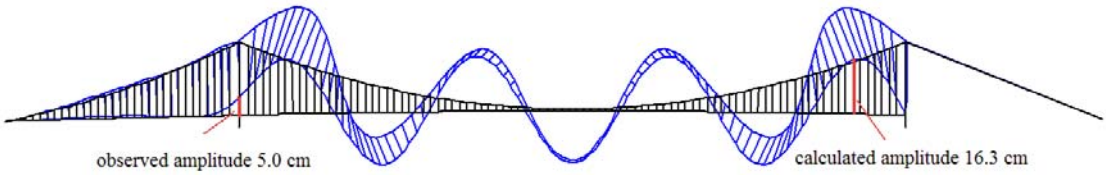


Fig. 10 VIV mode and maximum amplitude estimation

After that, four accelerometers were installed on the girder deck, among which two at mid span, and one at 3/8 and 1/4 span, respectively. The layout of these accelerometers is shown in Fig. 11.

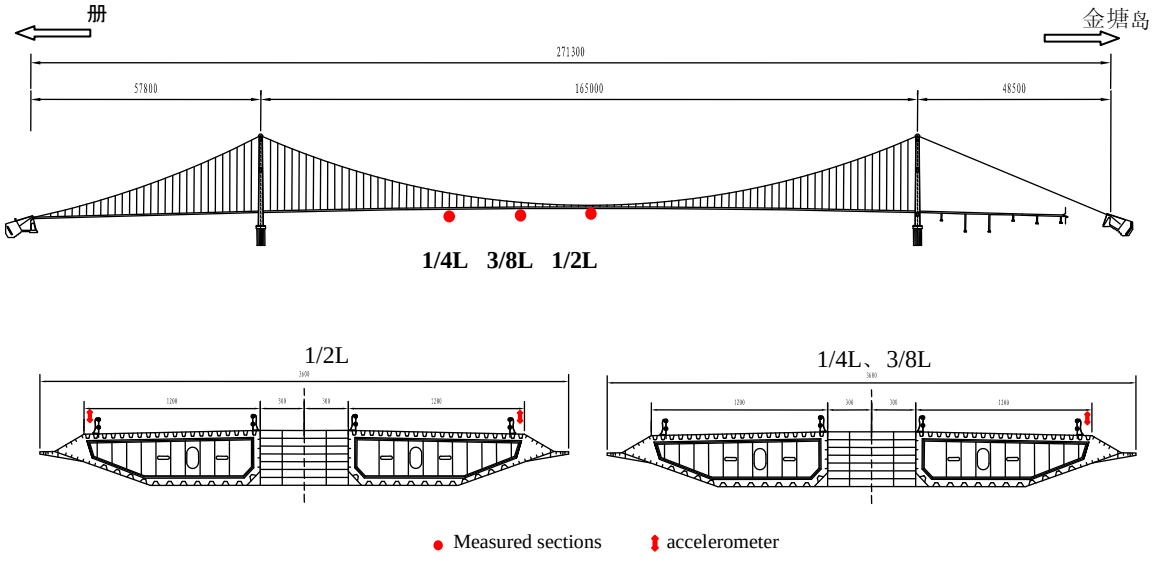


Fig. 11 Layout of accelerometers installed on the deck for VIV observation

Three days later, heaving VIV response lasting about 90 minutes was captured. The

wind speed during that day ranged from 2m/s to 9m/s. The time histories of vertical accelerations at four measuring points on the deck during VIV process and the power spectrum of VIV response at mid span of the deck are shown in Fig. 12. The statistic results of measured vertical accelerations and vertical displacements at these four measuring points are listed in Table 4 and Table 5 respectively. According to these results, the VIV frequency is 0.23Hz and the amplitude reaches 18.4cm.

