Design, Fabrication and Erection of the Stiffening Girder of 3rd Kurushima Kaikyo Bridge —The World's First Three-Linked Suspension Bridge with a Stiffening Box Girder of Over 1 000 m Long—*


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Synopsis:

Kurushima Kaikyo Bridge, which consists of three bridges crossing the Kurushima Strait located within Seto Inland Sea National Park, is the first three-linked suspension bridge in the world. Many advanced technologies were adopted in the construction of the bridges. Especially, the introduction of a newly-developed self-positioning barge and a quick-joint-system enabled to install shop-fabricated stiffening box girders merely in 30 min by means of a direct hoisting method. 3rd Kurushima Kaikyo Bridge, which is one of the three bridges, spans over the west channel of the straits where the international sea route passes through. The bridge has a single span of 1 030 m long with two-hinged stiffening girders. This paper describes the design, shop fabrication and erection work of the stiffening girder at the site of 3rd Kurushima Kaikyo Bridge. The results obtained through this construction are expected to contribute to the success in the strait crossing bridge projects in the future.

1 Introduction

Honshu-Shikoku Bridge of the Nishiseto Expressway was opened to traffic on May 1, 1999 as a 60 km expressway that connects the two main islands of the scenic archipelago landscape by means of ten long-span bridges. The technical aspects of the construction of long-span bridges, which began with the construction of Ohmishima Bridge, entered a new stage with the completion of the Onomichi-Imabari route.

Kurushima Kaikyo Bridge is the world's first three-linked suspension bridges across the Kurushima Strait, which is famous for one of Japan's three major rapid tidal currents, and in erecting stiffening box girders to be constructed on an international sea route, new technologies such as a self-positioning barge developed for use in the present work[1)] and a special device, called "Quick Joint"[2)] were introduced. Furthermore, visual design[3)] with a continuously varied tower height was adopted in the bridge and many advanced technologies were adopted; these include tension field joints,[4)] a corrosion protection method[5)] for main cables by a combination of S-shaped wire wrapping and a corrosion

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Photo 1 View of 3rd Kurushima Kaikyo Bridge

protection system with dehumidified air (which was the first corrosion protection in this field in Japan), an air crossing method[6] for pilot ropes by use of a helicopter following the adoption for Akashi Kaikyo Bridge, application of parallel wire strands (PWS)[7] to hanger cables, and the use of tunneling anchorage[8] and substructures in the sea.[9] The bridge could be thus constructed by the virtue exact incorporation of all the latest technical advances.

In the three-linked suspension bridges, 3rd Kurushima Kaikyo Bridge (**Photo 1**) is a single span two-hinged suspension bridge with a bridge span of 1 030 m situated on the Imabari side of the Kurushima Strait and is the longest box girder stiffened suspension bridge in Japan. The field erection of stiffening box girders was completed without an accident by overcoming severe natural conditions and social restrictions. Especially, worthy of mention is the fact that the erection was completed in a very short period of about 2 months thanks to the adoption of a direct hoisting method using lifting beams and an all-hinge erection method.

Using knowledge obtained through the work for Kurushima Bridge stiffening box girders (Part 6) for which an NKK Corp.-Nippon Sharyo, Ltd.-Kawasaki Steel Corp. joint venture received orders (hereinafter referred to as this work) in this 3rd Kurushima Kaikyo Bridge, this paper reports on the design, fabrication and erection of the stiffening box girders of this bridge, with special focus on the technical features of this work.

2 Outline of Work

Kurushima Kaikyo Bridge is composed of three-linked suspension bridges (total length: 4.1 km) that connect Imabari City in Ehime Pref. with Oshima. The general plan of the bridge is shown in **Fig. 1**. The bridge-construction sea area is divided into three channels, where the tidal current velocity is as high as 10 knots maximum and the curved archipelago provides complex topographic features. An overview of 3rd Kurushima Kaikyo Bridge over the west channel is listed in **Table 1**. Because the bridge was to be erected on an international sea route where about 1 000 large and small ships sail a day, sailing safety measures were of course very important and avoiding the blockage of the sea for a long time was a great restriction on the execution of work.

