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Seismic performance and its favorable structural system of three-tower suspension bridge

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Abstract. Due to the lack of effective longitudinal constraint for center tower, structural stiffness of three-tower suspension bridge becomes less than that of two-tower suspension bridge, and therefore it becomes more susceptible to the seismic action. By taking a three-tower suspension bridge-the Taizhou Highway Bridge over the Yangtze River with two main spans of 1080 m as example, structural dynamic characteristics and seismic performance of the bridge is investigated, and the effects of cable's sag to span ratio, structural stiffness of the center tower, and longitudinal constraint of the girder on seismic response of the bridge are also investigated, and the favorable structural system is discussed with respect to seismic performance. The results show that structural response under lateral seismic action is more remarkable, especially for the side towers, and therefore more attentions should be paid to the lateral seismic performance and also the side towers. Large cable's sag, flexible center tower and the longitudinal elastic cable between the center tower and the girder are favorable to improve structural seismic performance of long-span three-tower suspension bridges.

Keywords: three-tower suspension bridge; structural dynamic characteristics; seismic performance; structural system

1. Introduction

Currently, the widely built suspension bridges are two-tower structures. Into the 21st century, the world's bridge construction entered into a new era of building longer and longer sea-crossing bridges. Suspension bridge is required to cross deep and wide straits, and must have longer spans to reduce the cost of the anchorages and substructures. The multi-tower suspension bridge, which has no sharing anchorage, is one of the most hopeful and rational solutions, and frequently proposed in many sea-crossing bridges. With comparison to the common two-tower suspension bridge, the main spans of three-tower suspension bridge can be greatly shortened, which leads to significant reduction of the tensional forces in cables, the size of anchorages and foundation, and finally the cost. The previous studies also show that under certain condition, the multi-tower suspension bridge is more suitable to the bridge site than other bridge structures, and it is also an economic solution even the span length ranges from 2000 to 3000 m (Gimsing 1997). For the sake of its aesthetics, structural efficiency, and economy of construction, three-tower suspension bridge

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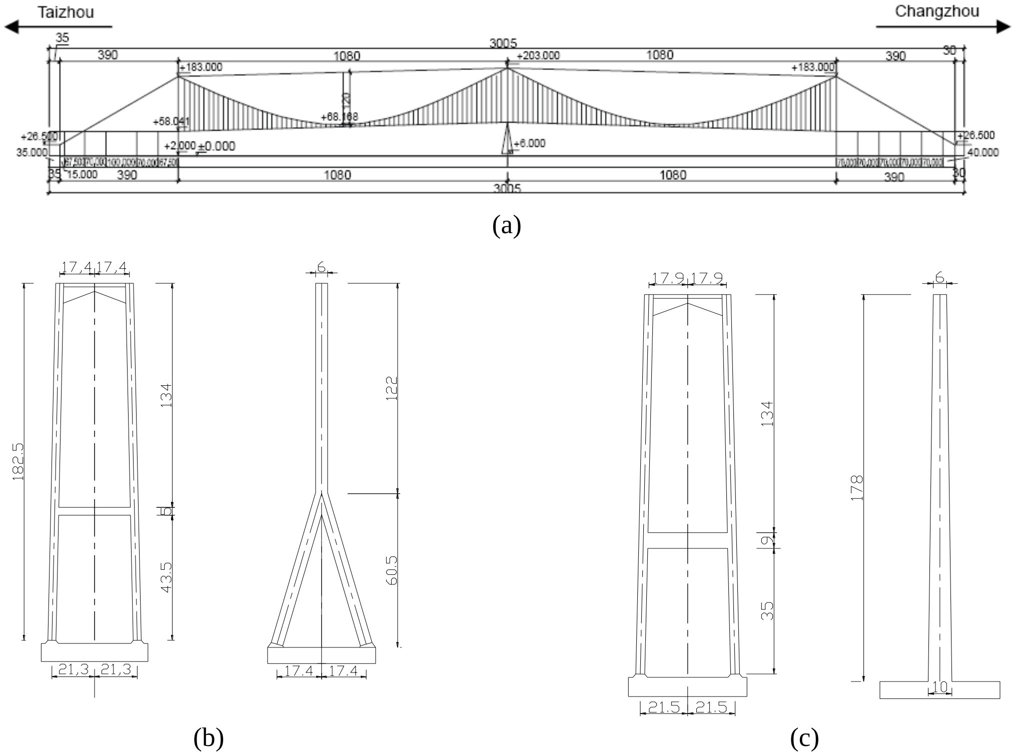


Fig. 1 general layout of the Taizhou Highway Bridge over the Yangtze River (Unit: m): (a) vertical layout of the example bridge, (b) the center tower, (c) the side tower

has attracted great interest in recent years, and also has been proposed frequently in many long-span bridges such as the Chacao Strait Bridge in Chile, the old and new San Francisco-Oakland Bay Bridge in America, the Messina Strait Bridge in Italy, the Gibraltar Strait Bridge in Spain, several highway bridges over the Yangtze River in China including the Wuhang Yangluo Highway Bridge, the fourth Nanjing Bridge, the Maanshan Highway Bridge, the Yingwuzhou Bridge and the Taizhou Highway Bridge etc(Gimsing 1997, Yang, 2009). At present, the Maanshan Bridge, the Taizhou Bridge and the Yingwuzhou Bridge in China designed as three-tower suspension bridges are being constructed (Yang 2009).

