Construction and control technology of the main bridge superstructure of Sutong Bridge

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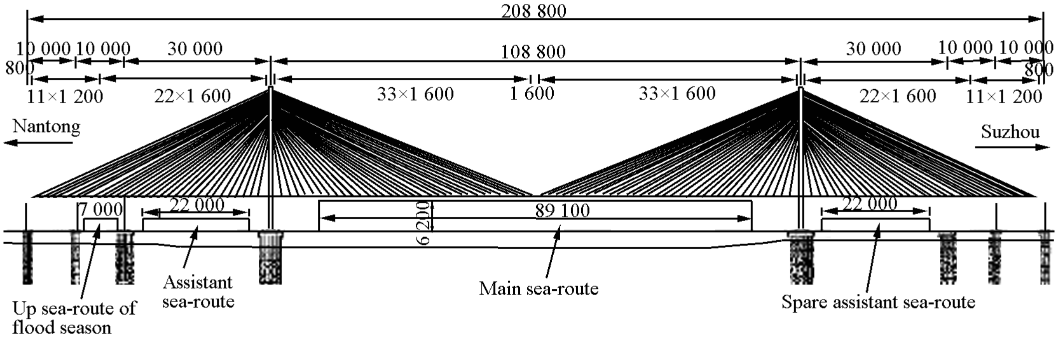
Abstract: The Sutong Yangtze River Bridge (short as Sutong Bridge) is now the largest span cable-stayed bridge in the world. The construction of the superstructure of the middle bridge covered several stages including erection of the big block girders for the side span, assistant span and tower area, erection of standard girders and closure of the middle span. The big block girders were hoisted by a floating crane, and the standard girders were hoisted by a double crane system on the deck. The pushing assistant method was adopted for the middle span closure construction. Furthermore, key technologies and innovative methods used in the processes of girder erection and cable assemblage in all stages were expatiated systematically. An all-stage self-adaptive geometry control method was used in the construction process. By accurately controlling the unstressed dimensions and shape of all structural components in each step, and realization that the control system and the controlled system adapt to each other, the goal was to make control of the final line shape and inner force of the bridge structure achievable. Two solutions, including GPS based and total station based dynamic geometry monitoring systems, were used to resolve the measure problem under the wide - range of wind-induced vibrations in the long cantilever state. Finally, research on the wind-induced vibration of the superstructure during the construction period was executed. Buffeting response analysis to the longest single and double cantilever states were carried out. The analysis and evaluation of wind resistance safety of the main girders under the longest single cantilever state was made, and corresponding wind resistance measures were suggested. The as-built geometric error and cable force error were controlled in a required design range, and this whole technological achievement can be a benchmark for construction of other large span cable-stayed bridges in the future.

Key words: Sutong Bridge; superstructure; construction method; construction control; geometry control; wind-resistant measure

1 Introduction

The Sutong Yangtze River Bridge (short as Sutong Bridge) is located between Suzhou and Nantong in Jiangsu Province, China. It is the longest cable-stayed bridge in the world, and is illustrated in Fig. 1. The span layout of main bridge is (100 + 100 + 300 + 1 088 + 300 + 100 + 100) m. The deck, of width 31.0 m, will carry 6 lanes of traffic, with a longitudinal slope of 1.5 % and a cross slope of 2 %. There are 136 pairs of cables in all, and the longest cables are up to 577 m long, with a weight of 59 t.