Incidentally, The scope of work is from the center of the span to the 9P main tower.

Table 1 Outline of the project

Name of route		Route No. 317
Dimension of superstructure	Bridge type	Single span 2-hinged suspension bridge with stiffening box girder
	Span	1 030 m
	Road standard	1st class and 3rd grade
	Design speed	80 km/h
	Design live load	B-type
	Roadway width	2.5 m*1 + 4@3.5 m*2 + 2.5 m*1
	Plan alignment	Straight with short clothoid at 9P
	Vertical alignment	1.8% parabolic curve
	Slab type	Orthotropic steel deck t = 12 mm
	Main cable	PWS (636ϕ), 5 mm × 127 wires × 102 strands
	Hanger	PWS, Steel rod (at span center)
Amount of steel (Stiffening girder only)		12 876 t
Construction period (Stiffening girder only)		1995. 7. 25〜1998. 9. 30 (1 164 d)

*1 Walk way *2 Road way

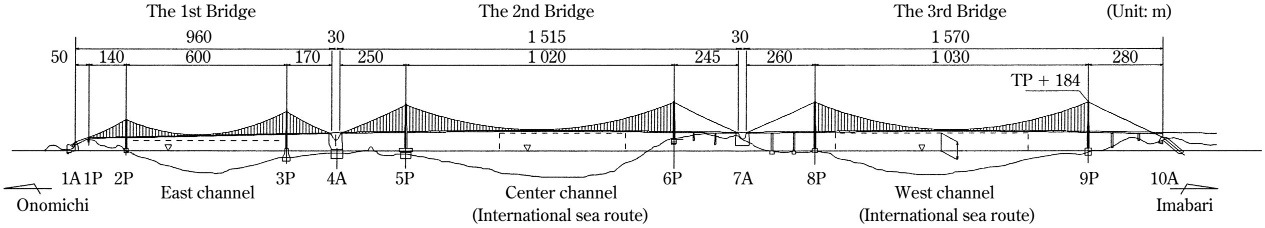

Fig. 1 General plan of Kurushima Kaikyo Bridge

3 Design Technology

3.1 Structural Features

In terms of the cross-sectional shape of a stiffening girder, a streamlined flat one-box girder that has a hexagonal cross section (**Fig. 2**) was adopted in consideration of wind stability, economy, maintainability, prevention of false images on the radar screen, etc. The stiffening girder has the following structural features:

(1) Because of their excellent maintainability and economy, truss members were used in the diaphragm and center web.

(2) PWS (parallel wire strands) were applied to hanger ropes. And the main cable of the hanger rope and the stiffening girder were pin connected.

(3) Because of the road alignment of the Imabari side approach, the alignment of the stiffening girder also contains some plane curves.

3.2 Analysis of an Entire System of Suspension Bridge and Design of Stiffening Girder

In order to have an accurate understanding of the behavior of the highly flexible suspension bridge, a structural analysis of the whole suspension bridge system was carried out by conducting an analysis for shape determination by a finite deformation theory intended for use in-plane frame models, then by conducting fixed-load and influence line analyses based on the linear finite deformation theory in which the shape and the geometrical stiffness by forces generated by cable members were considered. The latter structural analysis involved the so called "fish bone model," in which a stiffening girder is replaced with a bar element that passes the shear center of the stiffening girder and a hypothetical overhanging rigid member is set between the hanger fixtures in the direction at right angles to the bridge axis.

On the other hand, in designing a stiffening girder, the fairing and center web were evaluated as a stress member and a plate of cross section with full web having equivalent shearing stiffness, respectively. After that stress intensity was checked. In examining the suspension bridge members during erection and transportation, structural analyses during the erection stage and agitation analyses during marine transportation were carried out. The results of these analyses were reflected in the reinforcement of the stiffening girder, its erection plan and the design of temporary works.

Incidentally, the maximum stress of stiffening girder generated in the static analysis was $\sigma_{cmax} = 107\,\text{N/mm}^2$ at the leading end of the fairing by wind loads, and the maximum amount of movement in the axial direction was under the normal condition $l_{max} = 974\,\text{mm}$.