On the other hand, the multi-tower suspension bridge has been recognized as a questionable structure due to its large deflection (Forsberg 2001, Fukuda 1975, 1976, Gimsing 1997, Nazir 1986). For three-tower suspension bridge, the two side towers are effectively restrained by the side cables anchored at the anchorages, however the center tower lacks of the effective longitudinal constraint, structural stiffness therefore becomes less than that of the two-tower suspension bridge, and it is considered to be a structural system with greater flexibility, and becomes more susceptible to the dynamic action such as traffic load, wind and earthquake. Up to now, many investigations on structural static and dynamic performance have been conducted by Fukuda (1975, 1976), Nazir (1986), Gimsing (1997), Zhu (2007), Wang (2007), Zheng *et al*. (2009), and Yoshida *et al*. (2004). Some investigations on the wind stability of three-tower suspension bridge have been conducted by Chen (2006), Yoshida *et al*. (2004), Zhu (2007) and Zhang (2008, 2010). Unfortunately, few

Table 1 The material and cross-sectional properties of the example bridge

Members		A/m^2	J_d/m^4	I_y/m^4	I_z/m^4	E/(GPa)	ρ/(kg/m^3)
Girder		1.50	8.44	192.11	2.91	210	13146.67
Cables		0.286	-	-	-	200	7850
Hangers		0.00263	-	-	-	200	7850
Center tower	18*	1.556	7.261	7.203	5.599	210	7850
	60	3.865	29.546	94.633	14.544		
	87	3.160	16.269	32.863	10.178		
	114	3.024	14.070	25.120	9.386		
	143	2.881	11.744	18.121	8.552		
	173	1.924	7.236	8.179	5.798		
Side towers	26	38.172	568.448	442.210	284.638	35	2600
	105	29.56	365.321	293.219	179.918		
	186	28.381	319.278	259.788	146.461		

Note: A = cross-sectional area; J_d = torsional moment of inertia; *Iy* = lateral bending moment of inertia; *Iz* = vertical bending moment of inertia; *E*- elastic module; ρ- mass density; *-height of the tower's sections from the ground.

investigations on the seismic performance of three-tower suspension bridges have been conducted by Jiao *et al.* (2010), Deng *et al.* (2008), and Wang *et al.* (2009).

The present paper focuses its attention on the seismic performance, and also attempts to find a favorable structural system for three-tower suspension bridges. By taking a three-tower suspension bridge-the Taizhou Highway Bridge over the Yangtze River with two main spans of 1080 m as example, seismic performance of the bridge is investigated by MIDAS/Civil finite element analysis, and effects of the cable's sag to span ratio, structural stiffness of the center tower, the longitudinal elastic cable between the center tower and girder and the central buckle between main cables and the girder on the seismic response of the bridge are also investigated, and the favorable structural system is discussed with respect to seismic performance.

2. Description of the example bridge

Fig. 1(a) shows the Taizhou Highway Bridge over the Yangtze River taken as the example bridge herein, which is a three-tower suspension bridge with two 1080 m main spans and two 390 m side spans (Yang *et al.* 2008). Two main cables are formed by prefabricated parallel wire strands, and spaced at 35.8 m, whose sag to span ratio is 1/9; the hangers are made of the galvanized steel wires with intervals of 16 m. The deck is a streamlined steel box girder of 3.5 m deep and 39.1 m wide. The towers all have a door-shaped front view, the side towers are concrete towers with an I-shaped side view as shown in Fig. 1(c), and but the center tower is a steel tower with an inverse Y-shaped side view as shown in Fig. 1(c). The elastic longitudinal cables between the center tower and the girder are installed to restrain the longitudinal displacement of the girder. The material and cross-sectional properties of the bridge are listed in Table 1.