Unit: cm

Fig. 1 Main bridge schematic diagram of Sutong Bridge

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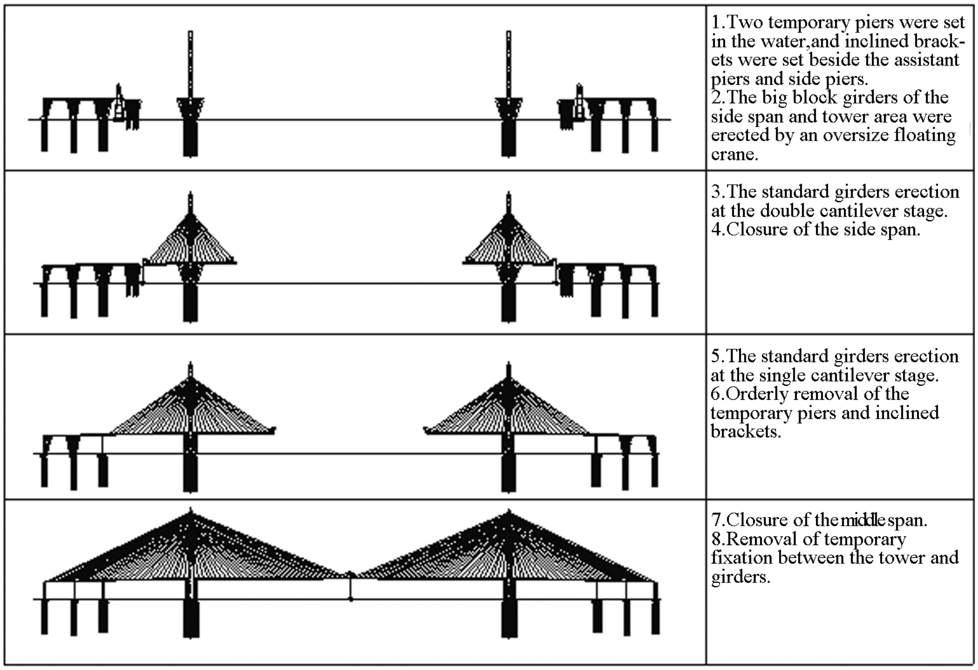
2 Construction technology

2.1 General construction program

After the cable tower and assistant piers at the side span were finished, erection of steel box girders could be separated into 5 parts, including girders erection of assistant span and side spans, girders erection in the tower area, standard girders erection, closure of the side span and closure of the middle span.

The general construction process of the superstructure was as follows: big block girders erection of the assistant span and side span→ girders erection of the tower area, installation of temporary girders fixation to the tower and assembly of cranes at the deck→ standard girders erection under the double cantilever state and cable erection→ closure of side span→ standard girders erection under the single cantilever state and cable erection in both sides and middle span→ closure of the middle span. The procedure is illustrated in Fig. 2.

To reduce the difficulty of building brackets in deep water and the risk of the brackets being damaged by ship impacts during the construction period, inclined brackets were set up alongside the transition piers and assistant piers to support the girders. At the same time, large-sized temporary piers of height about 100 m were built in deep water of the side spans to restrict the length of the longest double cantilevers to 156.8 m. The structural safety under the double cantilevers state could then increase, and the side span could be closed and moved into a single cantilever state construction, ahead of time.


1.Two temporary piers were set in the water,and inclined brackets were set beside the assistant piers and side piers.
2.The big block girders of the side span and tower area were erected by an oversize floating crane.

3.The standard girders erection at the double cantilever stage.
4.Closure of the side span.

5.The standard girders erection at the single cantilever stage.
6.Orderly removal of the temporary piers and inclined brackets.

7.Closure of the middle span.
8.Removal of temporary fixation between the tower and girders.

