Design and Experimental Study of a Harp-Shaped Single Span Cable-Stayed Bridge

Xudong Shao[1]; Hua Zhao[2]; Lifeng Li[3]; Wanghu Peng[4]; Guangdong Liu[5]; and Banfu Yan[6]

Abstract: This paper presents issues in the design concept, analysis, and test results of a harp-shaped single span cable-stayed bridge, Hongshan Bridge, located in Changsha, Hunan Province, China. The bridge has a 206 m span, with a pylon inclined at 58° from the horizontal and 13 pairs of parallel cable stays without a back stay. This paper discusses the design approach for the main components of the bridge. Emphasis will be put on the following three aspects. First, the weight of the pylon and all dead loads of the main girder in addition to part of the live loads must be in a balanced condition. Second, the main girder should be an orthotropic steel-concrete composite box girder because of the superior safety and weight reduction of this type of structure. Third, the cable stays should be anchored at the neutral axis of the pylon to prevent the development of high secondary moments caused by other anchor approaches. Furthermore, based on results from tests carried out on three models, namely, scaled full model tests in a scale of 1:30, scaled section model tests in a scale of 1:6, and wind tunnel tests, the following four key issues were studied: (1) the local stability of orthotropic steel-concrete composite box girder subjected to combined bending and axial loads; (2) the characteristics under loads of 13-m-long cantilever beams; (3) the safety of the bridge under some other dangerous conditions; and (4) the characteristics of wind resistance and wind tunnel testing.

DOI: 10.1061/(ASCE)1084-0702(2005)10:6(658)

CE Database subject headings: Bridges, cable-stayed; Composite structures; Cantilevers; Model tests; Creep; China; Bridge design.

Introduction

Hongshan Bridge is a major bridge over the Liuyang River, located on the second north loop road in Changsha, Hunan Province, China. The bridge is a critical project for the construction of the loop road, lying within a leisure scene of Hongshan District. Its southern end links with the Sifangpeng interchange, and its northern end links with the great Laodaohe Bridge. Less than 2 km away to its east is the expressway to the airport; less than 3 km away to its north is the Park of the "Window of the World," and to its west lies Changsha University. Because of its geographical importance, the government of Changsha decided to build this bridge as a symbolic construction in Changsha and approved a harp-shaped single span cable-stayed bridge proposed and designed by Hunan University, in Changsha, Hunan Province, China.

[1]Professor, College of Civil Engineering, Hunan Univ., Changsha, Hunan, China.

[2]Lecturer, College of Civil Engineering, Hunan Univ., Changsha, Hunan, China.

[3]Associate Professor, College of Civil Engineering, Hunan Univ., Changsha, Hunan, China.

[4]Graduate Student, College of Civil Engineering, Hunan Univ., Changsha, Hunan, China.

[5]Professor, College of Civil Engineering, Hunan Univ., Changsha, Hunan, China.

[6]Lecturer, College of Civil Engineering, Hunan Univ., Changsha, Hunan, China.

Note. Discussion open until April 1, 2006. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on October 10, 2003; approved on October 26, 2004. This paper is part of the ***Journal of Bridge Engineering***, Vol. 10, No. 6, November 1, 2005. ©ASCE, ISSN 1084-0702/2005/6-658–665/$25.00.

Design Concept and Essence of Harp-Shaped Single Span Cable-Stayed Bridge

A perspective and an elevation of the harp-shaped single span cable-stayed bridge, Hongshan Bridge, are illustrated in Figs. 1 and 2, respectively.

Pylon and Foundation

The pylon is a prestressed concrete box section with a longitudinal dimension of 12 m and a transversal dimension of 8.2 m. The pylon is inclined at 58° from the horizontal with a height of 136.8 m (above the deck). The weight of the pylon is a crucial factor for the design of this harp-shaped cable-stayed bridge with an inclined pylon and without a back stay retention cable, because it has to balance the loads on the deck structure. The weight of the pylon should be configured in such a way so as to remain in full compression under the balanced load condition; i.e., the pylon should be in axial compression when the girder is loaded by all dead loads and half live loads, as shown in Fig. 3.

For parallel cable stays under the balanced load condition, i.e., where the weight of dead loads of the inclined pylon between the space of b will be balanced with the weight of all dead loads and half live loads of the main girder between the space of a, the geometrical and static balanced relationships between the pylon and the main girder can be calculated as