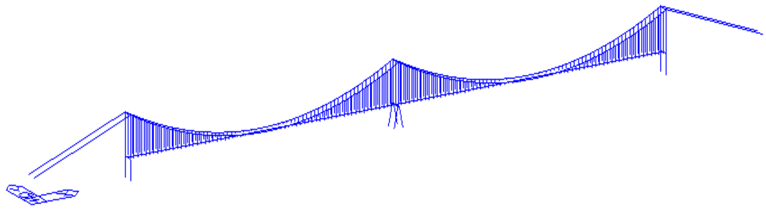


Fig. 2 Three-dimensional finite element model of the example bridge

3. Three-dimensional finite element model

In order to analyze structural dynamic characteristics and seismic response, a three-dimensional model of the example bridge is established on the basis of finite-element program MIDAS/Civil, as shown in Fig. 2. The coordinates system of the model was set as *x*-axis along the longitudinal direction of the bridge, *y*-axis along the lateral direction and *z*-axis in the vertical direction. In the FE model, the deck is simulated by the single-girder model, the deck and towers are simulated by 3D beam elements, main cables and hangers are simulated by 3D truss elements, and rigid diaphragms are provided to simulate the connections between the deck and hangers. The pavement and the railings on the steel box girder were simulated by mass elements without stiffness. The nonlinear stiffness characteristic of the main cables due to gravity effect is approximately simulated by linearizing the cable stiffness using the Ernst equation of equivalent modulus of elasticity. The tops of main cables are fixed on the tops of towers, and the bottoms of main cables are fixed on the anchorages. The bottoms of all the towers are fixed on the earth at the bases. At the side towers, the longitudinal movement, the rotations about the *z*-axis and the *y*-axis of the girder are left free, whereas the other movements and rotations are restricted; at the center tower, except the lateral movement, the other movements and rotations are left free. The longitudinal elastic cables between the center tower and the girder are simulated by spring elements, and their tensile stiffness is 6.4×10^5 kN/m.

4. Structural dynamic characteristics analysis

On the computed equilibrium position of the bridge in completion, the first 200 modes of the example bridge are calculated by the subspace iteration method based on MIDAS/Civil software. Table 2 shows the modal properties of the first 20 modes of the example bridge, which are compared to those obtained by ANSYS software.

It is found from Table 2 that the results obtained by MIDAS/Civil are very identical to those by ANSYS, and consequently the finite element model of the example bridge is verified to be valid, and also some features on structural dynamic characteristics of the bridge can be concluded as follows: (1) the fundamental natural frequency is very small, and contrarily the fundamental period is very long, which demonstrates that the three-tower suspension bridge is also a structural system with great flexibility similarly as the traditional two-tower suspension bridge; (2) the frequency ratio of the fundamental in-plane and out-of-plane modes is 1.318:1, the out-of-plane structural stiffness is less than that in plane, which makes the bridge more susceptible to the lateral action such as wind and earthquake; (3) there are 12 girder-dominated, 6 cable-dominated and 2

Table 2 The modal properties of the example bridge

Modes	Natural frequency(Hz)		Mode description
	MIDAS/Civil	ANSYS	
1	0.0727	0.0716	Girder, AS-L
2	0.0958	0.0802	Girder, AS-V
3	0.0970	0.0951	Girder, S-L
4	0.1469	0.1149	Girder, AS-V
5	0.1544	0.1176	Girder, S-V
6	0.1765	0.1371	Girder, S-V
7	0.1844	0.1709	Girder, AS-V
8	0.2303	0.1852	Girder, S-V
9	0.2341	0.2306	Girder, S-L
10	0.2360	0.2379	Girder, AS-L
11	0.2425	0.2398	Girder, S-V
12	0.2433	0.2451	Center tower, L; Cable, S-L
13	0.3063	0.2729	Cable
14	0.3063	0.2867	Cable
15	0.3063	0.2931	Cable
16	0.3063	0.2931	Cable
17	0.3081	0.2922	Girder, AS-T
18	0.3094	0.2968	Center tower, L
19	0.3166	0.3012	Cable
20	0.3181	0.3059	Cable

Note: *S*=symmetric, *AS*=anti-symmetric, *L*=lateral; *V*=vertical, *T*=torsion.

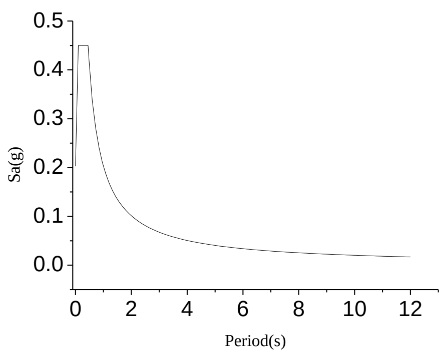


Fig. 3 The design horizontal seismic acceleration response spectrum

tower-dominated modes in the first 20 vibration modes of the bridge, the frequencies of girder-dominated modes are relatively small, and while the frequencies of cable and tower-dominated modes are very large, and consequently the girder is confirmed to be much more susceptible to the dynamic action.

5. Seismic response analysis

5.1 Earthquake ground motion

According the guidelines for seismic design of highway bridges (JTGT B02-01-2008) (Ministry of Communications 2008) and the geological condition of bridge site, a standard response spectrum of Class II field is selected to make response spectrum analysis, the design basic acceleration of ground motion is taken as 0.2g, and under earthquake action E1, the design horizontal seismic acceleration response spectrum is plotted in Fig. 3, and for the vertical design seismic acceleration response spectrum, it is taken as 65% the horizontal seismic acceleration response spectrum. In seismic response analysis, the uniform ground motion is considered.

