Earthquakes and Structures, *Vol. 14, No. 6 (2018) 493-503*
DOI: https://doi.org/10.12989/eas.2018.14.6.493

Investigation of seismic performance of super long-span cable-stayed bridges

Xin-Jun Zhang*, Chen-Yang Zhao and Jian Guo

College of Civil Engineering & Architecture, Zhejiang University of Technology, Hangzhou 310023, P.R. China

(Received August 17, 2017, Revised March 31, 2018, Accepted April 2, 2018)

Abstract. With the further increase of span length, the cable-stayed bridge tends to be more slender, and becomes more susceptible to the seismic action. By taking a super long-span cable-stayed bridge with main span of 1400m as example, structural response of the bridge under the E1 horizontal and vertical seismic excitations is investigated numerically by the multimode seismic response spectrum and time-history analysis respectively, the seismic behavior and also the effect of structural nonlinearity on the seismic response of super long-span cable-stayed bridge are revealed. Furthermore, the effect of structural parameters including the girder depth and width, the tower structural style, the tower height-to-span ratio, the side-to-main span ratio, the auxiliary piers in side spans and the anchorage system of stay cables etc on the seismic performance of super long-span cable-stayed bridge is investigated numerically by the multimode seismic response spectrum analysis, and the favorable earthquake-resistant structural system of super long-span cable-stayed bridge is proposed.

Keywords: super long-span cable-stayed bridge; seismic performance; multimode seismic response spectrum analysis; time-history analysis; structural parameters

1. Introduction

Due to its esthetic appearance, efficient utilization of structural materials and other notable advantages, the cable-stayed bridge has been recognized as the most appealing structure, and has gained much popularity in recent decades. The completion of the Sutong Bridge (1088 m, China) and the Stonecutters Bridge (1018 m, Hong Kong) indicate that the main span of cable-stayed bridge has reached kilometer scale in practice, a length that previously can be realized only by suspension bridges. In 2012, the Russky Island Bridge in Russia with main span of 1104 m is completed. Even longer span of cable-stayed bridges are being investigated, tentatively designed, or under construction in the world and particularly in Asia, for example a 1200 m cable-stayed bridge is proposed to link the Masan and the Geoje Island in South Korea, and in the Honshu-Shikoku contact line in Japan, a 1400 m cable-stayed bridge is schemed. Because of its superiority in structural stiffness, wind resistance, cable replacement, construction and no anchorages, the cable-stayed bridge is employed frequently in the sea-crossing bridge design competition. Today, long-span cable-stayed bridge is of great interest, mainly as an alternative and a more economical solution than suspension bridge. Studies on the limit span length of cable-stayed bridge also demonstrate that cable-stayed bridge is suitable for the main span below 1200 m, and in the span region from 1200 m to 1500 m, it is still competitive (Xiang 2012, Gimsing and Georgakis 2012).

However, due to its great flexibility and low structural damping, modern long-span cable-stayed bridge is usually very susceptible to dynamic loads such as wind and earthquake. Therefore, the wind-resistant and earthquake-resistant designs are key issues for its successful construction. Up to now, several investigations on the seismic response and its control of long-span cable-stayed bridge have been conducted. Ye and Fan (2007) investigated the seismic response of the Sutong Bridge with three different lateral connection systems between the side piers and girder by the nonlinear time-history analysis, and the damper arrangement and optimal parameters are also proposed. Wang and Huang (2008) applied the elastic restraint and viscous dampers to the deck-tower connection of the Sutong Bridge, and analyzed the proper elastic stiffness constant of elastic restraint and viscous damper parameters. Sharabash and Andrawes (2009) introduced a new passive seismic control device made with shape memory alloys for cable-stayed bridges. Soyluk and Sicacik (2012) investigated the effects of soil-structure interaction and spatially varying ground motion on the dynamic analysis of cable-stayed bridges. Camara and Astiz (2012) developed two new procedures to predict the seismic response of cable-stayed bridge subjected to multi-axial strong ground motions: the Extended Modal Pushover Analysis (EMPA) and the Coupled Nonlinear Static Pushover analysis (CNSP). Ismail *et al*. (2013) investigated the near-fault seismic performance of a cable-stayed bridge in USA with a recent isolation system referred to as Roll-N-Cage isolator. Wu *et al*. (2014) conducted the stochastic seismic response analysis of the Sutong Bridge, and investigated the influence of structural elastic modulus (the girder and tower), the pile-soil-structure interaction, the seismic ground motion model, the cable vibration and the number of modes on seismic internal forces and displacements. Han and Ye (2015) explored several methods to reduce the longitudinal displacements at bridge

∗Corresponding author, Professor
E-mail: xjzhang@zjut.edu.cn

ISSN: 2092-7614 (Print), 2092-7622 (Online)