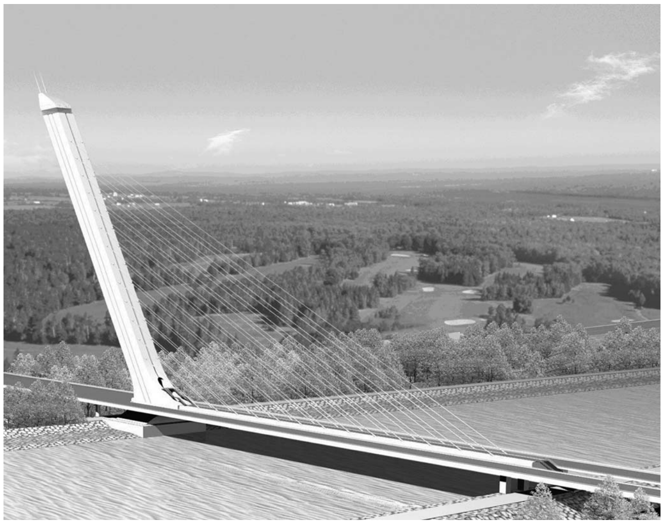

Fig. 1. Perspective view of Hongshan Bridge

$$
b = \frac{\sin\alpha}{\sin(\theta-\alpha)} \cdot a
$$

$$
W_T = \left(\frac{\tan\theta}{\tan\alpha} - 1\right)\left(W_d + \frac{W_L}{2}\right) = C \cdot \left(W_d + \frac{W_L}{2}\right) \tag{1}
$$

where a and b represent the distance between each two cable stays on the girder and the pylon, respectively; W_d and W_L=weight of the dead loads and live loads, respectively, between the space of a on the main girder; W_T=weight of dead loads of the inclined pylon between the space of b, and α and θ=horizontal angle of the cable stays and the inclined pylon, respectively.

In principle, the segment dead loads of the pylon, represented by W_T, should be based on Eq. (1). Accordingly, the value of C in Eq. (1) is 2.432, which will lead to the greater concrete volume of the pylon. Thus, the pylon is designed as a varied shape section with a thick wall at the bottom and a thin wall at the top, resulting in lowering the gravity center of the pylon and enhancing the load-carrying capacity of the pylon bottom. The resulting concrete volume of the pylon is approximately $6,700\ \mathrm{m}^3$.

Another characteristic of the bridge is that the walkway lies in the center of the deck between the two cable faces, and the walkway is 2 m above the traffic lanes, as illustrated in Fig. 4. A hole

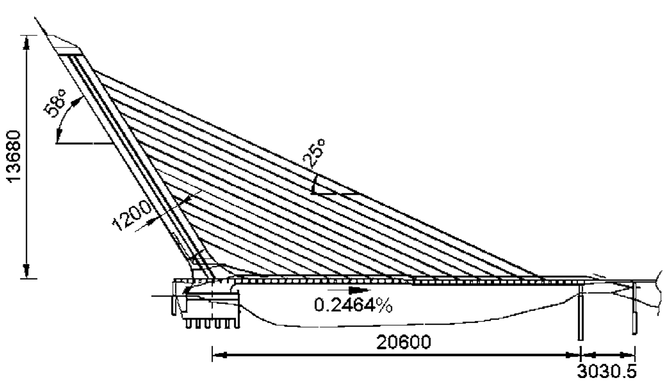

Fig. 2. Elevation of Hongshan Bridge (cm)

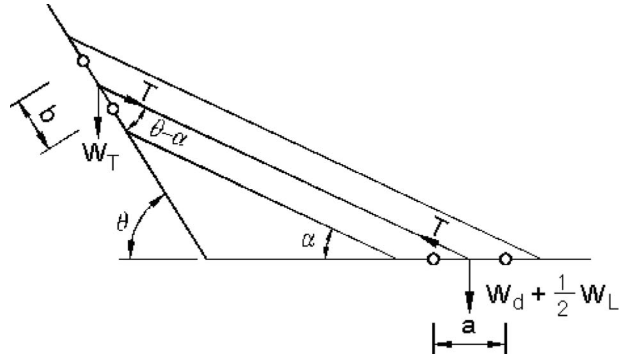

Fig. 3. Illustration for calculating weight of inclined pylon

about 3 m in width and 9 m in height is left at the bottom of the pylon for the convenience of pedestrians who cross the bridge. Also, a sightseeing elevator is offered in the pylon with an enclosed viewing platform on top of it.

Due to the good geological quality and bare bedrock at the bridge site, the type of pylon foundation adapts well to a spread footing with dimensions of 31 m (longitudinal)×30 m (transversal)×9 m (vertical).

Main Girder

The main girder is an orthotropic steel-concrete composite box girder, which is adopted on the basis of the following analysis and comparison.

Although the span of the bridge is not very long, its structural form is special, and the weight of the main girder is connected and balanced by the cable stays through optimal inclination of the pylon, which is 58° from the horizontal. In terms of the present form, the value of C in Eq. (1) is 2.432; that is to say, if the main girder is increased by 1 unit weight, the pylon must be correspondingly increased by 2.432 unit weight, and at the same time, the cost of the infrastructure and the size of the cable stays have to be increased accordingly. Therefore, it is unfit to use a concrete girder in such a harp-shaped cable-stayed bridge in comparison with other alternatives.