Fig. 2 Schematic diagram of construction process

2.2 Erection of steel box girder

2.2.1 Construction of big block girders and temporary fixation of girders to the tower

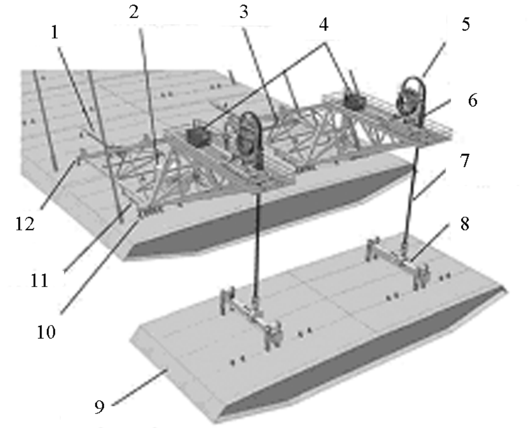
Unilateral side spans and assistant spans, in all, were up to 343 m long and were separated into 9 big block girders, the largest big block being up to 60 m in length and 1 200 t in weight. The big block girders were lifted and laid-aside on the temporary brackets and piers. Then connection of girders and system transfer was carried out after precise adjustment of the position of the girders.

The temporary elastic fixation system, composed of parallel wire strands and steel bearings, was used for temporary connection of the girders to the tower. It afforded elastic control to the main girder during the construction period and limited vertical, longitudinal, cross direction displacement and rotation of main girder, to ensure structural safety and to provide definite boundary conditions for the construction control analysis and closure maneuverability for the middle span.

2.2.2 Erection of standard girders

Standard girders were erected by cranes on the deck, and the standard procedure was as follows: girders lifting →matching and welding of girders →first tension to cable, loosen crane hook →cranes move forward and provide second tension to cable.

Since the standard girders were wider and heavier than the ordinary ones, if the conventional single crane on the deck was adopted, reaction of the support points would be high and might cause problems for the local strength and stability of the steel box girder. Since the supporting conditions were different, there could be a great difference between the local deformation of the erected girders and the erecting girders, which would make matching of the girders more difficult, and would result in greater residual deformation and stress. Accordingly, an erection program with separated double cranes on the deck was adopted (Fig. 3).



1—Support beam and hydraulic pressure moving system of crane;
2—DL-P40 computer control room;
3—Data cable of crane;
4—DL-L125/1/E hydraulic power unit;
5—DL-S290 modular of steel strand jack;
6—DL-S290 steel strand jack (safe work load: 290 t);
7—DL-S290 steel strand (safe work load: 290 t);
8—Carrying beam with hydraulic adjusting system for longitude slope of deck;
9—Main girder segment, weight up to 450 t;
10—4 support of cranes with hydraulic loads balance system;
11—2 support beams for movement forward;
12—2 back anchor connections

Fig. 3 Schematic diagram of double crane system on deck

Installation analysis indicated that the compression stress of the bottom plate of the main girder, around 5 segments (about 80 m) away from the river-side end of cantilever, was high under the action of the construction loads and the weight of the erecting girders during standard girders erection at the long cantilever construction stage. Over-tension of the cables was carried out to reduce compression stress in this area. When the length of the single cantilever increased to 400 m, erection of the No. 25 girder segment started, and the cable tension process was changed to tension 3 times compared to the usual two times process. Over-tension of the cable was taken at the second tension, and third tension of the over-tensioned cable was carried out after erection of the subsequent two girder segments. The pull-out lengths at the cable ends were adjusted to the final state.

2.2.3 Closure of middle span

The design base temperature was 20 ℃, and no disadvantageous influence on the permanent structure would be caused if the closure of the middle span was carried out at this temperature. In practical construction, since the largest longitudinal length of single cantilever girder expanded to 540.8 m, and the environmental temperature was influenced by the closure method and natural conditions, the environmental and structural temperature would be different from the base temperature. A push assistant closure program was suggested.

The key points of the program included: adjust the girders at both sides of the closure gap → install stiff frames → partially loosen the vertical cable of the temporary girders fixation to the tower and pull the two girders at both sides of the closure gap towards the bank side (change the length of the closure gap) → use cranes on the deck to lift the closure segment (length at the base temperature) → pull the two girders at both sides of the closure gap back by stretching when the temperature was relatively steady (non-sunlight) → synchronously weld the two joint seams and remove the temporary girders fixation to the tower.