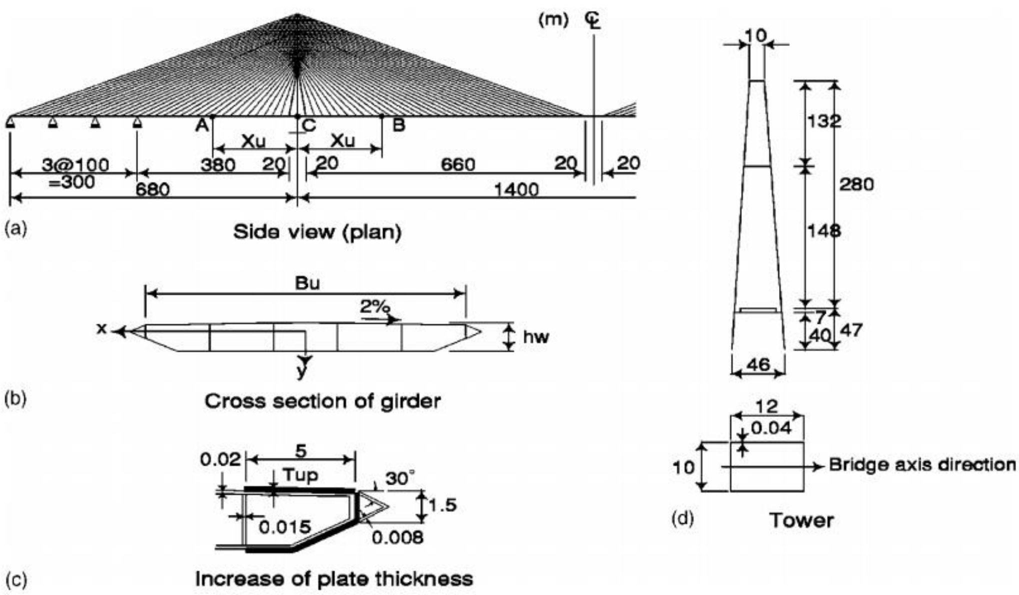

Fig. 1 General layout of a cable-stayed bridge with main span of 1400 m

ends and the internal forces in the tower under earthquake action for a floating system cable-stayed bridges with kilometer-scale main span, and proposed the rational longitudinal earthquake-reduction system through analyzing and comparing the elastic connecting device, the liquid viscous damper, and the combination of former two between the tower and girder. To mitigate wind or seismic load-induced vibrations, Zhu *et al*. (2015) investigated the effects of fluid viscous damper for a cable-stayed bridge under randomly generated earthquake excitation, and proposed its optimized design parameters. Naderian *et al*. (2016) presented the integrated finite strip method along with the application of a very robust and efficient time history method using the Newmark scheme for dynamic analysis of long-span cable-stayed bridges. As seen from above, pervious researches mainly focus on the seismic response analysis, the seismic performance and countermeasures of cable-stayed bridges with main span slightly over or below 1000 meters, and for even longer cable-stayed bridge, its seismic performance and reasonable earthquake-resistant structural system have not yet been investigated comprehensively.

In this work, by taking a super long-span cable-stayed bridge with main span of 1400 m as example, structural response of the bridge under the E1 horizontal and vertical seismic excitations is investigated by the multimode response spectrum and time-history analysis respectively, the seismic behavior and the effect of structural nonlinearity on the seismic response of the bridge are revealed. Furthermore, the influence of structural parameters including the depth and width of the girder, the tower structural style, the tower height-to-span ratio, the side-to-main span ratio, the auxiliary piers in side spans and the anchorage system of stay cables etc on the seismic performance of the bridge is investigated by the multimode seismic response spectrum analysis, and the favorable earthquake-resistant structural system of super long-span cable-stayed bridge is also proposed.

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

Components	E (Mpa)	A (m^2)	J_d (m^4)	I_y (m^4)	I_z (m^4)	W (kN/m)
Girder	2.1×10^5 (2.1×10^5)	1.761 (2.046)	8.330 (9.739)	193.2 (261.1)	3.939 (4.432)	258.4 (280.4)
Stay cables	2.0×10^5	0.0087 0.038	0.0	0.0	0.0	2.926- 0.693
Tower	2.1×10^5	1.76	30.67	40.32	39.27	189.6

Note: E-elastic modulus; A-cross-sectional area; J_d-torsional moment of inertia; I_y-lateral bending moment of inertia; I_z-vertical bending moment of inertia; W-weight per unit length; values in parentheses are reinforced values.

2. Modeling of the example bridge

2.1 Description of the example bridge

Fig. 1(a) shows the side view of a 1400 m cable-stayed bridge model (Nagai *et al*. 2004). Center and side spans are assumed to be 1400 m and 680 m respectively, three auxiliary piers are installed in each side span at a distance of 100 m in order to increase in-plane flexural rigidity of the bridge. The two inclined stay cable planes are both fan-shaped, and there are 136 stay cables in each stay cable plane. The deck shown in Fig. 1(b) is a streamlined steel box girder of 35 m wide and 3.5 m high, and suspended by diagonal stay cables anchored to the deck at 20 m intervals. To cope with the large bending moment from wind load in the girder near the tower, as shown in Fig. 1(c), the plate thickness at the edge of deck cross section is increased. The required distance for reinforcement from the tower is defined as Xu seen in Fig. 1(a), which is 80 m herein. Fig. 1(d) shows a front view and the assumed cross section of the tower. The tower height above from deck level is 280 m, which is one-fifth of the center span length. Table 1 gives the cross-sectional and material properties of the example bridge.

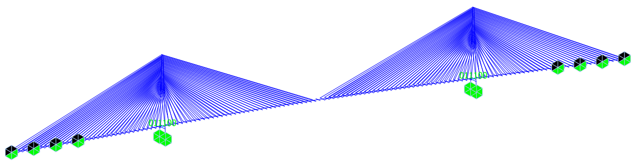

Fig. 2 3D dynamic finite element model of the example bridge

2.2 Finite element modelling

The example bridge is treated as a three-dimensional dynamic finite element model as plotted in Fig. 2, in which the deck is modeled by the single-girder model; the deck and towers are modeled by 3D beam elements, and the axial force-bending moment interaction in the girder and towers is considered; the stay cables are modeled by 3D bar elements, and the tension stiffness of stay cables, which varies with the sag, is modeled by using an equivalent straight truss element with an equivalent modulus of elasticity; the rigid diaphragms are provided to model the connections between the deck and stay cables. Both the outer and inner boundary conditions are applied to the finite element model. The tower bottoms are fixed at the bases. As for the girder, it moves freely along the longitudinal direction; at the tower, there is no vertical support for the girder, however the lateral support is provided by the vertical sliding bearings between the girder and tower columns; at the anchorage piers and intermediate auxiliary piers, the vertical (*Z)* and lateral (*Y*) movements and also the rotation about *X*-axis are all subordinate to the anchorage and auxiliary piers, and the other movements are left free.

3. Dynamic characteristic of the example bridge

On the computed equilibrium state of the example bridge in completion, the dynamic characteristics of the example bridge is analyzed by Midas/civil. Table 2 shows the girder's modal properties of the example bridge from the first symmetric lateral bending mode to the first asymmetric torsion mode of the girder.

As found in Table 2, the results obtained from Midas/civil and ANSYS are almost the same, and thus the finite element model of the bridge is verified to be accurate. Some features on the dynamic behavior of super long-span cable-stayed bridge can be concluded as follows: (1) the fundamental period is 17.57 s, it is very long, which demonstrates that the bridge is a structural system with great flexibility; (2) the lateral bending mode comes firstly, and then the vertical bending mode comes, and the torsional mode comes finally, the frequency ratio of the fundamental in-plane and out-of-plane modes is 3.183:1, which indicates the out-of-plane structural stiffness is much less than that in plane, and therefore the bridge becomes more susceptible to the lateral dynamic action such as wind and earthquake; (3) the vibration frequency distributes densely within a narrow band, the coupling among modes is remarkable, and therefore the CQC method should be used for the modal combination in the seismic response spectrum analysis of long and particularly super long-span cable-stayed bridge.