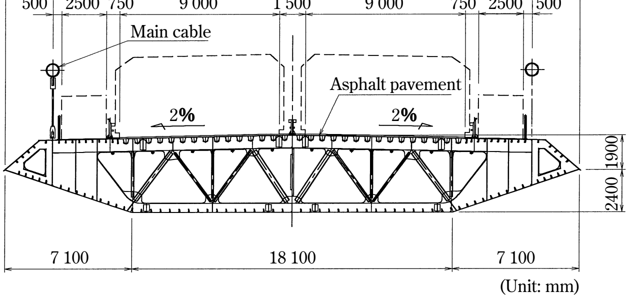

Fig. 2 Cross section

3.3 Design of Steel Deck Plate and Floor System

In the design of the steel deck plate, given the adoption of trough ribs as the longitudinal ribs of roadway area and plate ribs as those of walkway area, the F. S. M. (finite strip method) was applied to the former and the equivalent beam grid[10] to the latter as means of most effectively understanding the behavior of each portion. Furthermore, because the center stringer and diaphragm that support the steel deck plate are truss, its structure members in the design of the steel deck plate was evaluated with precision by considering the effect of deflection of supporting portions due to shearing deformation as additional working stress.

The floor system is composed of a center stringer and a diaphragm, which are trusses, and a side web of a plate of cross section with full web. In order to ensure consistency with the deformation of the whole suspension bridge system, working stresses were calculated by continuous beam grid analysis in which the shearing deformation of trussed members and the relative displacement between the hanger fixtures were considered. For the diaphragm, additional stresses were examined by applying the theory of BEF analogy[11] to the torsional deformation of the stiffening girder. Furthermore, because the action as the transverse ribs of the steel deck plate and the action as the upper chord of the truss work simultaneously and out-of-plane deformation behavior[12,13] was also considered, safety was ascertained by the three-dimensional finite element method.[14] The loading positions in the finite element method were determined from the proportionality[15] between the stress concentration near a scallop and the shearing force of the transverse ribs, and the scallop shape and the plate thickness of the transverse ribs for local stresses were secured.

3.4 Design of Structure of Hanger Fixture

Based on the amount of out-of-plane deformation of the hanger fixture two types of pins, i.e., a unidirectional pin and a universal pin capable of rotation in all directions, were applied to the hanger fixture. The basic structure of the fixture was determined by simple calculations[16] from the tension and bend angle of hanger

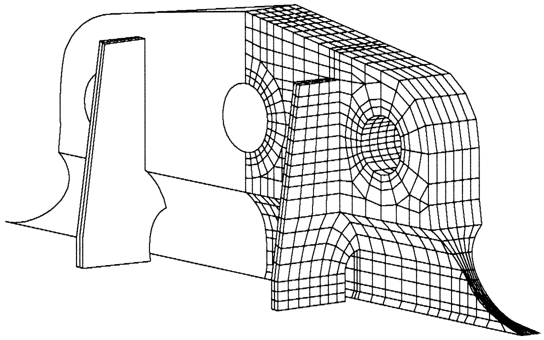

Fig. 3 FEM model for hanger fixing plate

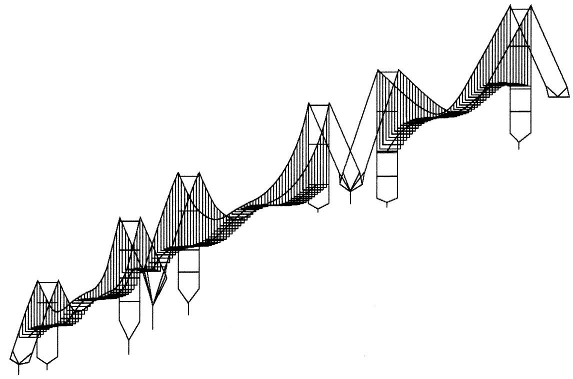

Fig. 4 Model of time history response analysis for three-linked bridges

ropes, obtained from the analysis of the entire system of suspension bridge and a structural analysis during the erection stage, and by the three-dimensional finite element method (**Fig. 3**). The details of the structure were determined and verified and safety was checked. Based on the results of a fatigue test conducted beforehand, it was decided to fabricate the anchor plate (pin plate) of the unidirectional pin by cutting out one steel plate and obtain joints with the steel deck plate by full penetrated welding. The plate bending state of the steel deck plate resulting from being off-center to the side web, which is the brace member of this welding joint, was examined for stress by the finite element method.