As compared with the aforementioned concrete girder alternative, if a steel structure is used for the main girder, the weight of the girder will lessen and the volume of the pylon will decrease consequently. In this case, the weight of the whole bridge and the cost of the infrastructure will be reduced. In terms of the geological conditions of the bridge site, the bedrock is bare at the foundation, the burying depth of the weak weathered rock layer is shallow, and the measured allowable bearing capacity of the rock layer is 1,200 kPa. Due to the advantage of geological quality at the bridge site, the cost of the spreading foundation is only 8 million RMB (1 million dollars), which is less than 10% of the whole budget of the bridge. In this case, the reduced cost of the spreading foundation attained by using a steel box main girder is less than the increased cost of the construction of the superstructure for a steel box girder, which will approach 15 million RMB (1.8 million dollars), and the cost of maintenance will accordingly be increased (Shao et al. 2001). Economy is one of the basic requirements in bridge design, so it is advisable to reject the aforementioned steel box girder.

As the cable stays lie only on one side of the pylon in a harp-shaped arrangement, there is comparatively less supporting stiffness presented by the stays. Therefore, the stress amplitude of the

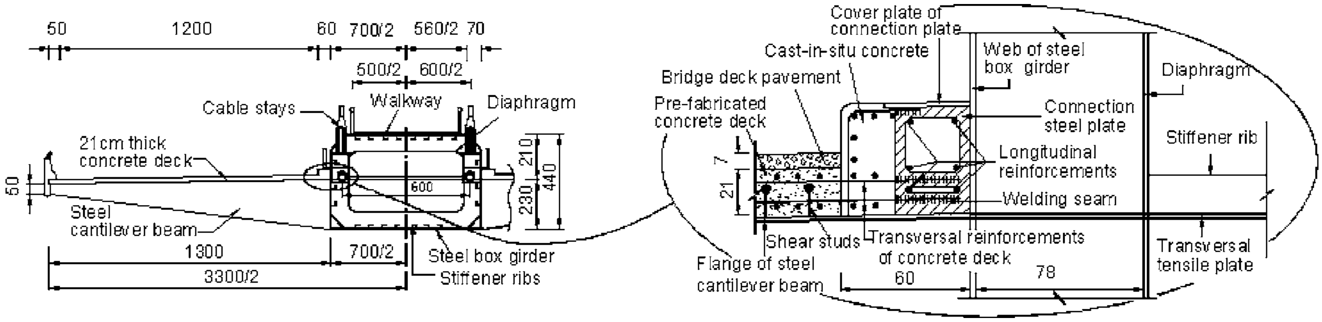

Fig. 4. Cross section of main girder [cm]

main girder is high under live loads. The concrete deck could not bear the high stress amplitude and would lead to cracking if a conventional composite girder were adopted in this type of bridge; thus, the special orthotropic steel-concrete composite box girder is selected for the main girder, and the concrete deck is placed near the neutral axis of the main girder, with the Alamillo Bridge (Pollalis 1999) as a reference. In terms of the combined action between the steel box girder and the concrete deck, the former undertakes moments, axial forces, and torsion, and the latter only undertakes horizontal axial forces offered by stays. In order to reduce the weight and the effect of creep, the prefabricated concrete deck is only 21 cm thick, but the ratio of reinforcement of the deck is increased accordingly.

The cross section of the main girder is shown in Fig. 4.

There is a closed rectangle orthotropic steel box girder with a height of 4.4 m and a width of 7.0 m at the center of the bridge, and the thickness of the box girder web and the top and bottom flange are all 28 mm. On each side of the steel box girder is a pair of 13-m-long cantilever steel beams every 4 m along the longitudinal axis. The reinforced concrete deck is installed on the cantilever beams, and they are connected together by shear studs, the diameter and the spacing of which are 19 and 160 mm, respectively. Also, there are longitudinal stiffener ribs on the wall of the box girder. Every 4 m along the longitudinal axis are diaphragms. On each side of the cantilever beam there are three traffic lanes.

In order to reduce out-of-plane bending deformation of the steel box girder under the tensile forces caused by cantilever beam webs and flanges, the diaphragms and transversal tensile plates are installed in the steel box girder where the box girder webs and the cantilever beams are connected. The reinforcements of the concrete deck and the steel box girder web are connected as an integer through the connection steel plate, as is illustrated in Fig. 4.

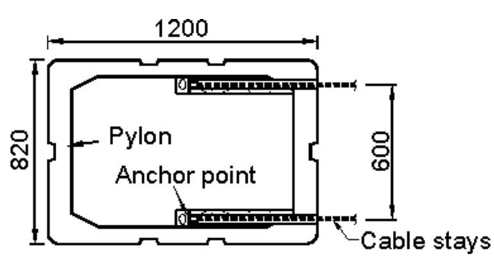

Fig. 5. Anchoring approach of cable stay on pylon (cm)

Cable Stays

There is a double plane of cables and 13 pairs of parallel cable stays with a horizontal angle of 25° in the bridge. The transversal spacing of the cable stays is 6 m, and the spacing along the main girder and the pylon is 12 and 9.3 m, respectively. The length of the cable stays lies between 65.9 and 289.8 m. Also, all of these are galvanized steel wires, 7 mm in diameter and with high strength and low relaxation, and the tensile standard strength is 1,670 MPa. The cable stays are prefabricated cables with two specifications, 223ϕ7 and 283ϕ7, and each specification is matched to the corresponding chilled cast anchor device.

