Technical Note

Conceptual Design of a New Three-Tower Cable-Stayed Bridge System with Unequal-Size Fans

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Abstract: Multitower cable-stayed bridges with three or more towers often have economic advantages over ultralong-span double-tower cable-stayed bridges or suspension bridges, in situations when deep foundations are not required. However, the internal towers of multitower cable-stayed bridges are not connected to stiff supports or foundations, and therefore the stiffness of the internal towers is lower than the side towers. As a result, when unbalanced live loads are applied to one main span, the deformation and internal forces of the internal towers and the main girder can be excessively large. Therefore, solving the low stiffness problem of the internal towers is an important issue for multitower cable-stayed bridges. In this paper, a new type of three-tower cable-stayed bridges is proposed. Since the stiffness contributed by the flanking towers is much greater than the central tower, the proportion of the main span supported by flanking tower cables can be increased, while the span supported by the central tower can be reduced. This can be achieved by modifying the design of the three fans, which originally had equal sizes. This new system is therefore called a three-tower cable-stayed bridge with unequal-size fans. The stiffness, internal forces, and cost of the new system were compared to the conventional three-tower cable-stayed bridges with identical fans, and it was found that this new system could be an excellent alternative to the conventional designs. **DOI: 10.1061/(ASCE)BE.1943-5592.0001257.**

Author keywords: Three-tower cable-stayed bridge; Stiffness; Displacement; Internal force; Vehicle load.

Introduction

Three-tower cable-stayed bridges have become increasingly attractive among long-span bridges because of their excellent spanning capability and strong adaptability to geological conditions with their self-anchoring systems. Unlike conventional double-tower cable-stayed bridges, the central tower of three-tower cable-stayed bridges is not connected to a stiff support or foundation that can effectively restrain the displacement, resulting in a lower overall stiffness, which has limited their applications in practice (Zhao 2006).

To improve the overall stiffness of the three-tower cable-stayed bridges, different methods have been attempted in practice. The method of increasing the stiffness of the tower was adopted on the Rion-Antionion Bridge in Greece (Teyssandier 2002). For the Ting Kau Bridge in Hong Kong, sloping-stabilizing cables from the top of each internal tower to the junction of the deck with the adjacent towers were used (Bergermann and Schlaich 1996). In the Forth Replacement Crossing in Scotland, overlapping stay cables were used in the midspan region to increase the overall bridge stiffness (Carter et al. 2009).

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For the three-tower cable-stayed bridges, the stiffness contributed by the flanking towers is much greater than the central tower. In view of this feature, a new three-tower cable-stayed bridge system was proposed in this study, based on an improvement of the views proposed by Shao et al. (2014) and Kite et al. (2011). In this new system, the proportion of the main span supported by the flanking tower cables was increased while the span supported by the central tower cables was reduced. This can be achieved by modifying the size of the three fans which usually have an equal size. The new system was therefore called the three-tower cable-stayed bridge with unequal-size fans. With this new design, the overall stiffness of the cable-stayed bridge is improved and the risk during the construction is reduced by shortening the construction length of the double-cantilever beam of the central tower. The rationality of the new system was investigated through comparison with a cable-stayed bridge with overlapping cables at the midspans. The influence of some important parameters on the stiffness and economy of the new system was also explored.

Comparison of a Bridge System with Overlapping Stay Cables and the Proposed Bridge System

Two Designs

The proposed cable-stayed bridge system with unequal-size fans was inspired by the desire to optimize of the conventional cable-stayed bridge system with overlapping cables in the midspan, as shown in Fig. 1(a). In this bridge system, the main girder within the crossing cable region is supported by both the cables from the flanking towers and the cables from the central tower. When an out-of-balance live load is applied to one main span, the tower movement causes the cables to lift the adjacent main span. Over the region of the overlapping cables decompression is developed in the cables connected to the far flanking tower which is in turn tied back to the far anchor piers (Kite et al. 2010).

Considering that the cables from the flanking towers are end-anchored and can provide larger contribution to the bridge overall stiffness as than the cables from the central tower, a new system, as shown in Fig. 1(b), was proposed in the present study. Compared to the conventional system shown in Fig. 1(a), in this new system, the proportion of the main span supported by the flanking towers is increased while the span length supported by the central tower is reduced. In this way, the height of the central tower is reduced, and the stay cables are shortened. In addition, the overlapping stay cables from the central tower can be removed. Unlike the case with overlapping stay cables, the stay cables within the previous overlapping zone in the new system need to take the loads of the main girder alone; therefore, larger cable dimensions are required. As can be seen from Fig. 1(b), the new system has a shorter central tower than the conventional system in Fig. 1(a) and the sizes of the fans are different between the flanking towers and the central tower. This new system is therefore called a three-tower cable-stayed bridge with unequal-size fans.