3.5 Design of Girder End

A girder end is connected to the main tower by a tower-link bearing with larger rigidity than that of the hanger rope; therefore, the stress is such that a vertical shearing force is dominant. Because this shearing force is transmitted by the center stringer and side web to the tower-link, the girder end was modeled to a beam grid structure and thereafter the apportionment rate of each web was calculated and reflected in the design. For the end diaphragm, etc., it was regarded as a simple beam supported by the tower-link. Therefore, it was decided to consider the horizontal force of a wind shoe, which is caused by the vertical shearing force on the center stringer and by wind loads, and the horizontal force[17] which is caused by the vortex-induced vibration generated in the main tower in a low-wind-velocity region.

In checking the anchor portion of the tower-link, the following were taken into consideration: the axial force obtained from the analysis of the whole suspension bridge system, the in-plane bending moment caused by the rolling friction of the link bearing, the out-of-plane additional bending moment caused by the deformation of the link bearing, and the eccentricity of the pin plate by fabrication and erection errors.

3.6 Seismic Design

In seismic design, working stresses are determined using a fish bone model of the whole system by conducting response spectrum analyses of short- and long-duration[18,19] earthquakes. Furthermore, for long-duration earthquakes, the effect of phase difference inputs of earthquake motion on each substructure was ascertained by conducting a time-history response analysis of a model of three-linked bridges (**Fig. 4**).

The center stay was designed for a mechanism in which forces acting on the main cable were reduced by causing the center stay to be broken during an earthquake, and 60% of the maximum tension obtained from the analysis (equivalent to a seismic force with a return period of 25 y) was set as a breaking tension. Furthermore, by introducing a pre-stress tension, continuous relaxation was prevented and it was ensured that the center stay could be used in the elastic range.

3.7 Wind-Resistant Design

For wind stability, wind-tunnel tests were separately carried out using partial models (scale: 1/60), a full-bridge model (scale: 1/160) and a topographical model (scale: 1/1 500). From the test results, problems to be checked were ascertained for both the completion stage and the erection stage.[20] Therefore, in order to conduct a more detailed examination, a large full-bridge model wind-tunnel test was conducted using a large full-bridge model of 1/60 scale and a topographical model of 1/150 scale.[21]

In the large full-bridge model wind-tunnel test, it was ascertained that in the completion stage a flutter critical wind speed of 70.2 m/s was resisted both in a uniform flow and a turbulent flow. Stability was also verified for factors that have an effect on wind stability, such as handrails for walkways (including spaces for motorbike travel), travel rails for surface maintenance vehicles and fairing shape.

On the other hand, flutter occurred in a uniform flow in the middle period of the erection stage at the lower wind velocity than design value and it was ascertained that the installation of a cross hanger would be effective in damping flutters.

Furthermore, no vortex-induced vibrations were observed either in the uniform flow of the completion stage or in the turbulent flow a vibration phenomenon that could have pose a problem in practical application.

3.8 Design of Wind Shoe

To prevent a transverse force from acting on the tower-link in the case of an abnormality, the wind shoe must be installed with high-precision, that is, the clearance of a girder end to the sliding surface of the wind-tongue must be 2 mm on one side. Therefore, it was decided to adopt a new support system which combines a rubber bearing support plate and a wedge plate. In this support system, compared with the conventional wind shoe type based on a pivot bearing support, the new support system is much more workable in the connection to the narrow wind-tongue, and may also have superior resistance to lateral impact forces during an earthquake, etc.

Incidentally, it is also necessary for the connection to the tower-link to meet the requirement for the clearance of a girder end of 5 mm on one side perpendicular to the bridge axis. Accordingly, although the relative installation precision for the wind shoe becomes higher, satisfactory results have been obtained by using field measurements to rigorously control the precision of cutting, etc., in shop fabrication.