The characteristics of a harp-shaped cable-stayed bridge are different from those of conventional bridges. Also, the anchoring approach of cable stays on the pylon is also different from that of a conventional cable-stayed bridge. As illustrated in Fig. 5, it is better that cables be anchored at the neutral axis of the pylon than other alternatives. If the anchorages are laid on the front or back wall of the pylon, there would be additional bending moments caused by cable forces on the cross section of the pylon, which would account for 30% of the total bending moments on the pylon. These would lead to a dangerous effect of the pylon under long-term loads. Therefore, it is important to put the anchor points on the neutral axis of the pylon to get a better distribution of the stay concentrated forces and to reduce bending moments on the pylon cross section.

The cable forces play an important role in keeping the pylon and the girder in good condition under loads. According to the characteristics of a harp-shaped single span cable-stayed bridge, the cable forces should first satisfy the requirements of the

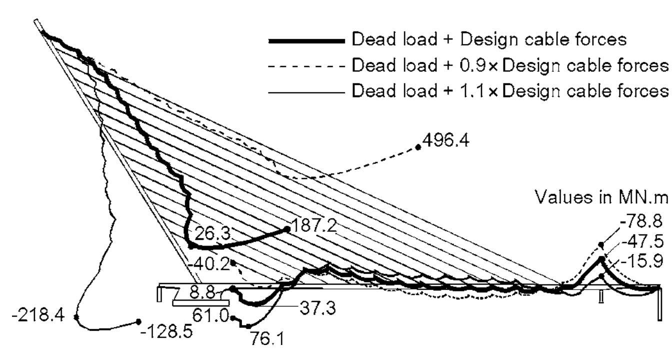

Fig. 6. Variation in bending moments due to ±10% variation in cable forces (MN.m)

characteristics of the pylon, say, let the positive and negative bending moments of the pylon be as small as possible in various load cases. When the cable forces are adjusted under the control of achieving satisfying internal forces of the pylon, the characteristics under loads of the girder are also in good condition, because the weight of the pylon and the girder are mutually matched.

For the harp-shaped single span cable-stayed bridge, the bending moments of the pylon and the girder will obviously vary due to variation of the cable forces (Casas 1994). Fig. 6 shows the effects of a 10% increase or reduction in design cable forces on the bending moments of the pylon and the girder. A reduction of only 10% causes a variation in the bending moment of the bottom part of the pylon from 187.2 to 496.4 MNm (more than 165%) and from −47.5 to −78.8 MNm (almost 64%) in the maximum moment of the girder. An even more dramatic result is the variation in moment in the pylon (from 26.3 to −218.4 MNm) when cable forces are 10% higher than the design forces. Although a great bending moment at the bottom part of the pylon is caused by a 10% increase or reduction in design cable forces, the pylon remains in compression. The tolerances of the cable forces in construction are limited to a 5% increase or reduction in design cable forces.

The maximum stress amplitude of the cable stays approximates 90 MPa; hence, the fatigue of the cable stays is not our major concern, for the following two reasons. First, the main girder is 4.4 m, which leads to its greater stiffness, while at the same time the main girder plays a major role in dispersing live loads. Second, the cable stays lie only on one side of the pylon, so there is no alternate positive and negative cable forces of the cable stays.

Scientific Research of Key Technical Issues

Study of Local Stability of Steel-Concrete Composite Box Girder under Combined Bending and Axial Loads

Three-Dimensional Finite-Element Results Considering Shrinkage and Creep of Concrete and Effect of Shear Lag on Deck

For steel concrete composite structures, the effects of concrete shrinkage and creep should be seriously considered. In common conditions, the effect of concrete shrinkage and creep will lead to a reduction in the concrete stress and an increase in the steel box girder stress. In the design of Hongshan Bridge, the following steps are adopted to reduce the effect of concrete shrinkage and creep. First, the storing time for the prefabricated deck is lengthened and the loading time is requested to be over 4 months. Second, the ratio of reinforcement in the deck is increased appropriately. Third, steel fiber concrete whose quality grade is C60 is used in the joints between decks.

At the same time, the effect of shear lag is another important factor that can not be ignored for the special cross section of the main girder. The extent of which the transversal, single-side, 13-m-long concrete deck takes part in the combined action between the steel box girder and the concrete deck relates directly to the safety and stress level of the steel box girder.

So as to understand the aforementioned two vital factors, a three-dimensional (3D) finite-element analysis of the main girder was made. In the analysis, we considered only the stress distribution patterns between the concrete deck and the steel box girder

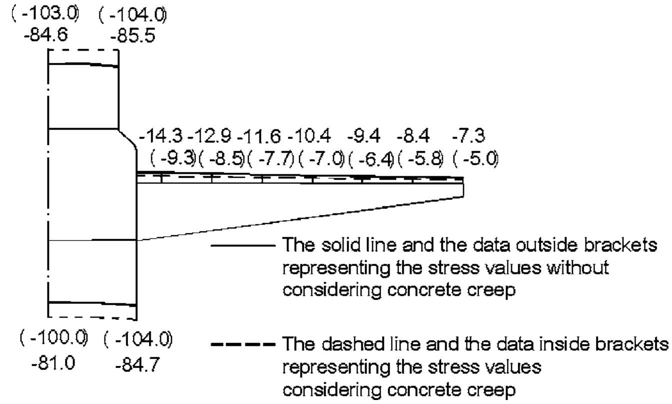

Fig. 7. Stress distribution patterns of steel box girder in axial forces (MPa)

under axial forces, because we always ignore the collaboration of the concrete deck when the main girder is subjected to bending loads.