Calculation Models for the Bridge with Overlapping Stay Cables and the Proposed Bridge System

To compare the features of the two different systems, finite element models for the two bridge systems were established. A main span of 600 m was selected as the basis for comparison, as shown in Fig. 1. Diamond-shaped pylons were used for both bridge systems and a 4.5-m-deep main girder was adopted, which is shown in Fig. 2. The main girder is made of steel and covered by an ultra-high performance concrete (UHPC) deck. A schematic of the cross section of the main girder is shown in Fig. 2 with detailed parameters summarized in Table 1. The cross-sectional areas of a

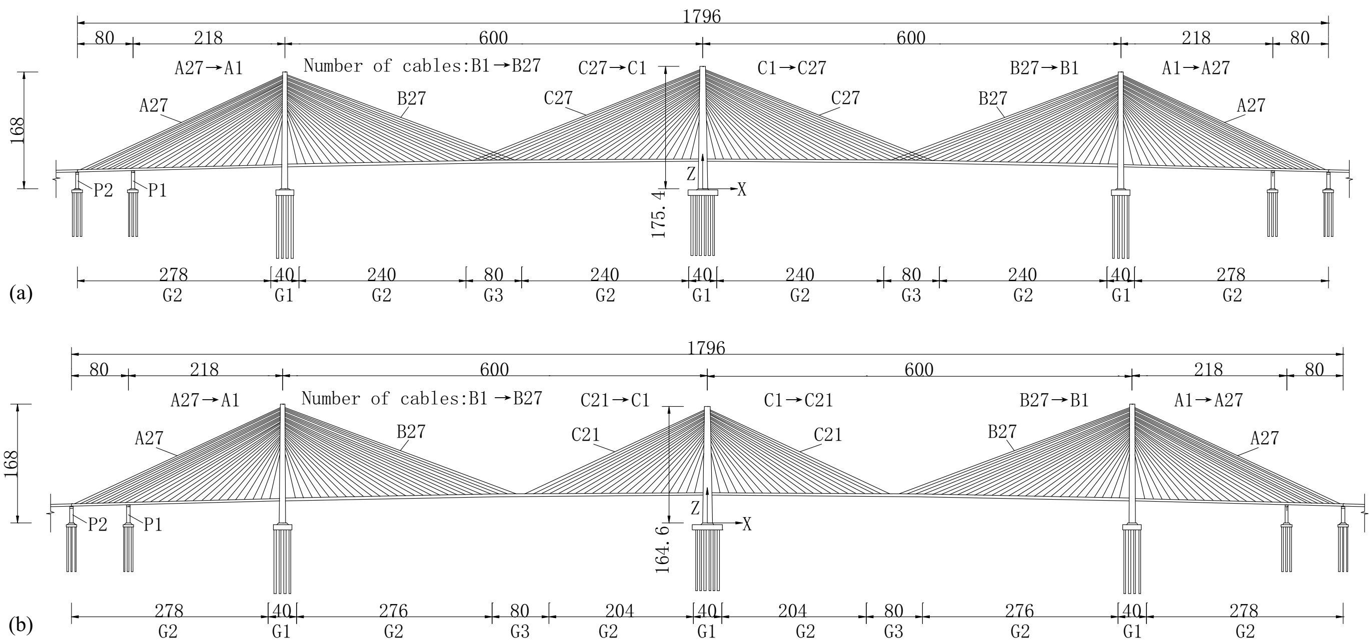

Fig. 1. Elevation layout of the bridge (unit: m); G1-G3 are girder section numbers, with section properties summarized in Table 1: (a) a conventional cable-stayed bridge with overlapping stay cables and (b) a proposed new cable-stayed bridge system

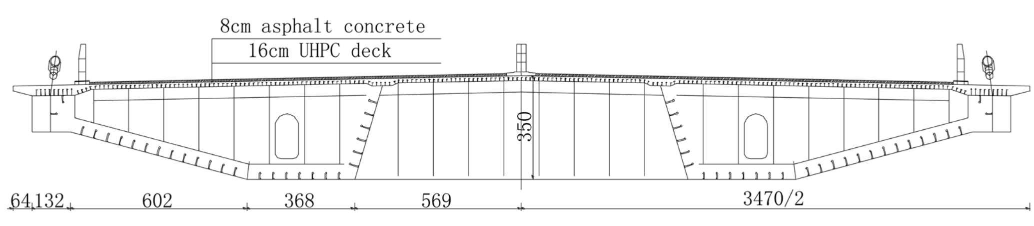

Fig. 2. Schematic of cross section of the main girder (unit: cm)