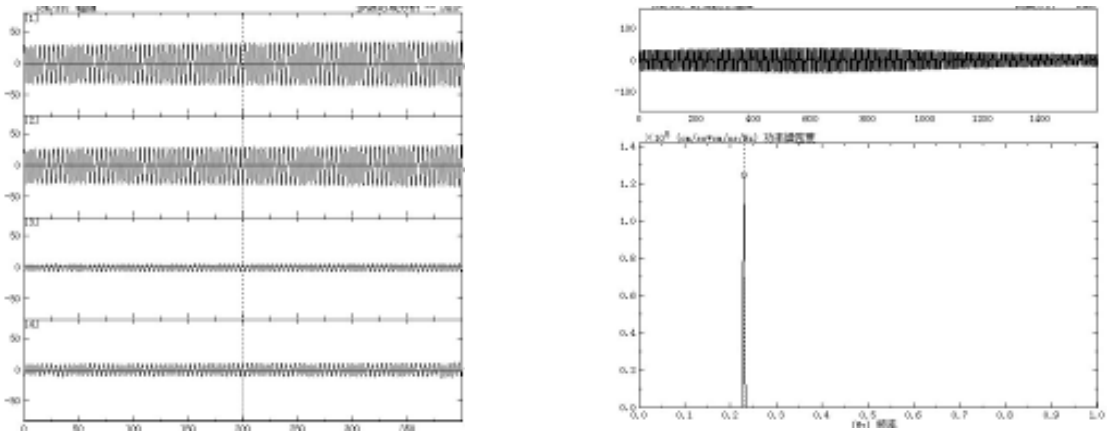


Fig. 12 Results of VIV field measurement

Table 4 Statistic results of measured vertical accelerations on the deck (cm/s2)

Case	No VIV			During VIV		
Measuring Point	Max	Min	RMS	Max	Min	RMS
1 (mid span)	4.6	-4.7	2.5	38.3	-38.4	21.8
2 (mid span)	3.9	-4.2	1.7	36.4	-36.8	21.6
3 (1/8 span)	2.1	-2.2	0.6	9.3	-9.6	5.2
4 (1/4 span)	3.1	-3.6	0.9	12.6	13.2	7.1

Table 5 Statistic results of vertical displacements on the deck (cm)

Case	During VIV	
Measuring Point	Max	Min
1 (mid span)	18.3	-18.4
2 (mid span)	17.4	-17.6
3 (3/8 span)	4.5	-4.6
4 (1/4 span)	6.0	6.3

According to the field measurement results of the structural modal properties shown in Table 3, the observed VIV mode corresponds to the 3rd symmetric heaving mode with modal damping ratio located between 0.27% and 0.65% (EFDD method).

4.4 Discussions

Comparative study between VIV responses obtained from small and large scaled sectional model tests indicates that the Reynolds number effect may significantly

influence VIV amplitude, which suggests that the largest possible geometrical and wind velocity scale in sectional model tests should be adopted for better VIV prediction. Generally speaking, larger VIV amplitudes correspond to higher Reynolds numbers, which could result in unsafe prediction of VIV response from small scaled model test.

To evaluate the effectiveness of large scaled sectional model test on VIV prediction, the test results were compared with the field measurement results. Firstly, the VIV response with respect to the first symmetric heaving mode in the wind tunnel test, as shown in Fig.7, needs to be transformed into those for the 3rd and 4th symmetric heaving modes, which were observed on site as the VIV modes. The equivalent modal mass of the 1st, 3rd and 4th symmetric heaving modes are 27511kg/m, 26320kg/m and 25922kg/m, respectively, which shows negligible differences between each other. Therefore, the influence of the differences in Scruton number (Sc) on VIV amplitude was not taken into account. The frequency scale λ_f and velocity scale λ_V of these two modes relative to the 1st symmetric heaving mode are listed in Table 6. Based on these relative wind speed scale ratio, the lock-in wind speed range of the VIV responses predicted for the 1st symmetric heaving mode has been shifted and enlarged as predicted VIV responses for the 3rd and 4th symmetric heaving mode.