5.2 Response spectrum analysis

Under the longitudinal, lateral and vertical seismic actions, seismic response of the bridge is analyzed by response spectrum analysis. In the seismic response spectrum analysis, the first 200 modes are considered, and the CQC method is used for modal combination. Structural displacement and bending moment envelope diagrams of the side towers, the center tower and the girder are plotted in Figs. 4, 5 and 6 respectively, the maximum internal forces of the bridge under the longitudinal, lateral and vertical seismic actions are given in Tables 3, 4 and 5 separately.

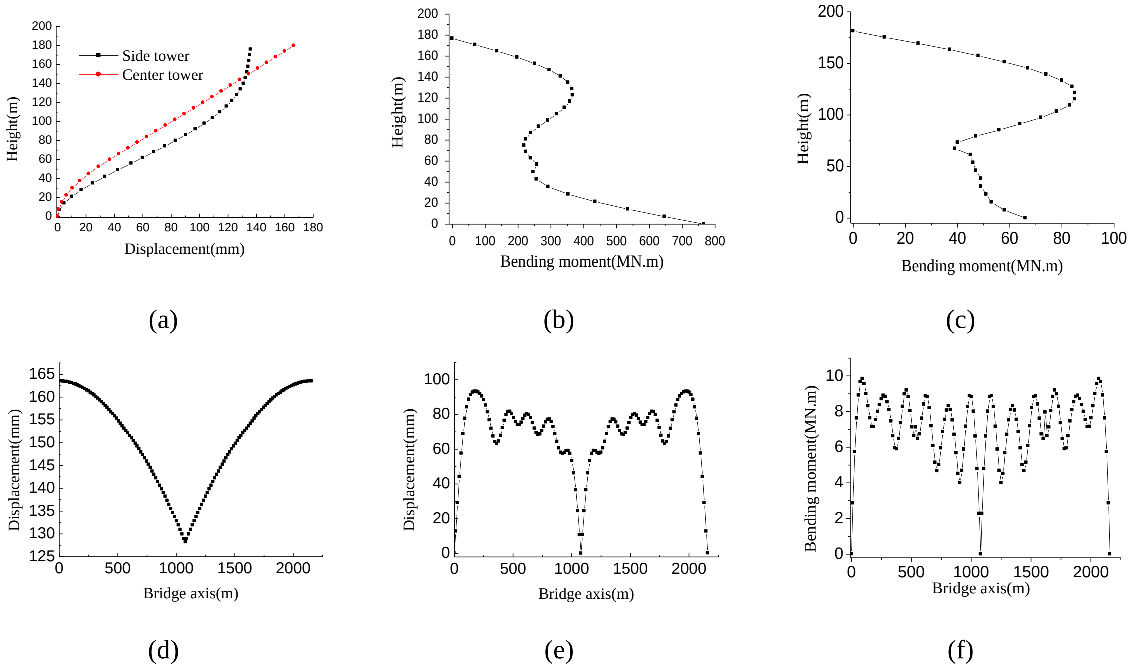
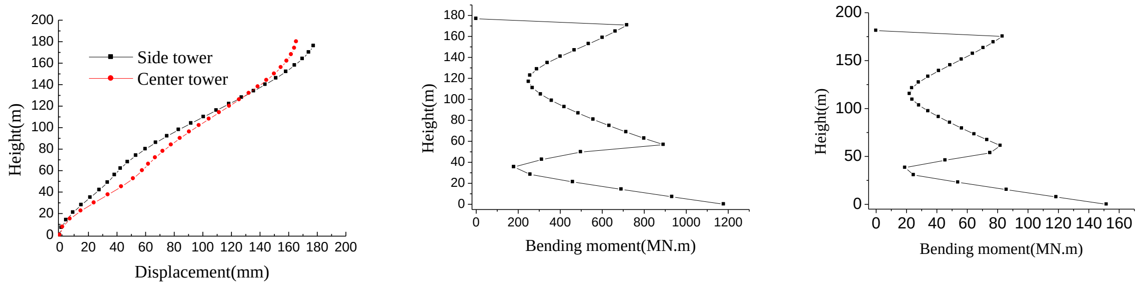
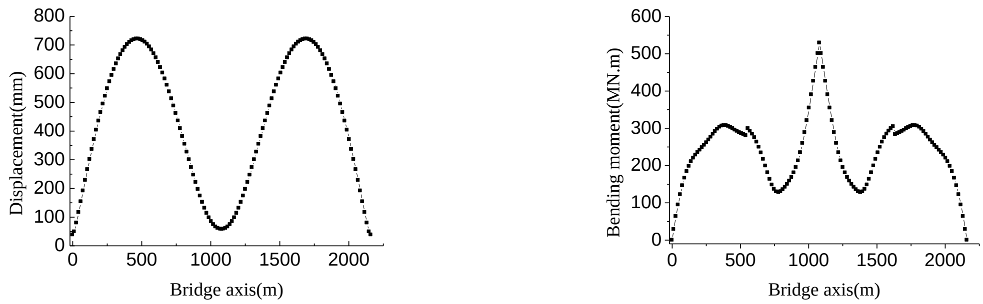