Table 1. Section Properties of Girder

Section	Area (m^2)	$I_{yy}(m^2)$	$I_{zz}(m^2)$	$I_{xx}(m^2)$	Roof (mm)	Middle web (mm)	Side web (mm)	Floor (mm)	Inclined floor (mm)
G1	11.4192	15.6131	1440.60	28.7112	160	20	28	20	16
G2	10.8519	13.7248	1397.13	25.4385	160	16	28	16	14
G3	11.1071	13.7755	1463.82	25.4952	160	16	24 × 2	16	14

Table 2. Cross-Sectional Area of Cables of Overlapping Stay Cables Scheme (Unit: mm^2)

Section	Area
A27	7,280
A26	7,280
A25	7,280
A24	7,280
A23	7,280
A22	7,280
A21	8,120
A20	8,120
A16	7,280
A11	6,020
A6	4,760
A1	4,340
B1	4,760
B6	4,340
B11	6,020
B16	6,860
B20	7,700
B21	7,700
B22	6,860
B23	7,280
B24	7,700
B25	8,120
B26	8,960
B27	8,960
C27	10,220
C26	8,960
C25	7,700
C24	7,280
C23	7,280
C22	7,280
C21	8,540
C20	8,120
C16	7,280
C11	6,020
C6	4,340
C1	4,760

Table 3. Cross-Sectional Area of Cables of the Proposed Scheme (Unit: mm^2)

Section	Area
A27	16,240
A26	16,240
A25	15,400
A24	14,980
A23	14,560
A22	14,140
A21	8,120
A20	8,120
A16	7,280
A11	6,020
A6	4,760
A1	4,340
B1	4,760
B6	4,340
B11	6,020
B16	6,860
B17	7,280
B18	7,280
B19	7,700
B20	7,700
B21	7,700
B22	17,080
B23	16,240
B24	15,400
B25	15,400
B26	16,240
B27	16,240
C21	8,540
C20	8,120
C19	8,120
C18	7,700
C17	7,280
C16	7,280
C11	6,020
C6	4,340
C1	4,760

few representative cables of the two schemes are shown in Tables 2 and 3.

In the finite element model of the bridge, beam element was used to simulate the main girder and the tower, and truss element was used to simulate the cables. For boundary conditions, all degrees-of-freedom of the pylon foot nodes were fixed. The main girder had vertical rigid support at the piers and was coupled with the closest node in each pylon to restrain the vertical movement, transverse movement, and rotation. The coupling nodes at the central tower also provide restraint to the longitudinal movement.

The following conditions and assumptions were used in the modeling process:

The traffic load grade used follows the Chinese code "General Code for Design of Highway Bridges and Culverts" (Ministry of Communications of P.R. China 2015). The traffic load is applied to the most unfavorable position on the basis of the influence line. Wind load is determined in accordance with the AASHTO-LRFD Bridge Design Specifications (AASHTO 2010). The loads specified and their material properties are described below:

1. the lane load consists of a uniformly-distributed load of 10.5 kN/m and a concentrated load of 360 kN;
2. the secondary dead load is 62.5 kN/m;
3. the design wind speed is 31.7 m/s;
4. the tensile strength of stay cables is 1860 Mpa, and the cable cross-sectional area is 3920–17080 mm^2;
5. the bridge deck is a 160-mm thick UHPC plate: the compressive stress limit of UHPC is $0.6 \times 150 = 90$ MPa (JSCE 2006) and the elastic modulus is 45 Gpa, while the design strength of the steel girder is 210 MPa and the elastic modulus is 200 Gpa; and
6. the pylon concrete grade is C55 with a design compressive strength of 22.4 MPa and an elastic modulus of 36 GPa.

The Results from Different Systems

Finite element analysis was performed and the axial force of the main girder under the dead load was obtained for the two different bridge systems. The results are shown in Fig. 3.

From the obtained results, it can be seen that under the action of the dead load, the maximum axial force of the conventional system was 148,088.8 kN, which appeared at the bearing position of the central tower. While for the proposed system, the maximum axial