Closure operation was carried in selected nighttime when the temperature was steady. The required distance to be pushed was less, because the change range of the environmental temperature was narrower. Using jacks to tension the longitudinal cables and to loosen the vertical cable, the jointed girders could be pushed. The distance to be pushed could be determined based on the closure temperature. Since the length of closure segment did not change in this method, the influence on stress and line shape of the as-built structure was fairly minor.

Fig. 4 Girder erection for middle span closure

2.3 Erection of cables

For cables No. 1 to No. 20, standard mature domestic technology for erection could be adopted. For cables No. 21 to No. 34, the length of these ultra-long cables exceeded those of other cable-stayed bridges in the world, and there was not any experience for reference.

Based on the characteristic of the ultra-long cables and the structural condition of the tower and girders, a method with flexibility and stiffness combined, 3 stages used and tension at the girder end was suggested. The main process was as follows: cable reel integrally lifted by a sling truss on the deck → unfold the cable on the deck using a lifting drum →fix the tower end of cables →drag the anchorage of the cable at the girders end using a lifting drum, continuous jack, tension jack and connections such as steel wire rope, steel strands, tension bars → tension from the girders end and adjust from the tower end.

An assembly dragging system set-up on the crane on the deck was used to assist in dragging the cables at the girders end. The temporary loads at the end of cantilever were reduced, and the work space needed was much less.

3 Construction control and monitoring technology

3.1 General control method

The bridge location was in an open river, and the meteorological conditions were complex. The structure was long and flexible, and was composed of a number of components. There were many uncertain factors in construction control, so the work was challenging. Thus, an all-step self-adaptive geometry control method (ASGC method) was developed which was different from the conventional construction control methods.

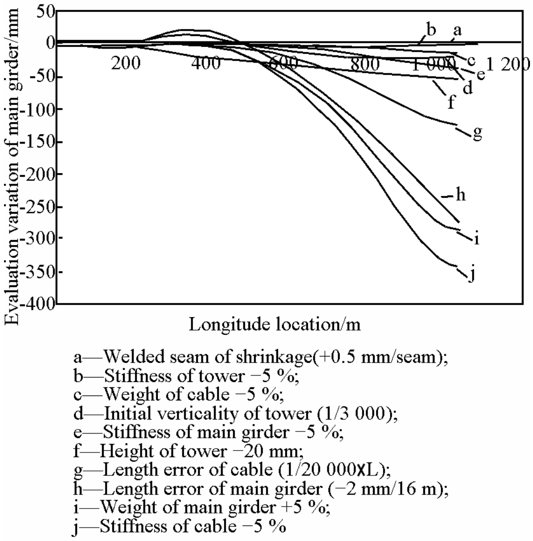
By controlling the unstressed dimensions and the shape of structural components accurately in all construction steps, the control system and the controlled system could match each other, and realizing control the final line shape and inner force. Construction control was extended to the fabrication and erection stages of the components. Dimensional errors in the structure could be controlled effectively, starting from the "headstream". And the distribution rule for errors of structural parameters could be found, and thus could be a reliable data basis for error adjusting at the erection site. After identification of the structural parameters and error evaluation of the erected structure, correction to the unstressed dimensional errors of the unfabricated segments was carried out.

The control system was composed of 6 modules, including a control module for key components fabrication, an evaluation module for the structural state of key cases, a module for model correction and parameter identification, a prediction module for subsequent construction state, a decision module for construction control, and a module for automatic sampling of data for geometry control and the data base.

3.2 Execution of construction control

Execution of construction control was divided into 3 stages including the planning stage, the fabrication stage and the erection stage.