Table 6 Scale ratio to the 1st symmetric heaving mode

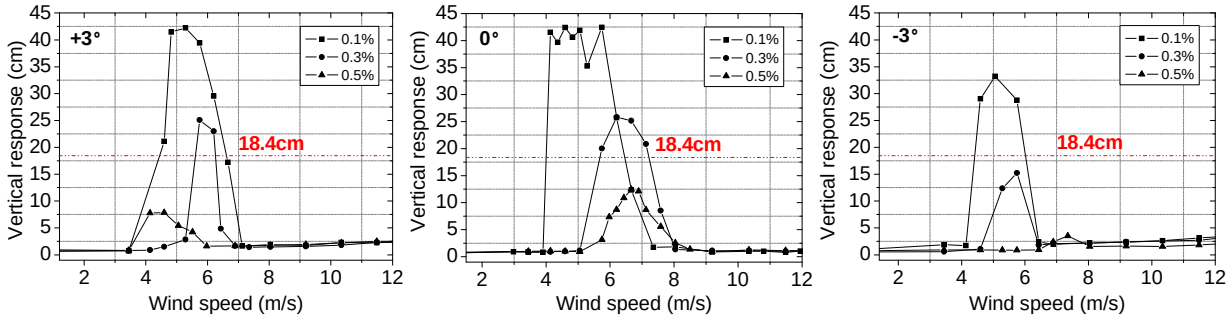
Mode shape	λ_f	λ_V
3-S-V	2.2977	2.2977
4-S-V	3.1968	3.1968

The second step is to convert VIV responses from 2D bridge girder section to 3D full structure. The conversion formula proposed by Zhang (Zhang & Chen 2011) based on the linear model of vortex-induced loading was adopted:

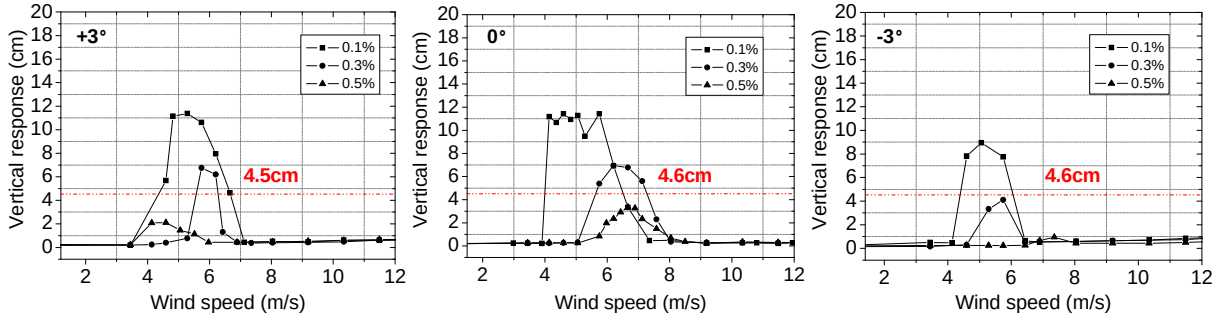
$$\lambda_i = \frac{y_{3D_i}}{y_{2D}} = \varphi(x_i)\frac{\int_0^L |\varphi(x)|dx}{\int_0^L \varphi^2(x)dx} \tag{1}$$

where λ is the conversion coefficient; y represents VIV response; i denotes node number; $\varphi(x)$ is the mode shape function. Based on the results in Table 6 and Eq. (1), the VIV response obtained from the large scaled sectional model was converted into the responses of the 3D full bridge girder with respect to the 3rd and 4th symmetric heaving modes. According to FE modal analysis results, the conversion coefficients of the 3rd symmetric heaving mode are 1.4756, 0.3793 and 0.6748 for nodes located at mid-span, 3/8 span and 1/4 span, respectively, while for the 4th symmetric heaving mode, the conversion coefficient of the node with the maximum amplitude as shown in Fig. 9 is 1.3211.