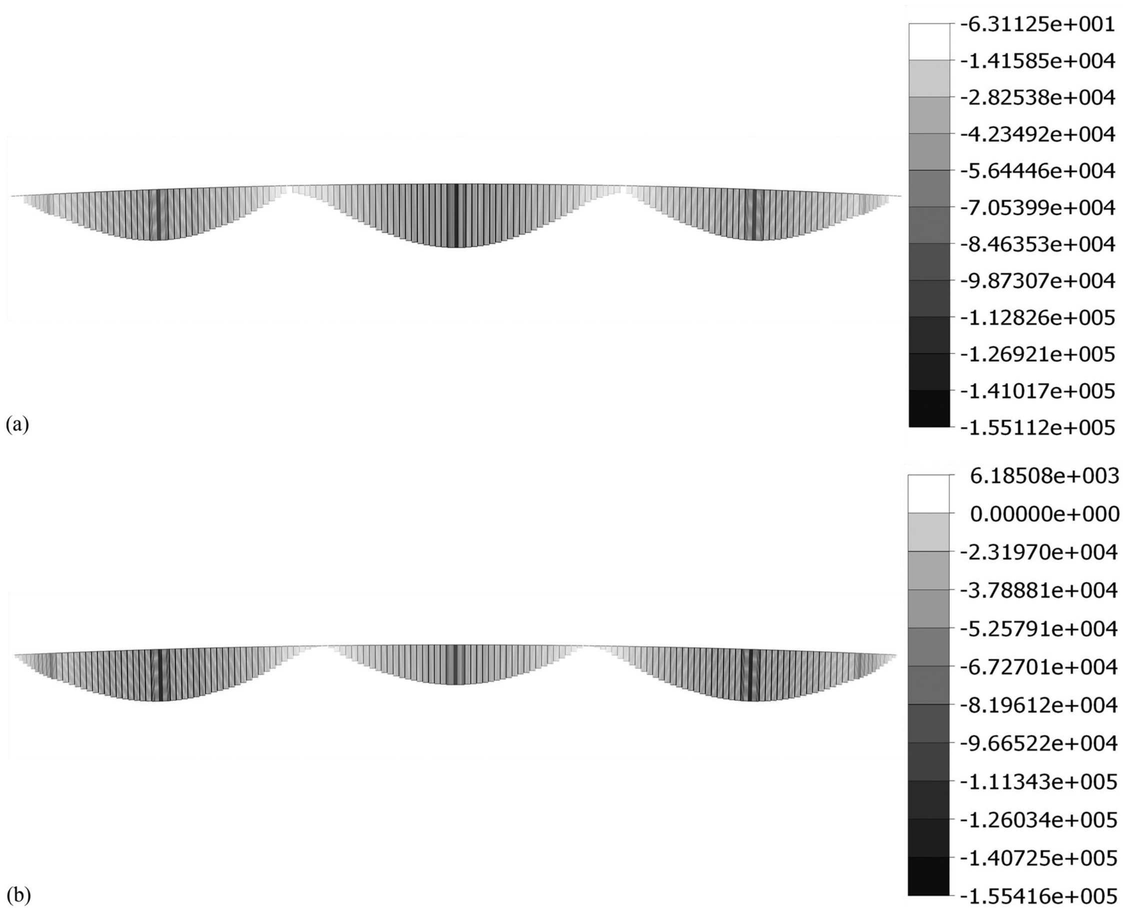

Fig. 3. Comparison of the axial force of the two different bridge systems under the dead load (unit: kN): (a) overlapping stay cables scheme and (b) proposed scheme

force was 155691.4 kN, which appeared at the bearing position of the flanking towers. Under the most unfavorable vehicle-loading scenario, the deflection of the main girder is shown in Fig. 4 for the two bridge systems, respectively. For both systems, the maximum deflection of the main girder occurred at the middle of the main span slightly to the side of the central tower. The maximum girder deflection was 920.7 mm for the system with overlapping stay cables and 879.3 mm for the system with unequal-size fans, which was 4.7% less than the former.

The main parameters of the two systems and loading results are shown in Table 4. From these data, the height of the central tower of the proposed system can be seen to be lower than that of the conventional system with overlapping stay cables and therefore the amount of stay cables is also reduced. However, the use of stay cables on the flanking towers is increased in the proposed system due to the increased diameters of the cables crossing the midspan. In addition, due to the constraint of the stabilizing cables from the flanking towers and the reduction of the height of the central tower, the maximum deflection and upturn of the main girder are both reduced.

Table 5 shows the maximum and minimum stresses in the towers, cables and girders of the two systems. Compared with the conventional system, which uses overlapping stay cables, the maximum compressive stresses of the main girder and the tower in the proposed system increase while the maximum tensile stress does not change much. However, the maximum tensile stress of the stay cable increases significantly, due to the length change of the stay cables and the inclination angle.

The first few vibration modes of the two schemes are shown in Fig. 5, where it can be seen that the fundamental vibration modes for both schemes feature the vertical symmetric bending of the main girder. The dynamic characteristics of the two schemes appear to be similar.