Table 2 The girder's modal properties of the example bridge

Modes	Frequency(Hz)	Mode shape*
Vertical bending	0.1811(0.1805)	1-S
	0.2102(0.2114)	1-AS
	0.2620(0.2608)	2-S
	0.3001(0.3015)	2-AS
	0.3739(0.3732)	3-S
	0.4254(0.4236)	3-AS
Lateral bending	0.0569(0.0572)	1-S
	0.1656(0.1647)	1-AS
	0.3125(0.3133)	2-S
	0.5058(0.5042)	2-AS
Torsion	0.4133(0.4106)	1-S
	0.5389(0.5356)	1-AS

Note: S-symmetric; AS-anti-symmetric; the value denotes the mode sequence number; the data in bracket is obtained from ANSYS.

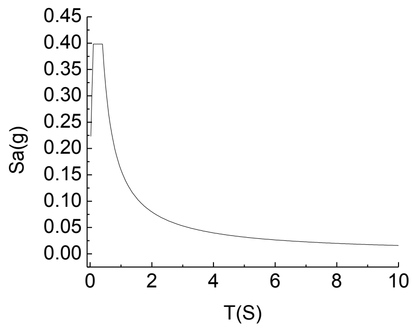

Fig. 3 The horizontal seismic design acceleration response spectrum under earthquake action E1

4. Seismic response analysis of the example bridge

4.1 Earthquake ground motion

4.1.1 Seismic acceleration response spectrum

According to the guidelines for seismic design of highway bridges (JTG/T B02-01 2008), the bridge type and the geological condition of bridge site, the seismic fortification intensity is 7, and the basic design acceleration of ground motion is 0.15 g; the bridge site is classified as Class II, its characteristic period is 0.40s, and structural damping ratio is taken as 3%. Under the 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.

4.1.2 The artificial seismic acceleration time-history curves

By taking the above design acceleration response spectrum as target spectrum, three artificial seismic acceleration time-history curves as shown in Fig. 4 are

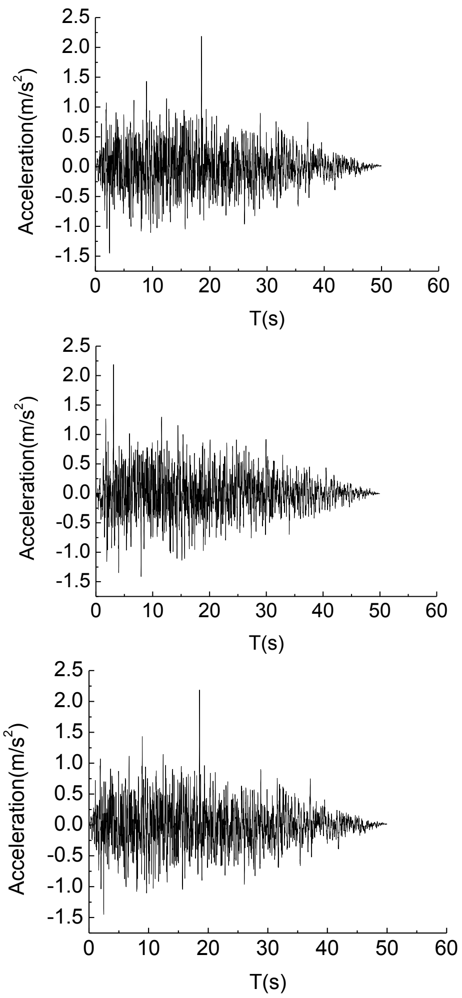

Fig. 4 Three artificial seismic acceleration time-history curves under earthquake action E1

simulated by the trigonometric series superposition method, which are the seismic excitations of nonlinear seismic response time-history analysis as follows.

4.2 Seismic response spectrum analysis

Under the longitudinal, lateral and vertical seismic excitations, structural response of the example bridge is investigated numerically by the multimode response spectrum analysis (Zhang and Yu 2015). Due to the dense distribution of natural frequency and remarkable modal coupling effect of the bridge, the CQC method is thus used for modal combination. Due to the paper length limitation, only the maximum seismic responses are listed in Table 3.

As observed from the analysis results, the seismic behavior of super long-span cable-stayed bridge can be concluded as follows:

(1) Under the longitudinal seismic excitation, the towers are undergoing the longitudinal vibration, and the girder is undergoing the coupled longitudinal-vertical vibration. As for the towers, the maximum longitudinal displacement occurs at the tower upper end, and there exists the maximum longitudinal bending moment, shear and axial force at the tower bottom section. For the girder, the longitudinal displacement is basically the same along the bridge axis, and at the two quarter points in the center span and also the midpoint between the tower and the first auxiliary pier near the tower in the side span, there exists the maximum vertical displacement. The maximum vertical bending moment and shear both occur at the first auxiliary pier near the tower in the side span. In general, the seismic response of the tower is much greater than that of the girder, therefore the longitudinal seismic excitation is unfavorable for the tower, and more attention should be paid to the seismic design of the tower bottom section.

(2) Under the lateral seismic excitation, the tower and girder are both undergoing the lateral vibration. The tower bends laterally, the maximum lateral displacement occurs at the tower upper end, and the maximum lateral bending moment, shear and axial force occur at the tower bottom section. The girder's maximum lateral displacement happens at midspan, and but its maximum lateral bending moment and shear force occur at the tower. As compared to the tower, greater seismic response is achieved for the girder, and the girder section at the tower becomes the key sections, which should be paid more attention to its seismic design.

(3) Under the vertical seismic excitation, the tower bends longitudinally, and the girder moves longitudinally and deflects vertically. As for the girder, the maximum vertical displacement and bending moment both occur at the midpoint of center span. The inertial force of the girder under the vertical seismic excitation is transferred to the tower by the stay cables, the axial force in the tower is thus increased remarkably, and accompanying with the longitudinal bending, large longitudinal bending moment and shear force happen at the tower bottom section.

(4) Through the comparison of results given in Table 3, it is found that structural responses under the longitudinal and lateral seismic excitations are both much greater than those under the vertical seismic excitation, and therefore the seismic performance of cable-stayed bridge under the horizontal earthquake action should be emphasized. The seismic excitation produces great structural response in the tower bottom section and the girder section at the tower, and therefore more attentions should be paid to the seismic design of these sections.