At the planning stage, unstressed dimensions and line shapes could be obtained by a 3-D FEM (finite element model) model and analysis for the whole bridge structure, such as the coordinates of the anchor points of the tower at the self-erected state, the unstressed length (USL) and line shape of the main girder, and the fabrication length of the cables. Based on this analysis, the erection position of each girder segment, and the target line shapes of the erection could be accurately determined. Parameter sensitivity analysis on the as-built state was carried out. The 5 main control parameters included the fabrication length of the girders, the weight of the girders, the elastic modulus of the cables, the USL of the cables and the height of the anchor points of the tower. The parameter sensitivity analysis results for the as-built line shape of the main girder are indicated in Fig. 5. The line shape of the main girder after the second stretching of No. 34 cable is shown in Fig. 6.



a—Welded seam of shrinkage(+0.5 mm/seam);
b—Stiffness of tower −5 %;
c—Weight of cable −5 %;
d—Initial verticality of tower (1/3 000);
e—Stiffness of main girder −5 %;
f—Height of tower −20 mm;
g—Length error of cable (1/20 000×L);
h—Length error of main girder (−2 mm/16 m);
i—Weight of main girder +5 %;
j—Stiffness of cable −5 %

Fig. 5 Sensibility analysis of parameters

At the fabrication stage, precision control was used in the fabrication of components, including fabrication control of the tower, match fabrication of the girders and

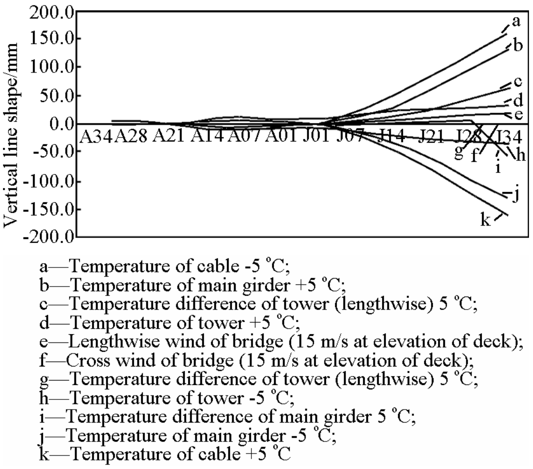


Fig. 6 Sensibility analysis of environmental factors after the second stretching of No. 34 cable

accurate fabrication of the cables. A long line match method (LLM) was adopted to match and fabricate the steel box girders, and the fabrication error was mostly reduced. The girders were fabricated in several batches, and the accumulative error of the preceding batch of girders was considered in the fabrication line shape of later batch of girders, so that the error could be controlled and adjusted. Each girder segment was weighed. Elastic modulus test and USL survey of each cable were undertaken during fabrication, and weighing of the cables was carried out in each batch. Error statistics for all components were determined, and an error data base was built up. The fabrication stage error, including geometric dimensional error and the material characteristic error were introduced into the analysis model, and the model parameters were updated. A new set of USL values of the cables and target line shapes for construction could be ascertained.

At the erection stage, a principle where the local line shape is controlled by the fabrication line shape, and the main line shape is controlled by the USL of the cables, was suggested. The girder segments were matched and connected using high precision butt devices and adjusting equipment to ensure that the actual constructional unstressed line shape of the steel box girder was in accord with the theoretical one. The structural geometric configuration including the line shape of the towers and the main girder, the USL of the girders etc and structural stress were investigated by advanced monitoring means. Environmental factors, such as temperature, wind etc, were surveyed synchronously and corrected accordingly. Corrected line shapes and theoretical line shapes were compared with each other, and the error was evaluated to determine whether it's within a controllable range or not. If yes, erection of the next girder segment can continue, if no, after error factor analysis, adjustment to the analysis model would be carried out, and adjustment to the USL of the later cables were calculated in order to reach the control aim.

3.3 Construction monitoring

Since the wind-induced vibration of the main girder was evident during the long cantilever stage, two dynamic geometric monitoring systems were adopted to investigate the real and exact positions of the main girder. Data from the total station was dominant in the normal situation, and the GPS data was used for checking. Under bad weather conditions (turbid, blowy), the GPS data could be supplied to the total station test data.