Effect of Fan Sizes on the Stiffness of Cable-Stayed Bridges

To further study the effect of the number of stay cables anchored on different towers on the structural performance of the bridge, Fig. 6(a) shows the conventional system with equal-size fans for all three towers and no stay cables cross the midspan while Fig. 6(b) shows an example with two pairs of stay cables crossing the midspan in the proposed system. As a result of the change in the number of stay cables, the height of the central tower changed, as shown in Fig. 6(b).

In the modification of the configuration of the bridge in the proposed system, the following principles were followed.

1. The vertical projection area of the cable at each anchor point in the main span was to be the same before and after the adjustment.
2. The horizontal projection area of the cable in the end anchorage region was the same as that of the corresponding cable in the tower.
3. Under the action of the dead load and live load, the stress of the steel girder was controlled to be less than 120MPa, and the stress limit for the UHPC material was set to 24.5MPa.

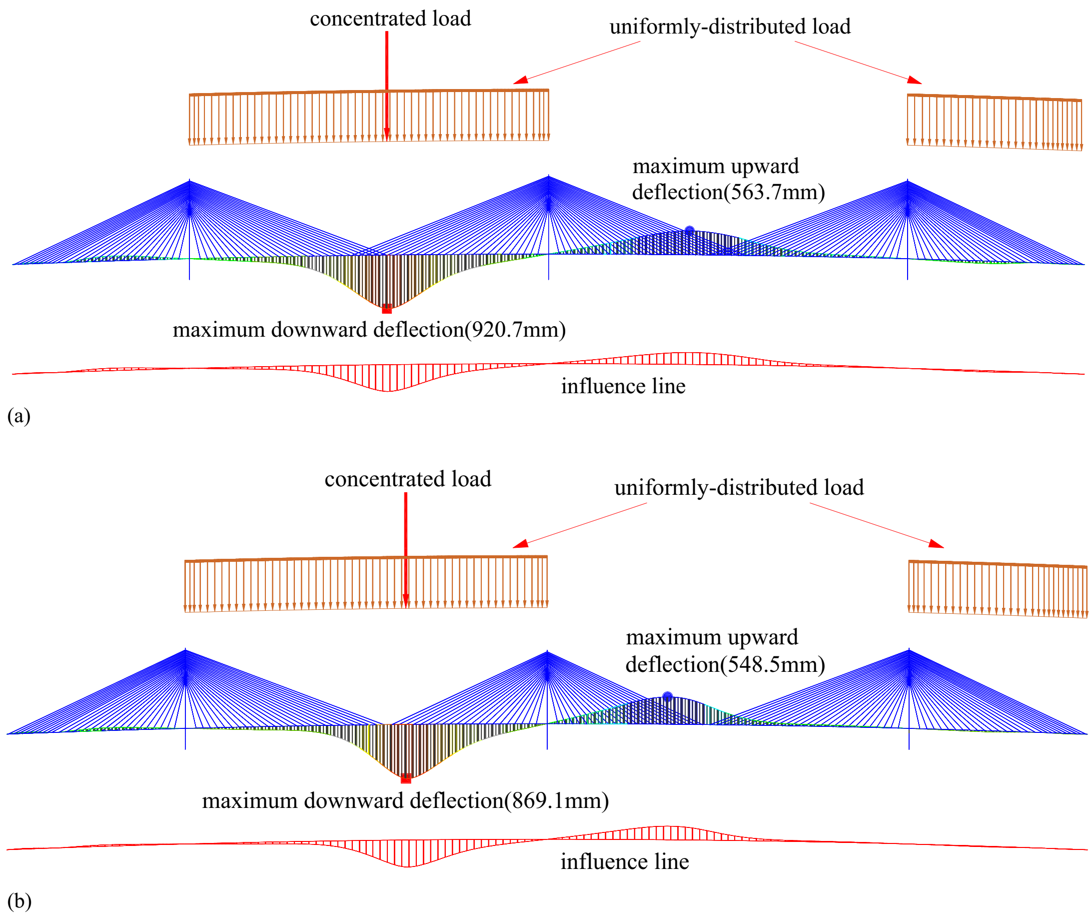

Fig. 4. The most adverse deformation of two different bridge systems: (a) a bridge with overlapping stay cables and (b) a bridge with unequal-size fans