4.3 Geometric nonlinear seismic response time-history analysis

To investigate the effect of geometric nonlinearity on the seismic performance of super long-span cable-stayed bridge, the seismic response of the example bridge under the horizontal and vertical earthquake ground motions is investigated by the geometric nonlinear time-history analysis (Zhang and Yu 2015), and the peak displacement and internal force of the tower and girder are given in Table 4. For the direct comparison of different cases, structural internal forces and displacements given in the following tables are the combination values, which are square root of the sum of squares of the maximum seismic responses under the longitudinal, lateral and vertical seismic excitations separately. In the analysis, only the geometric nonlinearity is considered, and three acceleration time-

Table 3 The maximum seismic response of the example bridge under earthquake action E1

Seismic excitations	Members	Bending moment(kN.m)	Shear force(kN)	Axial force(kN)	Displacement(mm)		
					Longitudinal	Lateral	Vertical
Longitudinal	Tower	2.62×10^6	2.89×10^4	4.40×10^4	908	-	-
	Girder	7.32×10^5	2.80×10^4	3.88×10^4	893	-	617
Lateral	Tower	3.00×10^5	2.42×10^4	5.86×10^4	-	152	-
	Girder	2.93×10^6	2.43×10^4	-	-	870	-
Vertical	Tower	5.82×10^4	1.32×10^2	2.06×10^5	34	-	-
	Girder	1.73×10^5	6.57×10^3	1.20×10^4	21	-	205

Note: under the longitudinal and vertical seismic excitations, the tower's bending moment and shear force are both in longitudinal direction, and as for the girder, they are in vertical direction; under the lateral seismic excitation, the bending moment and shear force are both in lateral direction; the same as follows.

Table 4 The peak values of time-history seismic response under earthquake action E1

Member		Bending moment(kN.m)		Shear force(kN)		Axial force(kN)	Displacement(mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	THA	2.83×10^6	3.11×10^5	2.62×10^4	2.98×10^4	2.42×10^5	1006	160	-
	RSA	2.62×10^6	3.00×10^5	2.89×10^4	2.42×10^4	2.19×10^5	909	152	-
Girder	THA	8.74×10^5	3.09×10^6	3.21×10^4	2.57×10^4	4.31×10^4	977	890	639
	RSA	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650

Note: THA-time history analysis; RSA-response spectrum analysis.

history curves plotted in Fig. 4 are taken as the seismic excitation separately, and the peak values are taken from the corresponding three seismic response time-history curves.

It is found from the geometric nonlinear time-history analysis that the peak values and their positions of seismic response of the tower and girder are basically identical to those of response spectrum analysis. In general, the seismic responses of geometric nonlinear time-history analysis are greater than those of response spectrum analysis. It is because that as compared to the linear response spectrum analysis, with consideration of the geometric nonlinearity, structural stiffness is reduced, and thus greater structural response is obtained under the same seismic excitation. Therefore for super long-span cable-stayed bridge, the geometric nonlinear time-history analysis is proposed to accurately predict its seismic response.

5. Parametric study on the seismic performance of super long-span cable-stayed bridge

In order to fully understand the seismic performance of super long-span cable-stayed bridge, the effects of structural parameters including the girder depth and width, the side-to-main span ratio, the tower structural style, the tower height-to-span ratio, the auxiliary piers in side spans and the anchorage system of stay cables on the seismic response of super longs-span cable-stayed bridge are investigated, and its favorable earthquake-resistant structural system is also proposed. It is to be noted that the case bridge is designed as each design parameter is changed, and the internal forces in the girder, tower and the stay cables of the bridge in completion are determined by Midas/civil.

5.1 The girder depth

Table 5 The cross-sectional properties of the girder with different depth

Girder depth(m)	A (m^2)	J_d (m^4)	I_y (m^4)	I_z (m^4)	W (kN/m)
3.5	1.761	8.330	3.939	193.200	258.4
3	1.709	6.201	2.875	186.820	254.1
4	1.815	10.741	5.176	199.580	265.5

Table 6 Effect of the girder depth on natural frequency(Hz)

Modes	Girder depth			Mode shape
	3.5 m (1)	3 m (2)	4 m (3)	
Vertical bending	0.1811	0.1818(0.4%)	0.1806 (-0.3%)	1-S
	0.2102	0.2091(-0.5%)	0.2113 (0.5%)	1-AS
	0.2620	0.2612(-0.3%)	0.2630 (0.4%)	2-S
	0.3001	0.2962(-1.3%)	0.3045(1.4%)	2-AS
	0.3739	0.3668(-1.9%)	0.3766 (1.9%)	3-S
	0.4254	0.4152(-2.4%)	0.4333 (2.3%)	3-AS
Lateral bending	0.0569	0.0556(-0.6%)	0.0608 (0.6%)	1-S
	0.1656	0.1646(-0.6%)	0.1698 (0.5%)	1-AS
	0.3125	0.3106(-0.6%)	0.3143 (0.6%)	2-S
	0.5058	0.4972(-1.7%)	0.5098 (0.8%)	2-AS
Torsion	0.4133	0.4079(-1.3%)	0.4158(0.6%)	1-S
	0.5389	0.5200(-3.5%)	0.5513 (2.3%)	1-AS

Note: The values in bracket =(the corresponding column-column(1))/column(1)×100%.

The girder depth is an important structural parameter affecting the flexibility stiffness of the girder and the total structural stiffness of the bridge. Based on the example bridge, two case bridges with the girder depth of 3m and 4m as shown in Table 5 are designed, except the girder, the cross sections of the tower and stay cables are remained the same as the example bridge. Structural dynamic

Table 7 Effect of the girder depth on the seismic response

Member	Girder depth(m)	Bending moment(kN.m)		Shear force(kN)		Axial force(kN)	Displacement(mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	3	2.59×10^6	2.94×10^5	2.79×10^4	2.03×10^4	2.10×10^5	885	148	-
	3.5	2.62×10^6	3.00×10^5	2.89×10^4	2.42×10^4	2.19×10^5	909	152	-
	4	2.64×10^6	2.54×10^5	2.82×10^4	2.23×10^4	2.12×10^5	876	147	-
Girder	3	7.14×10^5	2.72×10^6	2.69×10^4	2.02×10^4	3.83×10^4	890	854	619
	3.5	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650
	4	7.33×10^5	2.86×10^6	2.85×10^4	2.39×10^4	3.98×10^4	882	847	616

Table 8 The cross-sectional properties of the girder with different width

Girder width (m)	A (m^2)	J_d (m^4)	I_y (m^4)	I_z (m^4)	W (kN/m)
35	1.761	8.33	3.939	193.2	258.4
	(2.046)	(9.739)	(4.432)	(261.1)	(280.4)
32	1.642	7.583	3.269	151.5	212.7
	(2.070)	(9.561)	(3.943)	(234.6)	(283.2)
28	1.433	6.542	2.849	102.6	223.1
	(2.147)	(9.365)	(3.985)	(204.6)	(278.1)

Note: The values in parentheses are reinforced values.

characteristics and seismic response are analyzed, and the effect of girder depth on them is presented in Tables 6 and 7.