Fig. 7 shows the stress distribution patterns under the horizontal component force of the maximum cable force along the cross section of the steel box girder, the position of which is near the bottom of the pylon. The results illustrate that, after summing up all the stresses, if concrete creep is not considered, the ratio of the axial forces burdened between the concrete deck and the steel box girder is 0.402 to 0.598, while the ratio is 0.266 to 0.734 if concrete creep is considered. We can conclude from Fig. 7 that the effect of shear lag of the concrete deck near the bottom of the pylon is obvious.

Analysis and Tests of Local Stability of Steel Box Girder

For the local stability of the steel orthotropic box girder, there are no prescriptions and commentaries in the highway specifications and other structural specifications in China. We therefore made calculations and analysis for the steel box girder of the Hongshan Bridge by adopting the *Specifications and Commentaries for Superstructure Design of Honshu-Shikoku Bridge* (Honshu-Shikoku 1989) (hereafter referred to as the Honshu-Shikoku specifications). We also investigated the ultimate bearing capacity of the steel box girder of Hongshan Bridge with the help of two scale model tests.

Fig. 8. Scaled full model tests

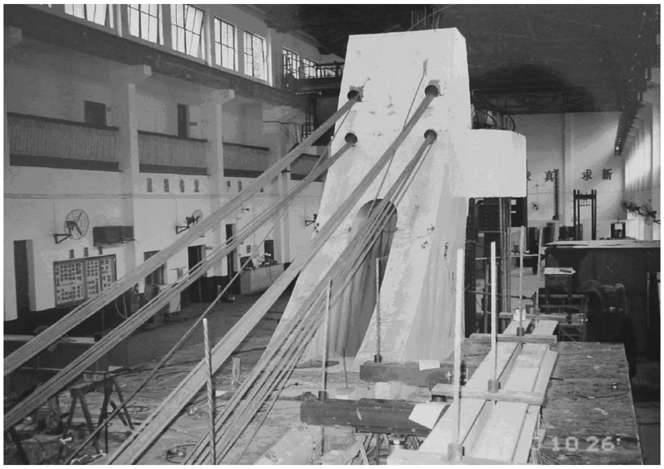

Fig. 9. Scaled section model tests

Introduction of Tests

1. Scaled full model tests of the Hongshan Bridge on a scale of 1:30 (Fig. 8).
   The aim of these model tests is to investigate the static and dynamic characteristics of the bridge in the construction and operation stages, to verify the correctness of the theory results, and to discover and compensate for the uncertain and weak parts of the structure. For the actual bridge, the thickness of the wall of steel box girder is 28 mm, the width and height are 7.0 and 4.0 m, respectively, and the material is 16Mnq steel in the preliminary design. According to the similitude theory, for the model bridge, the thickness of steel box girder wall is thus 0.9 mm, the width and height of the box girder are 233 and 147 mm, respectively, and the material is A3 steel. In addition, the 21-cm-thick concrete deck is simulated as an alloyed aluminum deck, 3.5 mm in thickness. As cable forces play a major role in this type of bridge, how to measure the cable forces in the model tests becomes very important. The stays in the model are simulated as steel wires with high strength, and the area of the steel wires is 1/900 of that of the cable stays in the actual bridge. The cable forces are measured by connecting a sensor at the girder end of each cable stay.
2. Scaled section model tests of the Hongshan Bridge on a scale of 1:6 (Fig. 9).
   The purpose of the scaled section model tests is to investigate the stress patterns of the main girder and the pylon near the bottom of the pylon and to master the capability of overloading, the modes of failure, and the capability of the safety reserve of the bridge. According to the similitude theory, for the model bridge, the thickness of steel box girder wall of the model bridge is 5 mm, and the width and height of the box girder are 1,100 and 720 mm, respectively. After considering the effect of creep, the 21-cm-thick concrete deck of the bridge is simulated as a 4 mm steel deck. Also, to account for the difficulty in selecting the material of the model bridge, the steel material is chosen as A3 steel. The steel box girder material of the actual bridge is as described previously.

Table 2. Design Safety Coefficient of Steel Box Girder of Hongshan Bridge

Material of steel	Thickness of steel (mm)	Number of stiffener ribs[a]	Allowable stress (MPa)	Combined stress (MPa)	Safety coefficient
14MnNbq	32	8	223	166	2.7
16Mnq	28	8	187	149	2.5

Note: Material 14MnNbq is used in position that lies from bottom part of pylon to first cable stay, while Material 16Mnq is used on other part of steel box girder.

[a]Number of stiffener ribs represents number of stiffener ribs on top or bottom flange of steel box girder.