Table 4. Comparison of Calculation Results of Main Parameters

Item	System with overlapping stay cables	Proposed system	Ratio
The flanking tower height (m)	167.9	167.9	1.000
The central tower height (m)	175.4	164.6	1.066
Maximum axial force of main girder at flanking tower (kN)	130,896.6	155,691.4	0.841
Maximum axial force of main girder at central tower (kN)	148,088.8	119,040.7	1.244
Total weight of cables of the flanking tower (t)	2,441.3	3,413.0	0.715
Total weight of cables of the central tower (t)	1,275.9	762.6	1.673
Maximum transverse displacement of main girder under wind load (mm)	169.9	169.1	1.005
Maximum transverse displacement of the flanking tower under wind load (mm)	63.7	64.1	0.994
Maximum transverse displacement of the central tower under wind load (mm)	63.0	56.2	1.121
Maximum deflection of main girder (mm)	920.7	879.3	1.047
Maximum upturn of main girder (mm)	563.7	555.0	1.015
Total deflection of main girder (mm)	1,484.4	1,434.3	1.035

4. In the process of adding cables, the cable spacing on the tower was arranged at 1.8 m, and the cable spacing on the girder was maintained at 12 m. At the same time, the height of the flanking towers was raised by 1.8 m each time when adding one cable, while the height of the central tower was lowered by 1.8 m.

As the number of flanking tower cables crossing the midspan was changed, the deflection of the main girder was investigated, and the results are plotted in Fig. 7(a). It can be seen from Fig. 7(a) that as the number of flanking tower cables crossing the midspan increased, the portion of the main span that can be restrained by the flanking tower increased, leading to an increase in the overall stiffness of the bridge. However, at the same time, the stiffness of the cables on the flanking tower also decreased due to the increase of the cable length. In addition, the stiffness of the cables on the central tower increased as the cable length decreased. Thus, the cable-stayed bridge system will reach maximum stiffness, with a certain number of the flanking tower cables crossing the midspan.

Table 5. Stress Comparison (Unit: MPa)

Position	Item	System with overlapping stay cables	Proposed system	Ratio
Top of the girder (UHPC)	Maximum value	5.4	5.2	1.038
	Minimum value	−20.3	−24.8	0.819
Bottom of the girder (steel)	Maximum value	12.0	11.3	1.062
	Minimum value	−96.2	−104.4	0.921
Tower	Maximum value	8.6	6.9	1.246
	Minimum value	−22.4	−27.3	0.821
Cable	Maximum value	749.3	876.0	0.855
	Minimum value	388.5	249.7	1.556

As can be seen from Fig. 7(a), the downward deflection reached a minimum value of 811.0 mm, with seven pairs of flanking tower cables crossing the midspan. The maximum value of the upward deflection decreased monotonically with the decrease of the number of central tower cables. The total deflection reached a minimum value of 1,116.4 mm when 14 pairs of cables on the flanking tower crossed the midspan. For comparison, the total deflection of the system with overlapping stay cables was 920.7 mm (downward) + 563.7 mm (upward) = 1,484.4 mm. The total deflection allowed in the Chinese code is 1/400 (1500 mm) of the main span for cable-stayed bridges. By comparison, the total deflection of the main girder of the proposed system was 95.5% (1,417.6 /1,484.4 mm), 83.2% (1,235.3 mm/1,484.4 mm)

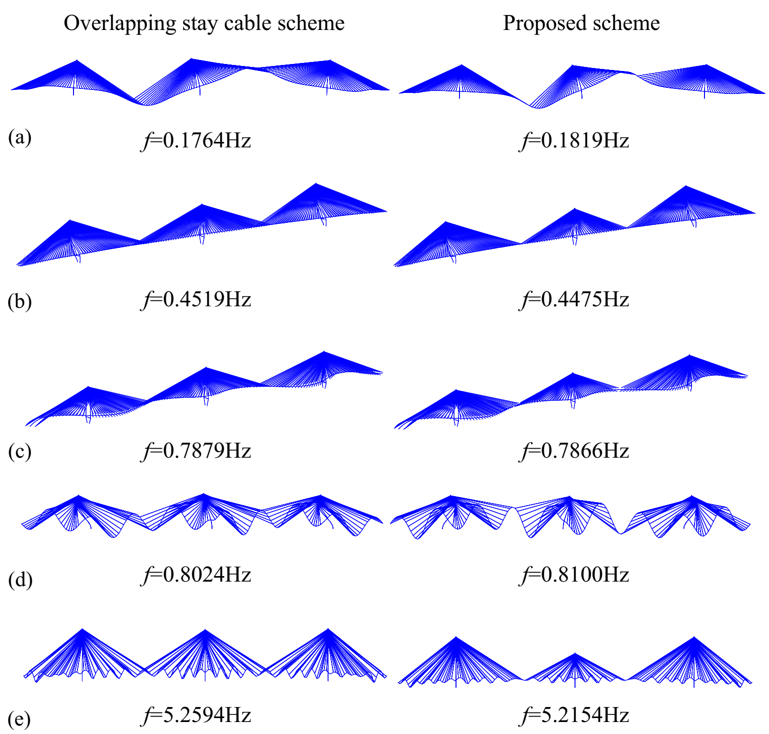

Fig. 5. Comparison of parts of vibration modes of two schemes: (a) 1st-mode shape, fundamental mode; (b) 9th-mode shape, lateral symmetric bending of the main girder; (c) 19th-mode shape, torsion of the main girder; (d) the 20th-mode shape, longitudinal floating of the main girder; and (e) 77th-mode shape, vertical symmetric bending of the main girder