Fig. 4 Structural displacement and bending moment envelope diagrams under longitudinal seismic action: (a) longitudinal displacements of towers, (b) longitudinal bending moment of the side towers, (c) longitudinal bending moment of the center towers, (d) longitudinal displacement of the girder, (e) vertical displacement of the girder, (f) vertical bending moment of the girder

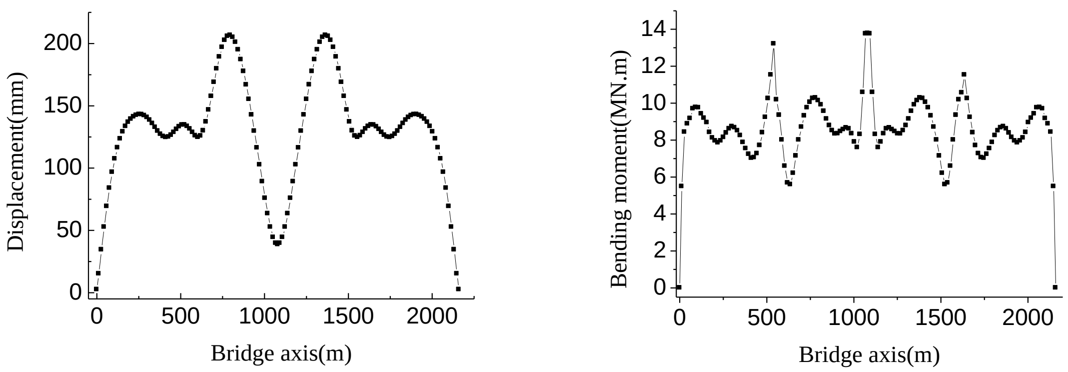


(a) (b) (c)



(d) (e)

Fig. 5 Structural displacement and bending moment envelope diagrams under lateral seismic action: (a) lateral displacements of towers, (b) lateral bending moment of the side towers, (c) l lateral bending moment of the center towers, (d) lateral displacement of the girder, (e) lateral bending moment of the girder



(a) (b)

Fig. 6 Structural displacement and bending moment envelope diagram under vertical seismic action: (a) vertical displacement of the girder, (b) vertical bending moment of the girder

Table 3 The maximum internal forces of the bridge under longitudinal seismic action

Internal forces	Bending moment (MN.m)	Shear force (MN)	Axial force (MN)
Center tower	85.3	3.11	43.2
Side tower	766	19.1	7.0
Girder	9.84	0.24	19.1

Note: MN denotes 1×10^6 N, MN.m denotes 1×10^6 N.m.

Table 4 The maximum internal forces of the bridge under lateral seismic action

Internal forces	Bending moment (MN.m)	Shear force (MN)	Axial force (MN)
Center tower	152	4.25	5.74
Side tower	1180	35.4	93.7
Girder	529	2.76	0

Table 5 The maximum internal forces of the bridge under vertical seismic action

Internal forces	Bending moment (MN.m)	Shear force (MN)	Axial force (MN)
Center tower	2.81	0.22	10.8
Side tower	84.7	2.18	106
Girder	13.8	0.457	5.45

Under the longitudinal seismic excitation, the side and center towers are undergoing the longitudinal vibration, and the girder is undergoing the longitudinal and vertical vibration. The maximum longitudinal displacements of both the side and center towers occur at their top ends, the longitudinal displacements of the upper part of the side tower are very identical, while for the center tower, it increases monotonously, which demonstrates that the longitudinal constraint of main cables for the side towers is much greater than that for the center tower. The maximum bending moment occurs at its bottom for the side tower, and for the center tower, it occurs at the points about two third of the tower height. Although the longitudinal displacement of the side tower is less than that of the center tower, due to its large structural stiffness, the longitudinal bending moment and shear force of the side tower are much greater than those of the center tower.

Under the lateral seismic excitation, the side and center towers and the girder are all undergoing the lateral vibration. The maximum longitudinal displacements of both the side and center towers occur at their top ends, which indicate that the longitudinal constraint of main cables has basically no effect on the lateral motion of the side and center towers. The maximum bending moment occurs at their bottom ends for the side and center towers. As for the girder, the maximum lateral displacement occurs at the midpoints of two main spans, whereas the maximum bending moment occurs near the center tower. Although little difference in the lateral displacement exists between the side and center towers, structural internal forces of the side tower are much greater than those of the center tower.