Table 6 and 7 show that the girder depth has little influence on the dynamic characteristics and the seismic response of the tower and girder of cable-stayed bridge. In general, with increasing the girder depth, the seismic displacement is decreased slightly, however the seismic internal force is increased remarkably, which indicates that structural stiffness is improved by increasing the girder depth, but the effect is limited. Therefore, the girder depth is not a sensitive parameter for the seismic performance of super long-span cable-stayed bridge, and it can be determined by structural static performance.

5.2 The girder width

The girder width is mainly determined by the design traffic volume, but it has significant influence on the lateral stiffness and the wind-resistant performance of cable-stayed bridge. In order to investigate the effect of girder width on the seismic performance of super long-span cable-stayed bridge, two case bridges with the girder width of 28 m and 32 m respectively are designed based on the example bridge, the cross-sectional properties of the girder with

Table 9 Effect of the girder width on natural frequency(Hz)

Modes	Girder width			Mode shape
	35 m(1)	32 m(2)	28 m(3)	
Vertical bending	0.1811	0.1932 (6.7%)	0.1871 (3.3%)	1-S
	0.2102	0.2216 (5.4%)	0.2102 (0%)	1-AS
	0.2620	0.2688 (2.6%)	0.2480 (-5.3%)	2-S
	0.3001	0.3058 (1.9%)	0.2839 (-5.4%)	2-AS
	0.3739	0.4003 (3.4%)	0.3838 (-2.6%)	3-S
	0.4254	0.4551 (0.4%)	0.4349 (-3.0%)	3-AS
Lateral bending	0.0569	0.0573 (0.7%)	0.0503 (-12.6%)	1-S
	0.1656	0.1601(-3.3%)	0.1311(-20.8%)	1-AS
	0.3125	0.3078 (-7.8%)	0.2540 (-26.7%)	2-S
	0.5058	0.4380 (-13.4%)	0.3510 (-30.6%)	2-AS
Torsion	0.4133	0.4414 (6.8%)	0.4257 (3.0%)	1-S
	0.5389	0.5798 (7.6%)	0.5642(4.7%)	1-AS

Note: The values in bracket =(the corresponding column-column(1))/column(1)×100%.

Fig. 5 The lateral configuration of inverse *Y*-shaped tower

different width are given in Table 8. Due to the decrease of girder width, the girder gravity and further the internal forces in the girder, towers and stay cables are decreased simultaneously. For simplicity, the cross sections of towers

Table 10 Effect of the girder width on the seismic response

Member	Girder width (m)	Bending moment (kN.m)		Shear force (kN)		Axial force (kN)	Displacement (mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	28	2.61×10^6	3.06×10^5	2.87×10^4	2.11×10^4	2.29×10^5	897	163	-
	32	2.54×10^6	2.92×10^5	2.84×10^4	2.07×10^4	1.99×10^5	893	158	-
	35	2.62×10^6	3.00×10^5	2.89×10^4	2.42×10^4	2.19×10^5	909	152	-
Girder	28	6.84×10^5	3.10×10^6	2.34×10^4	2.26×10^4	4.05×10^4	883	856	621
	32	6.92×10^5	2.91×10^6	2.66×10^4	2.17×10^4	3.67×10^4	877	863	621
	35	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650

Table 11 Effect of tower structural style on natural frequency (Hz)

Modes	*A*-shaped tower (1)	Inverse-*Y* shaped tower (2)	Mode shape
Vertical bending	0.1811	0.1810 (0%)	1-S
	0.2102	0.2100(-0.1%)	1-AS
	0.2620	0.2614(-0.2%)	2-S
	0.3001	0.2998(-0.1%)	2-AS
	0.3739	0.3735(-0.1%)	3-S
	0.4254	0.4249(-0.1%)	3-AS
Lateral bending	0.0569	0.0567(-0.4%)	1-S
	0.1656	0.1653(-0.2%)	1-AS
	0.3125	0.3119(-0.2%)	2-S
	0.5058	0.5068(0.2%)	2-AS
Torsion	0.4133	0.4311(4.3%)	1-S
	0.5389	0.5739(6.5%)	1-AS

Note: The values in bracket =(column(2)-column(1))/column(1)×100%.

and stay cables are the same as the example bridge, however structural arrangement of the stay cables are changed with different girder width. Their natural frequency and seismic responses are analyzed, and the effect of girder width on them is presented in Tables 9 and 10 respectively.

As shown in Table 9, the change of girder width has remarkable influence on structural dynamic characteristics, especially for the lateral bending modes. However, similarly as the girder depth, the effect of girder width on the seismic response of the tower and girder of cable-stayed bridge is also little. On the whole, the deck width is not a sensitive parameter for structural seismic performance and should be mainly determined by the demand of design traffic volume.

5.3 The tower structural style

The tower structural style mainly refers to its lateral configuration, which has significant influence on the lateral and torsional stiffness of cable-stayed bridge. Besides the *A*-shaped tower, the inverse *Y*-shaped tower is also widely employed in long-span cable-stayed bridges. In order to investigated the effect of tower structural style on the seismic performance of super long-span cable-stayed bridge, based on the example bridge, a case bridge with the inverse *Y*-shaped towers is designed as shown in Fig. 5. Except structural lateral arrangements of the towers and stay cables, the cross sections of the girder, towers and stay cables are not changed. Tables 11 and 12 show the effect of tower structural style on natural frequency and the seismic response of the tower and girder respectively.