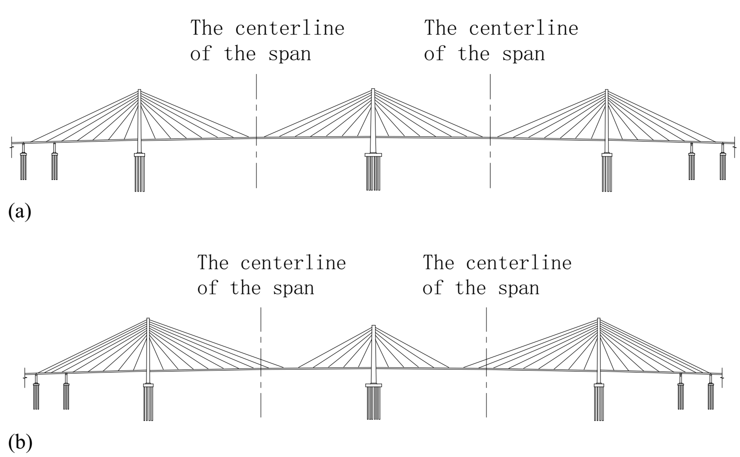

Fig. 6. Schematic diagram of cable-stayed bridge with size fans: (a) cable-stayed bridge with three equal-size fans (conventional system) and (b) cable-stayed bridge with two pairs of flanking tower cables crossing the midspan

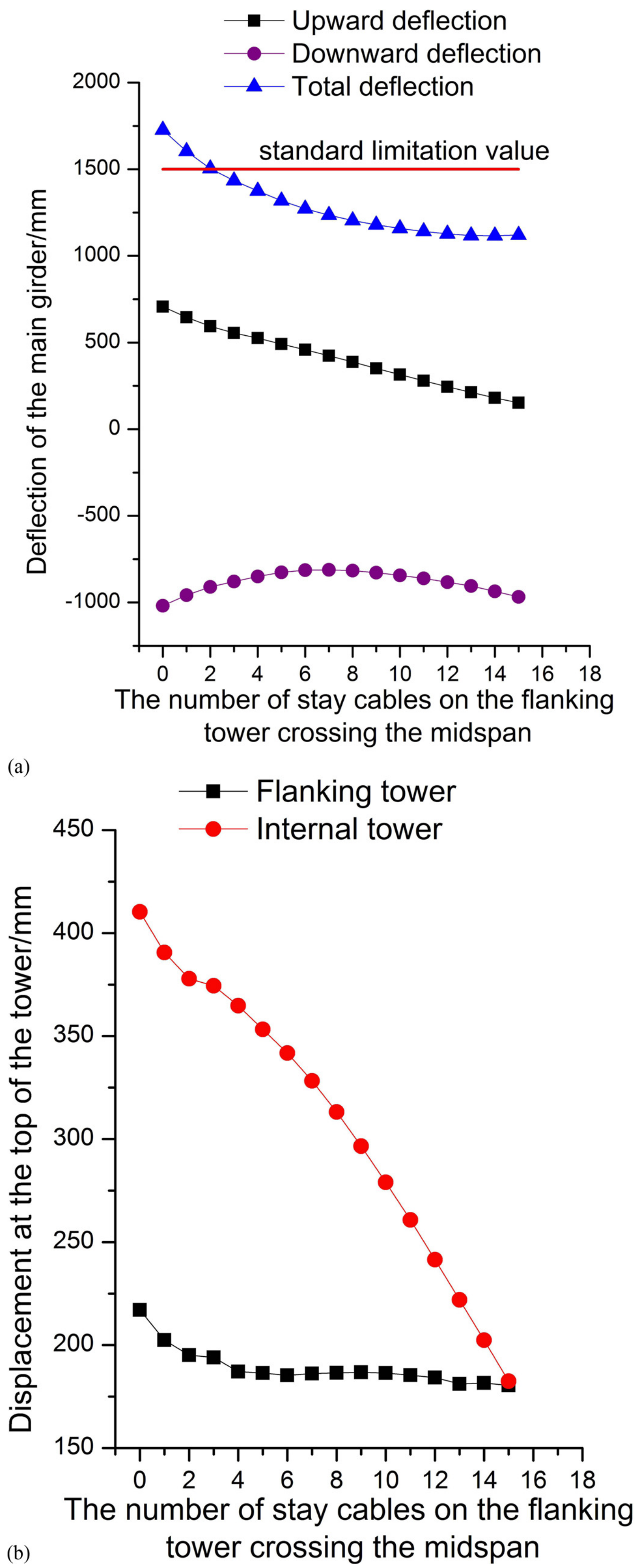

Fig. 7. Displacement of the proposed bridge system under the action of most unfavorable live load: (a) deflection of the main girder and (b) variation of the deflection at the top of the tower