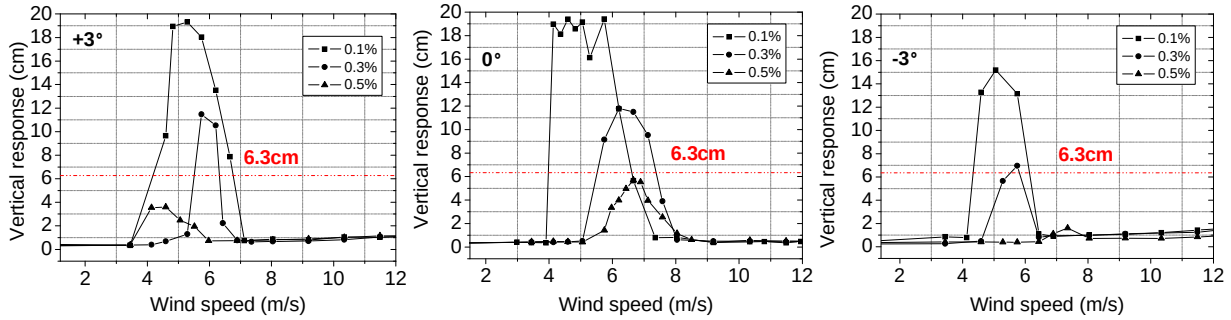
The converted vibration amplitudes and wind speed ranges of VIV for the 3rd and 4th symmetric heaving modes are illustrated in Fig. 13 and Fig. 14, respectively, and compared with corresponding field measurement results. Since the exact wind velocity and wind angle of attack during VIV have not been exactly measured for some difficulties, the measured VIV responses on-site are depicted as red dotted lines covering the whole wind velocity range concerned.



(a) 1/2 span (λ=1.4756)



(b) 3/8 span (λ=0.3793)



(c) 1/4 span (λ=0.6748)

Fig. 13 VIV amplitude with different damping ratios (3-S-V)

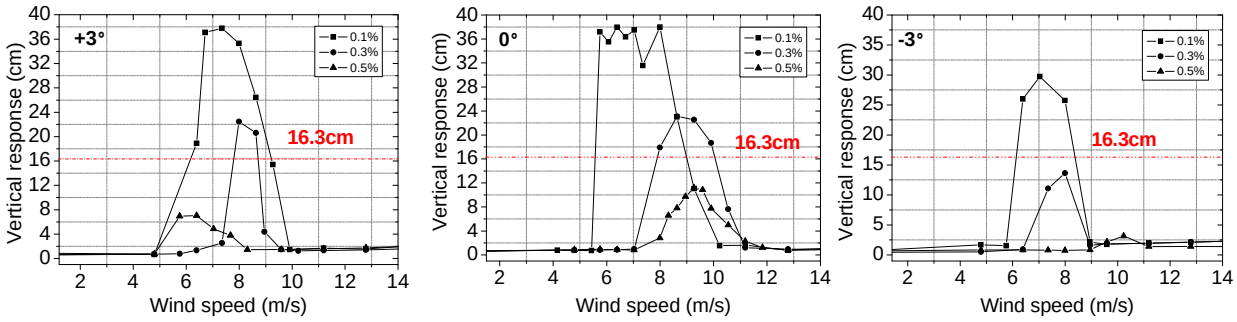


Fig. 14 Maximum VIV amplitude with different damping ratios (4-S-V) (λ=1.3211)

Comparisons between the converted wind tunnel results and field measurement results (Fig. 13) indicate that the structural modal damping ratio of the 3rd symmetric heaving mode is generally between 0.3% and 0.5%, which agree well with the

corresponding field measurement results which ranges from 0.27% to 0.65%. Similar results are presented in Fig. 14 for the 4th symmetric heaving mode. Well matched results verify the effectiveness of large-scaled sectional mode tests on VIV prediction, provided that the basic modelling parameters based on modal analysis are correct, and the conversion from wind tunnel tests to 3D full bridge concerning VIV amplitude and lock-in wind velocity range has been performed appropriately.

5. Conclusions

FE modelling principles of the twin box girder in the Xihoumen Bridge was compared and evaluated by comparison with the on-site identification results. It can be concluded that the separated twin box girder can be best modelled by double-spine deck model with two beam element located at the centroid of each single box, but certain refinement of the FE model still needs to be performed based on field modal identification results, which will affect the basic modelling parameters and hence the dynamic response prediction values of corresponding wind tunnel tests. VIV performance of the twin box girder was investigated through traditional sectional model and large scaled model wind tunnel tests. The Reynolds Number effect or scale effect may significantly affect the occurrence of VIV and corresponding amplitude. Comparative study with field measurement results indicates that the traditional sectional model test may lead to an unsafe prediction in VIV response, while for the large scaled sectional model test, effective prediction of VIV response can be obtained, if the basic modelling parameters based on modal analysis are correct, and the conversion from wind tunnel tests to 3D full bridge can be appropriately realized.

6. ACKNOWLEDGEMENTS

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