Under the vertical seismic excitation, large vertical displacement and also bending moment occur in the girder. As for the side and center towers, they bend longitudinally to a certain extent, due to its great longitudinal flexible stiffness, and large internal forces are encountered in the side tower.

Based on the above structural responses under the longitudinal, lateral and vertical seismic

Table 6 The combination structural displacements (mm)

Displacement	U_x	U_y	U_z
Center tower	166.3	165.4	/
Side tower	137.4	177.5	/
Girder	163.8	721.2	220.5

Note: *x*, *y* and *z* denote the longitudinal, lateral and vertical directions of the bridge; U_x, U_y and U_z denote the translations along *x*-axis, *y*-axis and *z*-axis separately

Table 7 The combination structural internal forces

Internal forces	N (MN)	Q_x (MN)	Q_y (MN)	Q_z (MN)	M_x (MN.m)	M_y (MN.m)	M_z (MN.m)
Center tower	44.1	3.11	4.60	/	156	85.3	/
Side tower	142	19.2	35.5	/	1180	771	/
Girder	19.8	/	2.76	0.515	/	15.0	529

Note: N denotes the axial force; Q_x, Q_y and Q_z denote the shear forces along *x*-axis, *y*-axis and *z*-axis separately; M_x, M_y and M_z denote the bending moments around *x*-axis, *y*-axis and *z*-axis separately.

excitations separately, the combination seismic response is computed and given in Tables 6 and 7.

$$E = \sqrt{E_X^2 + E_Y^2 + E_Z^2}$$

Where E is the combination seismic response; E_x, E_y and E_z are the seismic response of the bridge under the longitudinal, lateral and vertical seismic excitation separately.

Through comparison of the results in Tables 3-7, it is found that structural response under the lateral seismic action is much greater than that under the longitudinal and vertical actions, especially for the side towers, and therefore more attentions should be paid to the lateral seismic performance and the side towers.

5.3 Nonlinear time history analysis

To investigate the effect of structural ninlinearity on the seismic response of three-tower suspension bridge, the nonlinear time history analysis of seismic response for the bridge under the horizontal earthquake ground motions is conducted, and the peak displacements and internal forces of the center tower, the side tower and the girder are given and compared to those obtained by response spectrum analysis in Table 8 and Table 9 respectively. In the seimic time history analysis, structural geometric nonlinearity of the bridge is considered, and the acceleration time history curves of horizontal earthquake ground motions are simulated according to the design horizontal acceleration response spectrum as plotted in Fig. 3.

As seen in Table 8, the peak displacements obtained from the time history analysis are all greater than those of response spectrum analysis. It can be attributed to the fact that structural stiffness decreases with the geometric nonlinearity of the bridge under the horizontal earthquake ground action. Similarly as found in Table 9, except the center tower, the peak internal forces of the side tower and the girder are also greater than those of response spectrum analysis. As for the center tower, the peak bending moment and shear force decrease slightly, and but the peak axial force increases as compared to those of response spectrum analysis. In general, the numerical

Table 8 Comparison of the maximum displacements (mm) between the response spectrum and time history analysis

Seismic action direction	Center tower		Side tower		Girder	
	RS	TH	RS	TH	RS	TH
Longitudinal	166.3	189.2	135.9	168.4	165.6	189.1
Lateral	165.4	167.2	177.5	185.7	721.1	762.4

Note: RS-response spectrum analysis; TH-time history analysis.

Table 9 Comparison of the maximum internal forces between the response spectrum and time history analysis

Seismic action direction	Internal forces	Center tower		Side tower		Girder	
		RS	TH	RS	TH	RS	TH
Longitudinal	M (MN.m)	85.3	65.5	766	1110	9.84	9.88
	Q (MN)	3.11	2.41	19.1	23.7	0.24	0.29
	N (MN)	43.2	50.5	7.0	8.23	19.1	22.5
Lateral	M (MN.m)	152	148	1180	1330	529	631
	Q (MN)	4.25	4.19	35.4	39.0	2.76	2.84
	N (MN)	5.74	6.25	93.7	100.0	0	0

Note: RS-response spectrum analysis; TH-time history analysis; M-bending moment; Q-shear force; N-axial force.

results of the response spectrum and time history analysis are consistent, and however the nonlinear time history analysis is more favorable to accurately predicate structural seismic performance of the bridge with great flexibility.

6. Effects of structural parameters on seismic response of three-tower suspension bridges

To provide guidance for seismic design of long-span three-tower suspension bridges, effects of cable's sag to span ratio, structural stiffness of the center tower and longitudinal constraint of the girder on structural seismic performance are investigated numerically, and the favorable structural system of three-tower suspension bridges is also discussed.