Table 6. Material Consumption and Cost Breakdown of the Five Schemes

Item	Unit	Comprehensive unit price (CNY)	Overlapping stay cables (C_0)		0 pairs of crossing cables (C_1)		3 pairs of crossing cables (C_2)		7 pairs of crossing cables (C_3)		14 pairs of crossing cables (C_4)	
			Quantity	Cost (10,000 CNY)	Quantity	Cost (10,000 CNY)	Quantity	Cost (10,000 CNY)	Quantity	Cost (10,000 CNY)	Quantity	Cost (10,000 CNY)
Concrete tower	m^3	3,900	27,135	10,583	26,488	10,330	26,629	10,385	26,838	10,467	27,179	10,600
Cable	t	30,000	3,717	11,151	3,663	10989	4,175	12,525	4,887	14,661	6,472	19,416
UHPC deck	m^3	9,000	9,949	8,954	9,949	8,954	9,949	8,954	9,949	8,954	9,949	8,954
Steel girder	t	20,000	15,973	31,946	15,973	31,946	15,966	31,932	16,673	33,346	18,405	36,810
Total cost (Ci/C0 (%))	10,000 CNY			62,634 (100.0%)		62,219 (99.3%)		63,796 (101.9%)		67,428 (107.7%)		75,780 (120.9%)

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and 75.2% (1,116.4 mm/1,484.4 mm) of that of the conventional system with overlapping stay cables, respectively, when the number of pairs of flanking tower cables crossing the midspan was 3, 7, and 14.

The variation of displacement at the top of the tower under vehicle load with the number of flanking tower cables crossing the midspan in the proposed system is plotted in Fig. 7(b). As can be seen in Fig. 7(b), the tower top displacement of the central tower decreases with the decrease of tower height. However, due to the restraint by the stiff anchor system, the change of the displacement at the top of the flanking tower was very small.

In the proposed bridge system, with the number of cables on the flanking towers increasing, the axial forces in the main girder supported by the flanking tower cables also increased, leading to an increase in the use of steel and cables. Meanwhile, the axial forces in the main girder supported by the central tower decrease, leading to the reduced use of steel for the main girder supported by the central tower cables. Table 6 shows the amount of superstructure materials used in four schemes with different numbers of flanking tower cables crossing the midspan (0, 3, 7, and 14). For the purpose of comparison, the cost of the conventional system with overlapping stay cables was also calculated and provided. The unit prices were taken from Sun et al. (2013).

In Table 6, the total cost of the superstructure in the proposed system increases with the increasing number of stay cables crossing the midspan. In addition, the total cost of the proposed system is higher than that of the conventional bridge system with overlapping stay cables in that the cost of the cables will increase rapidly when the number of cables crossing the midspan increases. Based on the consideration of both the bridge stiffness and overall cost, it was found that using three stay cables crossing the midspan can not only ensure stiffness requirements but also effectively control costs.

Since was is no auxiliary pier for the central tower of the multi-tower cable-stayed bridge, the double cantilever length, which is the most dangerous part of the construction, was therefore much longer than that of the flanking tower. One distinct advantage of the proposed bridge system is that the length of double cantilever construction for the main girder at the central tower zone was reduced, thus reducing complexity and improving safety during construction. Before the application of this proposed system in practice, a parametric study should be performed, and the configuration of the proposed bridge system should be optimized, based on the considerations of both the bridge stiffness and the total cost.

Conclusion

To improve the overall stiffness of the three-tower cable-stayed bridge, a new three-tower cable-stayed bridge system with unequal-size fans was proposed in the present study. Based on finite element analysis results using a three-tower cable-stayed bridge with two equal 600-m spans, it was found that with an optimized number of stay cables crossing the midspan, the bridge stiffness can be substantially increased as compared with the conventional bridge system with overlapping stay cables. The results show that for this design, the proposed scheme with three pairs of flanking tower cables crossing the midspan can reduce the deflection of the main girder by 4.5%, while the total cost of the proposed system was only 1.9% higher than that of the overlapping system.

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