Analysis and Comparison

In terms of the Honshu-Shikoku (1989) specifications, considering the local stability, we calculate in detail the allowable compression stress of the two model tests. Also, we compare the calculation results with the tested failing stress (the two model tests are in destruction due to the steel box girder being bereft of local stability) so as to get the measured safety coefficient of the Honshu-Shikoku (1989) specifications. The results are listed in Table 1.

The measured results of the model tests illustrate that the safety coefficient of the allowable stress prescribed by the Honshu-Shikoku (1989) specifications is about 2.0.

Emphasis will be put on the following three important factors, considering the unusual structural style of the Hongshan Bridge: (1) The effect of creep is bound to lead to the increase in steel box girder stress, and the actual results may differ from the theoretical ones. (2) The concrete deck and the steel box girder undertake together the horizontal forces caused by cable stays. Although the effect of shear lag was calculated by 3D finite-element models and verified by model tests, there may be differences in the actual bridge, which will cause the steel box girder to undertake greater compression stress. (3) The possible difference of the cable forces between the actual and the theoretical may bring on the obvious increase of the stress in the steel box girder.

After the two model tests, in order to ensure the safety of the Hongshan Bridge, the safety coefficient of the local stability of the steel box girder was increased in the final design of the bridge. In the actual bridge, the material of the steel box girder was changed from 16Mnq steel to 14MnNbq steel from the bottom of the pylon to the position of the first stay to ensure its local stability, while the material used in the other part of the steel box girder was still 16Mnq steel. The numbers of stiffener ribs on the top and bottom flanges of the steel box girder were also changed in the actual design. Detailed results are given in Table 2.

Table 1. Verifying Safety Coefficient of Honshu-Shikoku (1989) Specifications by Model Tests

Scale of model	Material of steel box girder	Thickness of steel (mm)	Number of stiffener ribs[a]	Allowable stress	Failing stress	Safety coefficient
1:30	A3	0.9	6	133	260	1.95
1:6	A3	5.0	6	100	200	2.00

[a]Number of stiffener ribs represents the number of stiffener ribs on the top or the bottom flange of the steel box girder.

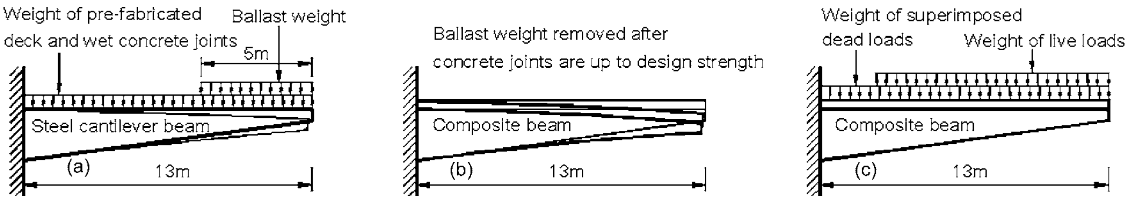

Fig. 10. Schematic drawing of characteristics under loads of each performance: (a) Performance I; (b) Performance II; and (c) Performance III

Analysis of 13-Meter-Long Steel-Concrete Composite Cantilever Beam under Loads

Hongshan Bridge is designed with six lanes, three lanes on each 13-m-long cantilever beam. To ensure its safety, it was necessary to study thoroughly the comprehensive characteristics under loads of the 13-m-long cantilever beam.

In a steel-concrete composite beam, the best status is steel in tension while the concrete is in compression. The composite cantilever beam of the Hongshan Bridge, unfortunately, is contrary to this. The prebending prestressing method had to be used during construction to prevent the concrete deck from cracking. The stress status was analyzed during the construction and operation stages.

Illustrated in Fig. 10 and as follows are the loading conditions of each performance:

1. Performance I: Put the prefabricated concrete deck on the steel cantilever beams and pre-add ballast weight equal to 400 kN on every cantilever beam, then cast in situ the wet concrete joints;
2. Performance II: After the wet concrete joints are up to the design strength, unload the ballast weight; and
3. Performance III: Load the superimposed dead loads and the live loads in the operation stage.

In the aforementioned three performances, the stress in the prebending stage for Performance I is calculated as the steel cross section, while Performances II and III are calculated as the steel-concrete composite cross section. In Performance II, the concrete deck of the steel-concrete composite beam is in compression, and the precompression stress is caused by the upward deflection of the steel cantilever beam as a result of unloading the ballast weight. In Performance III, the concrete deck is in tension under the action of the weight of the superimposed dead loads and live loads.

From Table 3, it can be found that the precompression stress of the concrete deck due to unloading the ballast weight in Performance II (3.24 MPa) is more than the tensile stress due to the superimposed dead loads and live loads in Performance III (−2.04 MPa). Thus, the concrete deck will be in compression during the operation stage.