6.1 Cable's sag to span ratio

The cable's sag to span ratio is an important design parameter for long-span suspension bridges, which directly affects the tension forces in the cables and also the gravity stiffness of the bridge. Generally, the cable's sag to span ratio ranges from 1/9 to 1/12; and for long-span suspension bridges, it is usually assumed as approximately 1/10. To understand how the cable sag affects the seismic performance of the bridge, seismic response analyses for the bridge with different cable sags are performed. Based on the example bridge, remaining the girder and the cables' height at midpoints of two main spans unchanged, two case bridges with the cable's sag to span ratios of 1/10 and 1/11 respectively are designed, structural seismic response is then

Table 10 Effect of cable's sag to span ratio on structural displacement under seismic action (mm)

Cable's sag to span ratio		1/9	1/10	1/11
Center tower	U_x	166.3	174.8	217.9
	U_y	165.4	220.9	277.2
Side tower	U_x	137.4	146.3	187.9
	U_y	177.5	182.7	210.2
Girder	U_x	163.8	172.1	211.4
	U_y	721.2	747.8	780.4
	U_z	220.5	237.1	178.3

Table 11 Effect of cable's sag to span ratio on structural internal forces under seismic action

Cable's sag to span ratio		1/9	1/10	1/11
Center tower	N (MN)	44.1	45.3	49.8
	Q_y (MN)	4.60	3.82	3.83
	Q_x (MN)	3.11	2.58	2.29
	M_y (MN.m)	85.3	10.2	13.6
	M_x (MN.m)	156	173	199
Side tower	N (MN)	142	148	98.2
	Q_y (MN)	35.5	28.0	26.4
	Q_x (MN)	19.2	18.4	22.3
	M_y (MN.m)	771	800	990
	M_x (MN.m)	1180	1200	1300
Girder	N (MN)	19.8	16.5	14.6
	Q_y (MN)	2.76	2.56	2.38
	Q_z (MN)	0.515	0.574	0.576
	M_y (MN.m)	15.0	17.5	16.8
	M_z (MN.m)	529	542	535

analyzed, and the results are presented in Tables 10 and 11.

As found in Tables 10 and 11, with decreasing of the cable's sag, structural displacements of the center tower, side tower and also the girder all increase significantly, and the prominent lateral bending moment of the center and side towers both increase greatly. But for the girder, structural internal forces are slightly influenced by the cable's sag. In conclusion, viewed from the aspect of seismic performance, large cable's sag is favorable for long-span three-tower suspension bridges.

6.2 Structural stiffness of the center tower

Structural stiffness of the center tower, which is an important problem for structural design of three-tower suspension bridges, affects significantly the vertical deflection of girder, the slip resistance between main cable and saddle on the center tower, the stress condition and longitudinal displacement of the center tower etc. For the example bridge, the center tower is a steel tower, whereas the two side towers are concrete towers, and therefore with comparison to the two side concrete towers, the steel center tower is considered to be a flexible tower. To understand how structural stiffness of the center tower affects structural seismic performance of three-tower

Table 12 Effect of structural stiffness of the center tower on structural displacement under seismic action (mm)

Structural stiffness of the center tower		Flexible	Rigid
Center tower	U_x	166.3	191.2
	U_y	165.4	171.18
Side tower	U_x	137.4	153.9
	U_y	177.5	177.1
Girder	U_x	163.8	159.8
	U_y	721.2	701.8
	U_z	220.5	380.2

Table 13 Effect of structural stiffness of the center tower on structural internal forces under seismic action

Structural stiffness of the center tower		Flexible	Rigid
Center tower	N (MN)	44.1	268
	Q_y (MN)	4.60	57.6
	Q_x (MN)	3.11	40.9
	M_y (MN.m)	85.3	809
	M_x (MN.m)	156	1950
Side tower	N (MN)	142	206
	Q_y (MN)	35.5	38.0
	Q_x (MN)	19.2	21.2
	M_y (MN.m)	771	862
	M_x (MN.m)	1180	1220
Girder	N (MN)	19.8	33.4
	Q_y (MN)	2.76	5.00
	Q_z (MN)	0.515	0.663
	M_y (MN.m)	15.0	27.3
	M_z (MN.m)	529	542

suspension bridges, a case bridge with rigid center tower is designed, the case bridge has the same design parameters as the example bridge, instead of steel center tower, the concrete center tower is adopted, and its material and cross-sectional properties are the same as the side towers. The seismic response of the case bridge is analyzed, and compared to the example bridge as shown in Tables 12 and 13.

In the case of rigid concrete tower, as compared to the example bridge with flexible steel center tower, longitudinal displacements of both the center and side towers increase greatly, especially for the center tower, the increase is greater than 15%, and however the lateral displacement changes little. Longitudinal and lateral displacements of the girder decrease, and but its vertical displacement increases remarkably. Structural internal forces of the side towers and girder both increase, however a remarkable increase in structural internal forces happens for the center tower, structural internal forces of rigid concrete center tower are more than 10 times those of the example bridge with flexible steel center tower, which is disadvantage for the center tower. Therefore considering the seismic performance, the center tower with less structural stiffness is favorable for three-tower suspension bridges.