4 Fabrication Technology

4.1 Outline of Fabrication

In shop fabrication, a large block process was adopted in which 4 panels of the diaphragm (length: 12.2 m) are regarded as a fabrication block and in a standard portion 3 fabrication blocks are assembled into one stiffening girder block (length: 36.6 m). In fabrication work, measures were taken to prevent a decrease in the fatigue strength of steel deck plate. The welding work, precision of the pin plate and the accuracy of positioning in relation to the tower at the stiffening girder end were especially important control items. **Photo 2** shows a temporary assemblage of 4 blocks at the girder end where the welded parts had been completely fabricated.

4.2 Full-Scale Drawings

Although the vertical alignment of this bridge is a parabolic gradient of 1.8%, it was approximated to a circular curve in full-scale drawings. Furthermore, the diaphragm was located in the normal line direction of the stiffening girder to rationalize the fabrication of the standard portions of the stiffening girder.

On the other hand, the 9P main tower side of the stiffening girder has a horizontal alignment composed of clothoid curves and high accuracy is required of connections to the main tower, etc. Therefore, the drawings of the diaphragm were prepared in such a manner that the diaphragm would be drafted in a vertical state.

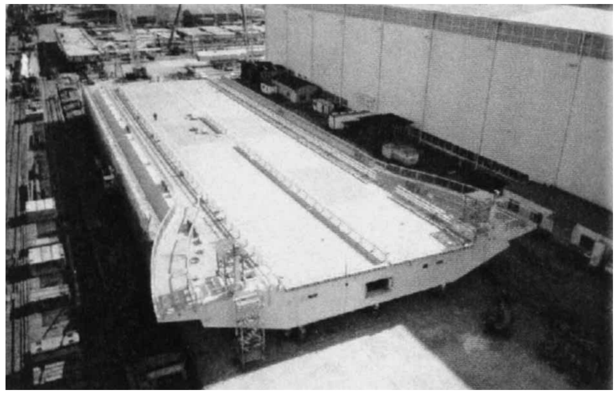

Photo 2 Assembled blocks at Harima Works

Incidentally, it was decided not to make a length correction of the stiffening girder in which the curvature of the earth is considered, because the value is negligibly small (5.9 mm) and is within the range of fabrication tolerance.

4.3 Shop Cambers

The stiffening girder is supported at the hanger-rope intervals (12.2 m) in the bridge axis direction and at the main-cable intervals (27 m) at right angles to the bridge axis. However, because the stiffening girder is very stiff and the amount of displacement under dead load is minute, it was decided not to set shop cambers. Furthermore, the shrinkage of the steel deck plate induced by the welding which was performed to produce field joints in the steel deck plate, was assumed to be 1 mm/joint. On the basis of this supposition, a structural analysis was conducted using a plane framework model for the entire system of one suspension bridge by the finite deformation theory. As a result, it was decided not to set shop cambers associated with the shrinkage of welded portions, because the amount of displacement was sufficiently minute for the tower intervals both in the vertical and horizontal directions.

4.4 Specifications of Steel for Hanger Fixtures

The pin plate was of such a structure that a 116 mm thick plate was cut up to 20 mm at the base and attached to the steel deck plate. This led to concerns that center segregation of an extra-thick plate would occur, which in turn would cause a decrease in strength. Therefore, in order to assure strength, etc. for a thickness of 1/2 t (t: plate thickness), the specifications of the chemical composition of the steel material called for a sulfur content of not more than 0.008% and a carbon content of not more than 0.48%. In addition, an ultrasonic wave flaw detection inspection was conducted.

On the other hand, the steel deck plate to which the pin plate is to be welded had a thickness of 10 mm and was out of the JIS standard as a lamellar-tear-resistant plate. Therefore, it was decided to adopt the specifications of steel material (sulfur-content-controlled steel material) in which the sulfur content would be 0.008% or less from the correlation between the sulfur content of chemical composition and the reduction of area as a steel that would guarantee a reduction of area[22] equivalent to the symbol Z25 in accordance with WES.