Table 3. Stress under Each Performance (Compression +; Tension −)

Performance	I	II	III	Total
Stress of top flange of concrete deck (MPa)	0.0	3.24	−2.04	1.20
Stress of bottom flange of steel beam (MPa)	113.3	−52.80	33.40	93.90

Safety Analysis at Critical Section Caused by Overload on Pylon and Breaking of Cable Stays

The harp-shaped single span cable-stayed bridge is an unusual cable-stayed bridge, and the weight of the main girder is balanced by the inclination of the pylon. Therefore, variation of the weight of the pylon and the girder will break the balanced relationship between them and lead to disadvantageous effects to the bridge. In the balanced condition, the moment in the pylon is small, but the calculation results illustrate that if the pylon is overloaded by 10%, i.e., if the weight of the pylon is 10% higher than the design weight of the pylon, the moment at the bottom of the pylon will be increased by 400%. On the other hand, as there are many accidents caused by broken cable stays in conventional cable-stayed bridges, it is necessary to study and test two dangerous situations in the 1:30 scaled full model tests for the safety of the bridge. First, if the pylon is overloaded by 10%, will the pylon be damaged and crack, or not? Second, if one of the cable stays is broken abruptly during the operation stage, will the main girder remain safe?

Pylon Overloaded by 10%

These tests are to investigate the characteristics under loads of the bridge under the conditions that the pylon is overloaded by 10% (i.e., where 10% is added to the weight of the pylon) and that there are no live loads on the deck. The overload of the pylon is applied by adding additional bob-weights to the pylon. In the design of actual bridges, the weights of the pylon and the girder are reciprocally matching, and they are in good condition, as mentioned previously. However, there are some potential factors that may cause the weight of the pylon of the actual bridge to exceed the design weight of the pylon—for example, the site changing or deformation of the formwork in the construction, the density of the concrete, the ratio of reinforcement in concrete, etc. These tests are intended to verify the safety of the structure in this disadvantageous condition; i.e., the weight of the pylon is overloaded by 10% and the weight of the girder remains the design weight.

The process of the tests is as follows:

1. Bob-weight the model bridge according to the similitude theory, then adjust the cable forces repeatedly to get the operation stage;
2. Overload the pylon; and

Table 4. Deformation of Top of Pylon

Value	Longitudinal (mm)	Vertical (mm)
Measured value	(←)2.63	(↓)1.65
Calculated value	(←)2.75	(↓)1.75

Table 5. Stress Distribution Patterns of Bottom Section of Pylon (Compression +; Tension −)

	Top flange (MPa)			Bottom flange (MPa)		
Measured point	1	2	3	4	5	6
Measured value	10.22	10.11	10.28	2.01	1.93	2.18
Calculated value	10.03	10.03	10.03	2.19	2.19	2.19

3. Measure the cable forces, the deformations, and the stresses. The deformations of the top of the pylon in these conditions are listed in Table 4. The deformation of the central part of the main girder, i.e., the position of the ninth stay, is −1.32 mm(↑), correspondingly, the deformation of the actual bridge is −4.0 cm(↑).

After the pylon is overloaded by 10%, the stress distribution patterns of the top and bottom flange at the bottom section of the pylon are as illustrated in Table 5.

Thus, we can arrive at the following conclusions: (1) If the pylon is overloaded by 10%, the positive moments of the pylon will be increased greatly, especially in the critical section at the bottom of the pylon, which will be increased by 4.2 times, and the axial forces will be increased by 10%. This is the main load case to control the strength and cracking of the bottom section of the pylon. (2) The average stresses of the bottom and top flanges at the bottom section of the pylon are 10.20 and 2.04 MPa, respectively, in this load case, and both are compression stress. (3) Because precompression stress of about 2.0 MPa has been exerted on the pylon of the actual bridge, the reserve of the compression stress will be great enough. Therefore, even if the pylon is overloaded by 10%, although the internal forces of the pylon will be increased obviously, the security of the pylon will be sufficient.

Deck is Covered by Eccentric Full Loads and One of the Cable Stays is Broken

The process of these tests is as follows:

1. Bob-weight the model bridge according to the similitude theory, then adjust the cable forces repeatedly to get the operation stage;
2. Add eccentric full loads (eccentric full loads means that loads are added only on one side of the deck and the traffic lanes at this side are full of loads) at one side of the main girder;
3. Slacken the ninth stay gradually, which locates at the side of the eccentric loads; and
4. Measure the cable forces, the deformation, and the stress.

Illustrated in Table 6 is the deformation of the bridge at the position where the heavy loading wagon is located.

Table 6. Vertical Deformation of Middle Span of Main Girder

Position	Edge of loading deck (mm)	Center of deck (mm)	Edge of unloading deck (mm)	Deformation difference[a] (mm)	Torsion angle (rad)
Measured value	8.16	6.01	3.73	4.43	0.00443
Calculated value	(↓)8.41	(↓)5.88	(↓)3.40	5.01	0.00501

[a]Deformation difference represents deformation difference between two-side roots of cantilever beams and is equal to values in Column 2 minus values in Column 4.

Table 7. Stress Distribution Patterns of Steel Box Girder (Compression +; Tension −)

	Top flange (MPa)			Bottom flange (MPa)		
Measured point	1	2	3	4	5	6
B-B						
Measured value	121.6	114.9	126.7	97.6	94.9	96.7
Calculated value	134.3	132.3	129.1	93.6	91.3	89.5
F-F						
Measured value	105.3	99.8	108.9	−42.7	−36.7	−38.9
Calculated value	107.6	105.3	103.1	−28.7	−21.2	−15.3

Note: Allowable stress of steel box girder is 200 MPa. Section B-B is girder section which locates at bottom part of pylon. Section F-F is girder section which locates at girder end of ninth stay.