As compared to the *A*-shaped tower, the vertical and lateral bending frequencies and the seismic internal forces of the tower and girder in the case of inverse *Y*-shaped tower are decreased a little, however the longitudinal displacement of tower, the lateral and vertical displacements of girder are significant increased, which indicates that structural stiffness is decreased for the bridge with inverse *Y*-shaped towers. As viewed from the aspect of

Table 12 Effect of the tower structural style on the seismic response

Member	Tower structural style	Bending moment (kN.m)		Shear force (kN)		Axial force (kN)	Displacement (mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	*A*-shaped	2.62×10^6	3.003×10^5	2.89×10^4	2.42×10^4	2.186×10^5	909	152	-
	Inverse *Y*-shaped	2.29×10^6	3.15×10^5	2.21×10^4	1.82×10^4	1.66×10^5	944	162	-
Girder	*A*-shaped	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650
	Inverse *Y*-shaped	6.17×10^5	2.83×10^6	2.29×10^4	2.28×10^4	3.91×10^4	896	985	707

Table 13 Effect of tower height-to-span ratio on natural frequency (Hz)

Modes	Tower height-to-span ratio			Mode shape
	1/5 (1)	1/4 (2)	1/6 (3)	
Vertical bending	0.1811	0.2035 (12.4%)	0.1539(-15 %)	1-S
	0.2102	0.2306 (9.7%)	0.1824(-13.2%)	1-AS
	0.2620	0.2725 (4%)	0.2316 (-11.6%)	2-S
	0.3001	0.3126 (4.2%)	0.2722 (-9.3%)	2-AS
	0.3739	0.4094(9.5%)	0.3275(-12.4%)	3-S
	0.4254	0.4637 (9.0%)	0.3752(-11.8%)	3-AS
Lateral bending	0.0569	0.0564 (-0.8%)	0.0573(0.7%)	1-S
	0.1656	0.1631 (-1.5%)	0.1671(0.9%)	1-AS
	0.3125	0.3122 (-0.1%)	0.3084(-1.3%)	2-S
	0.5058	0.5053 (-0.1%)	0.5225(3.3%)	2-AS
Torsion	0.4133	0.4488(8.6%)	0.3509 (-15.1%)	1-S
	0.5389	0.5508 (2.2%)	0.4758 (-11.7%)	1-AS

Note: The values in bracket =(the corresponding column-column(1))/column(1)×100%.

Table 14 Effect of the tower height-to-span ratio on the seismic response

Member	Tower height-to-span ratio	Bending moment(kN.m)		Shear force(kN)		Axial force(kN)	Displacement(mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	1/4	2.22×10^6	3.20×10^5	2.69×10^4	2.32×10^4	2.28×10^5	1003	165	-
	1/5	2.62×10^6	3.00×10^5	2.89×10^4	2.42×10^4	2.19×10^5	909	152	-
	1/6	2.37×10^6	2.55×10^5	2.65×10^4	2.02×10^4	2.07×10^5	557	103	-
Girder	1/4	7.76×10^5	2.83×10^6	2.67×10^4	2.33×10^4	4.37×10^4	913	890	685
	1/5	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650
	1/6	6.35×10^5	2.87×10^6	2.26×10^4	2.37×10^4	3.91×10^4	418	578	263

Table 15 Effect of the side-to-main span ratio on natural frequency (Hz)

Modes	The side-to-main span ratio			Mode shape
	0.49 (1)	0.39 (2)	0.29 (3)	
Vertical bending	0.1811	0.1856(2.5%)	0.1871 (3.3%)	1-S
	0.2102	0.2161(2.8%)	0.2176(3.5%)	1-AS
	0.2620	0.2717 (3.7%)	0.2759 (5.3%)	2-S
	0.3001	0.3115 (3.8%)	0.3169 (5.6%)	2-AS
	0.3739	0.3832 (2.5%)	0.3873 (3.6%)	3-S
	0.4254	0.4390 (3.2%)	0.4437 (4.3%)	3-AS
Lateral bending	0.0569	0.0586 (3.0%)	0.0607 (6.6%)	1-S
	0.1656	0.1697 (2.5%)	0.1744 (5.3%)	1-AS
	0.3125	0.3206 (2.6%)	0.3437 (10.0%)	2-S
	0.5058	0.5199(2.8%)	0.5240 (3.6%)	2-AS
Torsion	0.4133	0.4139 (0.1%)	0.4137 (0.1%)	1-S
	0.5389	0.5486 (1.8%)	0.5367 (-0.4%)	1-AS

Note: The values in bracket =(the corresponding column-column(1))/column(1)×100%.

Table 16 Effect of the side-to-main span ratio on the seismic response

Member	The side-to-main span ratio	Bending moment(kN.m)		Shear force(kN)		Axial force(kN)	Displacement(mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	0.29	2.33×10^6	2.91×10^5	2.64×10^4	2.51×10^4	1.77×10^5	883	149	-
	0.39	2.52×10^6	2.92×10^5	2.73×10^4	2.35×10^4	2.07×10^5	892	150	-
	0.49	2.62×10^6	3.00×10^5	2.89×10^4	2.42×10^4	2.19×10^5	909	152	-
Girder	0.29	7.03×10^5	3.13×10^6	2.57×10^4	2.34×10^4	3.68×10^4	841	862	626
	0.39	7.34×10^5	2.92×10^6	2.59×10^4	2.43×10^3	2.91×10^4	866	865	630
	0.49	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650

seismic performance, the *A*-shaped tower is more favorable for super long-span cable-stayed bridges.

5.4 The tower height-to-span ratio

The tower height, which is generally calculated above from the deck, is closely related to the inclined angles of stay cables, and has an important influence on the static performance of cable-stayed bridges. The tower height-to-main span ratio is generally between 1/4 and 1/7, and mostly close to 1/5. In order to investigate the effect of tower height on the seismic performance of super long-span cable-stayed bridge, two case bridges with the tower height-to-span ratios of 1/6 and 1/4 respectively are designed. For simplicity, the cross sections of towers and stay cables are the same as the example bridge, however structural arrangement of the stay cables are changed with different tower height. Tables 13 and 14 show the effect of tower height-to-span ratio on natural frequency and the seismic response of the tower and girder respectively.

As shown in Table 13, the tower height has remarkable influence on the vertical bending and torsional frequencies, they are enlarged as the tower height increases. Therefore considering from the aspect of structural stiffness, the increase of tower height is favorable for cable-stayed bridges. However it is found from Table 14 that with the decrease of tower height, structural displacements of the tower and girder are both decreased, and at the same time the seismic internal forces are also reduced, the seismic performance of the bridge is therefore improved. Furthermore, the small tower height helps to improve the economy of cable-stayed bridges, and however it leads to the excess axial force in the deck and also the great reduction of structural stiffness. Under satisfying the static performance, the smaller tower height is favorable for super long-span cable-stayed bridge.