The GPS based dynamic monitoring system consists of a reference station, monitoring stations, a commutation system, and a control center. The monitoring stations automatically compute their antenna positions in real-time, according to the received information, and send them to the control center. The control center management software analyzes the results and displays them on the computer screen immediately. All the raw data and results are saved to the local database so that one can retrieve them for further analysis. The sampling frequency of this system is up to 20 Hz and can satisfy the dynamic monitoring requirement.

The total station based dynamic monitoring system works in the following two modes: one is the fix point tracking, in which each point is rapidly tracked continuously for a specified time interval, in real-time and automatically. It measures the 3-D coordinates at a high speed (up to 3 Hz), and this mode is suitable for deformation monitoring in dynamic environments. The second mode is specified as period scanning. The total station scans the targets one after another automatically. Considering the vibration of the steel box girder, it measures each point for at least 4 cycles within 2 min.

Physical monitoring includes temperature monitoring, cable force monitoring and stress monitoring. The temperature of the tower and steel box girder was surveyed. The stress in the tower and main girder was surveyed by strain meter. Three methods including a rope meter, numerical reading of a tension jack and the frequency method were used for cable force monitoring, and could be used for cross checks.

4 Wind-resistance during construction period

4.1 Wind tunnel test

Wind tunnel tests of segmented models and an aeroelastic model for whole bridge were carried out with the aim of investigating wind-resistance safety during the construction period of the long cantilever. The scaling factor of the segmented model was 1∶50, and the guide plates, curbs, temporary handrails and counter-rails of the check trolley etc. were simulated in the model. The scaling factor of the aeroelastic model of the overall bridge was 1∶125. The model of the largest double cantilever case and the largest single cantilever case were shown in Fig. 7 and Fig. 8.

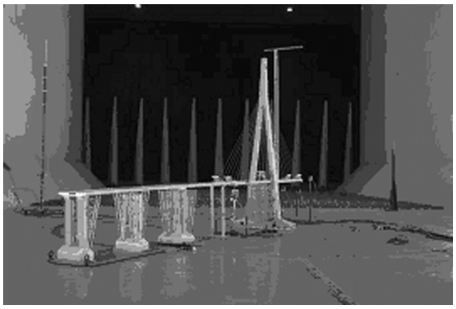


Fig. 7 The Largest double cantilever state

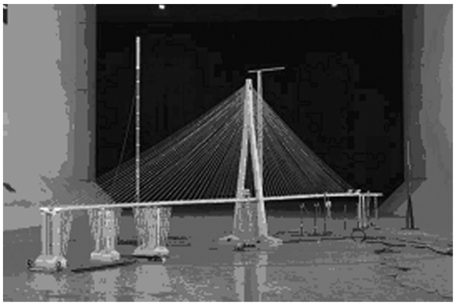


Fig. 8 The largest single cantilever state

Research results from the wind tunnel tests of segmented models indicate that the static wind resistance factor of the main girder was high and the divergent amplitude phenomenon did not appear in homogeneous flow.

Research results from the wind tunnel tests of the whole bridge model indicate that the divergent amplitude phenomenon did not appear in the longest double cantilever state (156.8 m) and the longest single cantilever state (540.8 m).

4.2 Research on the wind-resistance safety of the main girder during the construction period

4.2.1 Buffeting response analysis

To compare the buffeting response investigated in wind tunnel tests, a time domain buffeting analysis using the turbulent flow field, which was concurrent to the test power spectrum, was adopted.

A summary of the buffeting analysis in the longest single cantilever state is given in Table 1.