Table 14 Effect of longitudinal constraint of the girder on structural displacement under seismic action (mm)

Longitudinal constraint of the girder		F+C+E	F+C	F+E	F
Center tower	U_x	166.3	244.2	168.3	266.0
	U_y	165.4	165.4	164.7	164.7
Side tower	U_x	137.4	129.1	136.2	136.7
	U_y	177.5	177.5	178.1	178.1
Girder	U_x	163.8	208.1	170.9	290.1
	U_y	721.2	721.1	727.6	727.6
	U_z	220.5	443.5	211.6	535.0

Note: *F* denotes floating system; *E* denotes the longitudinal elastic cable; *C* denotes the central buckle

Table 15 Effect of longitudinal constraint of the girder on structural internal forces under seismic action

Longitudinal constraint of the girder		F+C+E	F+C	F+E	F
Center tower	N (MN)	44.1	19.7	44.9	22.4
	Q_y (MN)	4.60	4.25	4.67	4.29
	Q_x (MN)	3.11	4.90	4.10	5.49
	M_y (MN.m)	85.3	209	118	228
	M_x (MN.m)	156	152	157	153
Side tower	N (MN)	142	142	143	143
	Q_y (MN)	35.5	35.5	35.6	35.6
	Q_x (MN)	19.2	19.1	18.7	19.4.
	M_y (MN.m)	771	719	758	809
	M_x (MN.m)	1180	1180	1180	1180
Girder	N (MN)	19.8	10.3	20.2	1.39
	Q_y (MN)	2.76	2.76	2.78	2.78
	Q_z (MN)	0.515	0.645	0.499	0.487
	M_y (MN.m)	15.0	28.1	13.3	13.9
	M_z (MN.m)	529	529	525	525

6.3 Longitudinal constraint of the girder

To restrain the longitudinal displacement of the girder, the longitudinal elastic cables are installed between the center tower and the girder. To investigate the effect of longitudinal constraint of the girder on the seismic performance of three-tower suspension bridges, three case bridges with different longitudinal constraints of the girder are designed. For the case bridge 1, the girder floats longitudinally and without any longitudinal constrain; for the case bridge 2, the girder floats longitudinally, and the central buckles between main cables and the girder are installed at midpoints of two main spans; for the case bridge 3, the girder floats longitudinally, the longitudinal elastic cables between the center tower and the girder and the central buckles between main cables and the girder are both installed. Seismic response of these case bridges is then analyzed, and the results are given in Tables 14 and 15.

As found above, the longitudinal elastic cable and central buckle have little influence on the lateral seismic response, and however have significant influence on the longitudinal seismic response.

With comparison to the case bridge with the girder floating longitudinally, under the case that the longitudinal elastic cables are installed, longitudinal displacements of the center tower and the girder, and the vertical displacement of the girder all decrease greatly, and however longitudinal displacement of the side towers basically remains unchanged. As a result, longitudinal bending moment and shear force of the center tower decrease remarkably, the axial force in the girder increases significantly, and but the vertical bending moment and shear force change little. In the case of central buckles installed at midpoints of two main spans, longitudinal displacements of the center tower and the girder, and the vertical displacement of the girder also decrease greatly, and however the decrease is much less than that in the case of longitudinal elastic cables installed. Longitudinal bending moment and shear force of the center and side towers decrease slightly, however the axial force, the vertical bending moment and shear force in the girder increase significantly. In the case of both the longitudinal elastic cables between the center tower and the girder and the central buckles between main cables and the girder installed, structural seismic response is basically identical to that in the case of only the longitudinal elastic cables installed, and therefore the fact demonstrates that the longitudinal elastic cables play the leading role in the seismic response, and the influence of central buckle on the seismic response is very limited. In conclusion, the longitudinal elastic cable between the center tower and the girder is favorable to reduce the seismic response of three-tower suspension bridges.

7. Conclusions

By taking a three-tower suspension bridge-Taizhou Highway Bridge over the Yangtze River with two main spans of 1080 m as example, structural dynamic characteristics and seismic performance of the bridge is investigated by MIDAS/Civil software, and the effects of cable's sag to span ratio, structural stiffness of the center tower, and longitudinal constraint of the girder on structural seismic response of the bridge are investigated, and the favorable structural system of three-tower suspension bridges is discussed with respect to seismic performance. The results show that structural response under lateral seismic action is more remarkable, especially for the side towers, and therefore more attentions should be paid to the lateral seismic performance and also the side towers. Large cable's sag, flexible center tower and the longitudinal elastic cable installed between the center tower and the girder are favorable to improve structural seismic performance of long-span three-tower suspension bridges.

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