Table 17 Effect of auxiliary piers on natural frequency (Hz)

Modes	Number of auxiliary piers			Mode shape
	3 (1)	2 (2)	1 (3)	
Vertical bending	0.1811	0.1688(-6.8%)	0.1412(-22.0%)	1-S
	0.2102	0.1861(-11.5%)	0.1547(-26.4%)	1-AS
	0.262	0.2383(-9.0%)	0.2279(-13.0%)	2-S
	0.3001	0.2854(-4.9%)	0.2808(-6.4%)	2-AS
	0.3739	0.3637(-2.7%)	0.3532(-5.5%)	3-S
	0.4254	0.4088(-3.9%)	0.3744(-12.0%)	3-AS
Lateral bending	0.0569	0.0539(-5.3%)	0.0519(-8.8%)	1-S
	0.1656	0.1615(-2.5%)	0.1511(-8.8%)	1-AS
	0.3125	0.2964(-5.2%)	0.2621(-16.1%)	2-S
	0.5058	0.4255(-15.9%)	0.3182(-37.1%)	2-AS
Torsion	0.4133	0.405(-2.0%)	0.401(-3.0%)	1-S
	0.5389	0.5138(-4.7%)	0.4833(-10.3%)	1-AS

Note: The values in bracket =(the corresponding column-column(1))/column(1)×100%.

Table 18 Effect of the auxiliary piers on the seismic response

Member	Number of auxiliary piers	Bending moment(kN.m)		Shear force(kN)		Axial force(kN)	Displacement(mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	1	3.20×10^6	3.19×10^5	2.90×10^3	2.50×10^4	3.20×10^5	966	175	-
	2	2.74×10^6	3.14×10^5	3.45×10^4	2.14×10^4	3.04×10^5	922	160	-
	3	2.62×10^6	3.00×10^5	2.89×10^4	2.02×10^4	2.19×10^5	909	152	-
Girder	1	7.65×10^5	3.27×10^6	3.69×10^4	3.45×10^4	3.68×10^4	932	899	702
	2	7.47×10^5	3.00×10^6	3.22×10^4	2.72×10^4	3.48×10^4	902	889	689
	3	7.32×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650

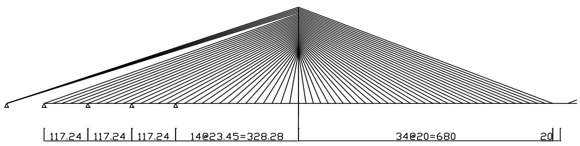

Fig. 6 The partially earth-anchored cable-stayed bridge scheme

5.5 The side-to-main span ratio

The side-to-main span ratio is also an important structural parameter affecting structural stiffness of the cable-stayed bridge. In order to reduce the deflection of the center span and improve the vertical stiffness, a small side-to-main span ratio is usually employed for long-span cable-stayed bridge, and it is generally between 0.25 and 0.5. To investigate the effect of the side-to-main span ratio on the seismic performance of super long-span cable-stayed bridge, under the same main span, two case bridges with a side span of 408 m and 544 m respectively are designed, the corresponding side-to-main span ratios are 0.29 and 0.39 respectively. Because the internal forces in the girder, towers and stay cables vary little with the side-to-main span ratio, for simplicity the cross sections of towers and stay cables are the same as the example bridge. The effect of the side-to-main span ratio on natural frequency and the seismic response of the tower and girder is shown in Tables 15 and6 respectively.

Table 15 shows that as the side span length decreases, the vertical and lateral bending frequencies are promoted remarkably, which indicates shorter side span is favorable to structural stiffness of cable-stayed bridge. As found in Table 16, the side span length has little influence on the lateral seismic response, however it has remarkable effect on the longitudinal and vertical seismic responses. With the decrease of side span length, the longitudinal and vertical seismic responses of the tower and girder are both reduced significantly, structural seismic performance is consequently improved. Therefore considering the aspects of both the static and seismic performance, shorter side span is favorable for super long-span cable-stayed bridges.

Table 19 Effect of the anchorage system of stay cables on natural frequency (Hz)

Modes	Fully self-anchored (1)	Partially earth-anchored (2)	Mode shape
Vertical bending	0.1811	0.1929(6.5%)	1-S
	0.2102	0.2302(9.5%)	1-AS
	0.2620	0.2840(8.4%)	2-S
	0.3001	0.3217(7.2%)	2-AS
	0.3739	0.3847(2.9%)	3-S
	0.4254	0.4356(2.4%)	3-AS
Lateral bending	0.0569	0.0584(2.7%)	1-S
	0.1656	0.1692(2.2%)	1-AS
	0.3125	0.3197(2.3%)	2-S
	0.5058	0.5215(3.1%)	2-AS
Torsion	0.4133	0.4178(1.1%)	1-S
	0.5389	0.5518(2.4%)	1-AS

Note: The values in bracket =(the corresponding column-column(1))/column(1)×100%.

Table 20 Effect of the anchorage system of stay cables on the seismic response

Member	Anchorage system	Bending moment(kN.m)		Shear force(kN)		Axial force(kN)	Displacement(mm)		
		Longitudinal	Lateral	Longitudinal	Lateral		Longitudinal	Lateral	Vertical
Tower	Partially earth-anchored	2.66×10^6	3.15×10^5	3.10×10^4	3.20×10^4	3.08×10^5	858	141	-
	Fully self-anchored	2.62×10^6	3.00×10^5	2.89×10^4	2.42×10^4	2.19×10^5	909	152	-
Girder	Partially earth-anchored	6.30×10^5	3.02×10^6	2.82×10^4	2.51×10^4	3.48×10^4	859	775	641
	Fully self-anchored	7.52×10^5	2.93×10^6	2.88×10^4	2.43×10^4	4.06×10^4	893	870	650

5.6 The auxiliary piers in side spans

In order to improve structural behavior and construction safety, several auxiliary piers are usually set in the side spans of cable-stayed bridge. To investigate the effect of auxiliary piers in side spans on the seismic performance of cable-stayed bridge, based on the example bridge, two case bridges with one and two auxiliary piers in each side span respectively are designed. For simplicity, the cross sections of towers and stay cables are the same as the example bridge. The effect of the auxiliary piers on natural frequency and the seismic response of tower and girder is presented in Tables 17 and 18 respectively.

With the decrease of auxiliary piers in the side spans, all the natural frequencies are decreased significantly, and therefore the total structural stiffness is depressed. Moreover as found in Table 18, greater seismic responses of tower and girder are achieved. Therefore, the installation of auxiliary piers in side spans is favorable to improve both the static and seismic performances of cable-stayed bridge. It is to be noted that the optimum number of auxiliary piers in side spans should be considered comprehensively from the aspects of economy, static performance and construction safety.