From the deformation results, we know that the measured deformation values are almost all less than the theoretical ones. When the deck is covered by eccentric full loads and one of the cable stays is broken, the maximum deformation of the bridge at the position where the heavy loading wagon is located is only 8.16 mm, and correspondingly at 24.5 mm on the actual bridge. All of the experimental results illustrate that the rigidity of the bridge is enough.

The stress distribution patterns of the two critical sections of steel box girder are shown in Table 7.

The measured stress may be different from the theoretical ones to some extent, but compared with the maximum combined stress under the condition that the deck bears eccentric full loads while one of the cable stays is broken, the stress level of the steel box girder does not reach the controlling value, which is 187 MPa (see Table 2). Thus, it is unnecessary to worry about this matter.

Study on Characteristics of Wind Resistance and Wind Tunnel Testing

For wind resistance safety of the Hongshan Bridge during the construction and operation stages, the characteristics of wind resistance of the bridge and wind tunnel testing of the model bridge were studied at the Southwest Communication University in China. This study included: (1) wind tunnel testing of scaled section models of the main girder; (2) a six-component thrust coefficient measurement test on a scaled rigid model of the pylon; (3) a wind tunnel testing of the scaled full aeroelastic model of the bridge on a scale of 1:120; and (4) analysis of the wind-induced response and comparison with the test results.

The main conclusions for the Hongshan Bridge are as follows: (1) The critical wind velocity of flutter of the bridge is far greater than the checked wind velocity of flutter whether in the construction or service stage, so the stability of wind resistance of the Hongshan Bridge is sufficient. (2) The analysis of the buffeting response showed that the bottom section of the pylon is the most dangerous section. During the construction and operation stages, the transversal moments at the bottom section are up to 348,250 and 330,837 kN.m, respectively. Simultaneously there is high axial force in the pylon; however, there is no tensile stress at the bottom of the pylon under any combined loads. (3) For the construction and operation stages, the vortex-shedding induced vibration will not appear under the designed wind velocity, which is

28.3 m/s. Therefore, it is unnecessary to consider the disadvantageous influence in fatigue of the structure and ease of driving caused by vortex-shedding induced vibration.

Conclusion

The design essence and main conclusions of the experimental study of a harp-shaped single span cable-stayed bridge, Changsha Hongshan Bridge, are presented in the paper. The principle for determining the weight of the pylon is appropriate; i.e., the pylon should be in a status of axial compression when the girder is loaded by all dead loads and half live loads. Because the girder needs to bear the alternate action of tensile and compressive stress, the concrete deck is installed near the neutral axis of the steel box girder. Thus, the concrete deck only undertakes horizontal component forces caused by cable stays and hardly undertakes moments, avoiding cracking of the concrete deck caused by longitudinal bending moments. In order to prevent the cracking of the long steel-concrete composite cantilever beam under loads, the prebending method is adopted to put prestressing forces on concrete deck. In addition, the two model tests verify the safety of the local stability of steel box girder, and the safety of the bridge under some other dangerous conditions. The careful design and experimental studies insure the safety of the harp-shaped single span cable-stayed bridge.

Acknowledgments

This program is funded by the Natural Science Foundation of Hunan Province, China (No. 02JJY3058) and the Scientific Foundation of the Construction Bureau of Changsha, Hunan, China.

Notation

The following symbols are used in this paper:

A3 = type of steel material in China, with yield strength of 235 MPa;
a = distance between each two cable stays on girder;
b = the distance between each two cable stays on pylon;
C = coefficient to determine weight of pylon;
C60 = grade of concrete, where design compression strength of 150 mm cube is 60 MPa;
W_d = weight of dead loads between space of a on main girder;
W_L = weight of live loads between space of a on main girder;
W_T = weight of dead loads between space of b on inclined pylon;
α = horizontal angle of cable stays;
θ = horizontal angle of inclined pylon;
14MnNbq = type of steel material in China, with yield strength of 380 MPa; and
16Mnq = type of steel material in China, with yield strength of 335 MPa.

References

Casas, J. R. (1994). "A combined method for measuring cable forces: The cable-stayed Alamillo Bridge, Spain." *Struct. Eng. Int. (IABSE, Zurich, Switzerland)*, 4(4), 235–240.

Honshu-Shikoku Bridge Authority. (1989). Specifications and commentaries for superstructure design of Honshu-Shikoku Bridge, Tokyo.

Pollalis, S. N. (1999). *What is bridge? The making of Calatrava's bridge in Seville*, MIT, Cambridge, Mass.

Shao, X. D., Li, L. F., Zhao, H., and Peng, W. H. (2001). "The design of harp style cable-stayed bridge—Changsha Hongshan Bridge." *J. Hunan Univ. (Sci. J.)*, 28(4), 88–93.