Table 1 Result of buffeting analysis

	Wind velocity (at deck)/ (m·s^{-1})	Buffeting response		
		Average	Mean square root	Peak
buffeting analysis(h = 0.3 %)				
Transverse/mm	44.9	2 580	$620^{\#1}$	4 750
Vertical/mm		260	$420^{\#1}$	1 720
Torsional /(°)		–	–	–
Wind tunnel test(h = 0.5 %)				
Transverse/mm	46.3	2 636	339	3 823
Vertical/mm		236	324	1 370
Torsional /(°)		0.518	0.081	0.802
Wind tunnel test(h = 0.3 %)				
Transverse/mm	46.3	2 636	$438^{\#2}$	$4\ 168^{\#3}$
Vertical/mm		236	$418^{\#2}$	$1\ 700^{\#3}$
Torsional /(°)		0.518	$0.105^{\#2}$	$0.884^{\#3}$

Note: #1—(peak-average)/3.5; #2— SQRT(0.5/0.3) × mean square root; #3—average + 3.5 × mean square root

In the case with a damping ratio of 0.003, the transverse buffeting response in the wind tunnel tests was less than that in the numeric analysis. It could have been caused by cable vibration in the transverse direction and thus increasing the structural damping. The vertical response in the buffeting analysis was in accordance with the result of wind tunnel tests of whole bridge aeroelastic model.

4.2.2 Safety assessment of the main girder

The safety of the main girder was unfavorable during the construction of the longest single cantilever state, under both normal or typhoon situations. The safety research in a series of key cases was undertaken, using the buffeting response from the buffeting analysis for assessment.

In accordance with the BS 5400-3∶2000 specification, the result shows that:

In normal construction situations (wind speeds of 17.5 m/s and 20 m/s), the strength utilization rate of the main girder was less than 1, which showed that the structure safety meets the requirements.

In typhoon conditions, after No. J34 cable tensioning was finished, the main girder strength utilization rate was greater than 1, and it didn't meet the structural safety requirements.

So, it was necessary to consider wind resistant measures and stress reduction measures to enhance the safety of the structure in the construction period.

For U-shaped stiffed ribs with even bottom plates, lower the stress of the bottom plate of girders where had the largest vertical buffeting response (100 m from the end of cantilever), can meet the safety requirements; For U-shaped stiffed ribs with inclined bottom plates, as the strength utilization rate is only 3 % of the standard one, slightly lower the stress of the inclined bottom plate of the main girder in tower area so that the strength utilization rate can be less than 1.0 to ensure the structural safety.

4.3 Wind-resistance measures

To ensure the structural safety when there is a typhoon warning, the following measures can be taken to ensure the safety of the main structure and construction equipment. a. Move the crane on the deck towards the tower 2 girders; b. Reduce or move the construction load; c. Move the mobile equipment, including the welding equipment and motor cranes to the tower area and fix them. Move the maintenance car to an area near the tower and remove the concrete balance weight on the tension platform; d. Dismantle the temporary barrier and washboard on the front end of cantilever; e. Relax the temporary connection cables appropriately; f. The crisscross cables can be set at the two ends of the cantilever and No. 34 cable could be tension more after closure of the middle span.

5 Conclusions

The elevation error of the girder control point of the Sutong Bridge in the as-built state was within (−16.6) ~ (+168.5) mm; the bridge axial error was within (−22.9) ~ (+22.5) mm; the main girder had a smooth curve and a good as-built state. The error of each cable force in the as-built state was within (−9.4) % ~ (+9.1) %. The number of cables in which the cable force had a less than 5 % error was up to 86 % in all, and the safety coefficients of the cable forces in operating situation were all 2.5 or more. The absolute value of the cable tower linear deviation was within 17 ~ 44 mm, and the tower had a smooth curved shape.

The cable-stayed bridge construction process was closely related to construction control and there was a high degree of coupling between them. Thus, for Sutong Bridge superstructure construction, the construction method was closely linked to the control method and a set of innovative technologies for the construction control were developed. At the same time, a set of construction control methods implemented to guarantee quality, operate simply and used advanced technology to achieve the ultimate goal of construction. The technology achievement can be a reference for the construction of ultra-large span cable-stayed bridges in the future.

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