5.7 The stay cable anchorage system

Generally, the stay cables of cable-stayed bridge are all anchored at the girder and tower, and for such a bridge, it is called as the fully self-anchored cable-stayed bridge. To overcome the shortcoming of excess axial force in the girder near the tower of self-anchored cable-stayed bridge, other anchorage systems of stay cables such as the earth-anchored system and the partially earth-anchored system are proposed (Gimsing and Georgakis 2012). In order to investigate the effect of the stay cable anchorage system on the seismic performance of super long-span cable-stayed bridge, a partially earth-anchored cable-stayed bridge as shown in Fig. 5, is designed in which the outmost 5 stay cables in each cable planes of side span are set to be earth-anchored, and the other 29 stay cables are still anchored at the girder. Except the stay cables, the cross sections of towers and girder are the same as the example bridge. The effect of stay cable anchorage system on natural frequency and the seismic response of tower and girder is presented in Tables 19 and 20 respectively.

As compared to the fully self-anchored cable-stayed bridge, all the natural frequencies as shown in Table 19 are increased in the case of partially earth-anchored cable-stayed bridge, which means structural stiffness is improved. As shown in Table 20, the seismic internal force of tower is slightly increased, however its seismic displacement is remarkably decreased in the case of partially earth-anchored cable-stayed bridge, which indicates that the constraint of stay cables for the tower is enhanced; as for the girder, the lateral seismic internal force is slightly increased, however its vertical seismic internal force is reduced, and particularly all the displacements are significantly decreased, which indicates that the support effect of stay cables for the girder is also enhanced. Therefore, as compared to the fully self-anchored cable-stayed bridge, the partially earth-anchored cable-stayed bridge has greater structural stiffness and better seismic performance and becomes more favorable for super long-span cable-stayed bridge.

6. Conclusions

In this work, the seismic response of a super long-span cable-stayed bridge with 1400 m main span under the E1 horizontal and vertical seismic excitations is investigated numerically by the response spectrum and time-history analysis respectively, the seismic performance and the effect of structural nonlinearity on the seismic response of cable-stayed bridge are revealed. Furthermore, the effect of structural parameters including the girder depth and width, the tower structural style, the tower height-to-span ratio, the side-to-main span ratio, the auxiliary piers in side spans and the anchorage system of stay cables etc on structural dynamic characteristics and the seismic performance of super long-span cable-stayed bridge is investigated, and some important conclusions can be drawn as follows:

(1) The horizontal seismic excitation produces significant seismic response of the girder and tower, there exists the greatest seismic internal forces in the tower bottom section and the girder sections at the tower, and therefore particular attention should be paid to the seismic design of these sections.

(2) The geometric nonlinearity has significant influence on the seismic response, and thus the geometric nonlinear time-history analysis is proposed to accurately predict the seismic response of long and super long-span cable-stayed bridges.

(3) In the cases of A-shaped towers, smaller tower height, shorter side span, several auxiliary piers set in the

side spans and several earth-anchored stay cables instead of the self-anchored stay cables, the super long-span cable-stayed bridge has smaller seismic response and better seismic performance. However the optimum values of these design parameters should be further investigated through combining the economy, the static performance and construction safety.

Acknowledgments

This research was financially supported by Zhejiang Provincial Natural Science Foundation of China under Grant No. LY18E080034 and LY15E080020, which were gratefully appreciated.

References

Camara, A. and Astiz, M.A. (2012), "Pushover analysis for the seismic response prediction of cable-stayed bridges under multi-directional excitation", *Eng. Struct.*, **41**, 444-455.

Gimsing, N.J. and Georgakis, C.T. (2012), *Cable-supported Bridges-Concept & Design*, 3nd Edition, John Wiley & Sons Ltd., Chichester, England, UK.

Han, Z.F. and Ye, A.J. (2015), "Research on the longitudinal earthquake-reduction system of cable-stayed bridges over kilometers", *Earthq. Eng. Eng. Dyn.*, **35**(6), 64-70.

Ismail, M., Casas, J.R. and Rodellar, J. (2013), "Near-fault isolation of cable-stayed bridges using RNC isolator", *Eng. Struct.*, **56**, 327-342.

JTG/T B02-01 (2008), Guidelines for Seismic Design of Highway Bridges, Ministry of Communications, Beijing, China.

Naderian, H., Cheung, M.S. and Shen, Z.Y., Dragomirescu, E. (2016), "Seismic analysis of long-span cable-stayed bridges by an integrated finite strip method", *J. Bridge Eng.*, ASCE, **21**(3), 1-18.

Nagai, M., Fujino, Y., Yamaguchi, H. and Iwasaki, E. (2004), "Feasibility of a 1400 m span steel cable-stayed bridge", *J. Bridge Eng.*, ASCE, **9**(5), 444-452.

Sharabash, A.M. and Andrawes, B.O. (2009), "Application of shape memory alloy dampers in the seismic control of cable-stayed bridges", *Eng. Struct.*, **31**, 607-616.

Soyluk, K. and Sicacik, E.A. (2012), "Soil-structure interaction analysis of cable-stayed bridges for spatially varying ground motion components", *Soil Dyn. Earthq. Eng.*, **35**, 80-90.

Sun, B. (2008), "Study on structural system of thousand-meter scale cable-stayed bridges", Ph.D. Dissertation, Tongji University, Shanghai. (in Chinese)

Wang, D. and Huang, P.M. (2008), "Research on deck-tower connection of super long-span cable-stayed bridge", *J. Zhengzhou Univ. (Eng. Sci.)*, **29**(4), 112-115.

Wu, F.W., Xu, C. and Zhao L. (2014), "Analysis on sensitivity of stochastic seismic response parameters of super long-span cable-stayed bridges", *J. China Railw. Soc.*, **36**(6), 107-113.

Xiang, H.F. (2012), "The trend of the world's mega-scale bridges-An inspiration of IABSE 2011 in London", *Bridge*, **3**, 12-16.

Ye, A.J. and Fan, L.C. (2007), "Lateral constraint systems for super-long-span cable-stayed bridge", *China J. Highw. Tran.*, **20**(2), 63-67.

Zhang, X.J. and Yu, Z.J. (2015), "Study of seismic performance of cable-stayed-suspension hybrid bridges", *Struct. Eng. Mech.*, **55**(6), 1203-1221.

Zhu, J., Zhang, W., Zheng, K.F. and Li, H.G. (2015), "Seismic design of a long-span cable-stayed bridge with fluid viscous dampers", *Prac. Period. Struct. Des. Constr.*, **21**(1), 04015006.

CC