4.5 Fatigue Design for Steel Deck Plate

A high-strength bolt joint for the trough rib of a steel deck plate requires scallops due to the butt welding conditions of steel deck plate. Therefore, due to shearing forces caused by the movement of vehicles, fatigue cracks may occur at the toe of a boxing weld and the edge of the scallop. In order to prevent this, the scallop size was minimized; for scallops present in portions of high travel frequency of vehicles, measures were taken to make the toes of welds smooth.

For fillet welds between the steel deck plate and the trough ribs, it was decided to ensure a depth of penetration equivalent to 75% of the rib plate thickness (6 mm) in order to prevent stress concentration[12] due to local deformation of the steel deck plate. Furthermore, in portions where a trough rib passed through a transverse rib, the use of a conventional scallop that would create a weak point vulnerable to fatigue was abolished. Corner cutting was performed in all such portions, after which the transverse rib backfilling was welded together.

5 Erection Technology

5.1 Outline of Erection

Because the stiffening girder of this bridge is a single-cellular box girder, stiffening girder blocks with full cross section were erected by the direct hoisting method. For the blocks near the towers where a self-propelled barge with an automatic positioning capability without mooring ropes could not enter, the swing erection method was adopted. The Kurushima Strait includes an international sea route where the channel is narrow and tidal currents are rapid. Moreover, the direction of sailing must sometimes be changed according to the tidal current flow. Therefore, the adoption of the direct hoisting method was made possible by meeting the precondition that in order to secure the safety of passing ships, erection work must be finished quickly.[23–27]

The time required for this erection work was substantially shortened from the conventional 3 h[28,29] to 27 min on the average[30] by the development of temporary works and erection facilities, including self-propelled barges. Furthermore, due to the shortening of the erection cycle by the all-hinge method and the reduction of the frequency of erection by increasing the size of stiffening girder blocks, the erection of all 33 blocks of the 3rd bridge was completed in 63 d (average time per block: 3.8 d).

Photo 3　Execution by the direct hoisting method

5.2 Direct Hoisting Method

5.2.1 Direct hoisting erection

The erection of 13 blocks from the center of the span to the closing block was carried out by the direct hoisting erection method following the procedure described below. A view of the work is shown in **Photo 3**.

(1) A self-propelled barge loaded with a stiffening girder block was positioned to stay immediately below the erection point.
(2) The hooks of two lifting beams installed on the main cable were connected to the stiffening girder block.
(3) The self-propelled barge was removed from the designated point during unloading and the stiffening girder block was lifted into position by the lifting beams and secured to hanger ropes.
(4) After the stiffening girder block was secured without stress, the stiffening girder was temporarily connected to an existing girder by means of an erection hinge.

The erection plan was formulated so that the erection was started during tidal current change and the erection time until the completion of lifting was 50 min.

The length of the stiffening girder block (36.6 m) and the weight of the stiffening girder block including loaded erection equipment and materials (approximately 500 t) are the largest scale ever used in Japan with the direct hoisting method.

5.2.2 Swing erection

The erection of two blocks near the tower was carried out by the swing erection method. A view of the block to be attached to the main tower is shown in **Photo**

Photo 4 Stiffening girder under the swing erection

4. After being lifted, the stiffening girder block was shifted toward the direction of the bridge axis by load shifting between the two lifting beams. It is in this point that the procedure for the swing erection method is different from that of the direct hoisting method. Furthermore, it was decided to lift the stiffening girder block at two points and to use a special lifting device comprised of a triangular eyebar for connecting the two lifting beams beforehand.

In consideration of the lifting capacity of the lifting beams during load shifting, it was decided to set the length of the stiffening girder block at 12.2 m and the maximum weight of the block at approximately 280 t.

5.2.3 Sequence of erection

The sequence of erection is shown in **Fig. 5**. As with the case of the conventional direct hoisting method,[31,32] the erection work was started from the middle of the span and pushed forward toward the two towers symmetrically (erection was performed alternately with the Yokogawa Bridge Corp.-Mitsui Engineering & Shipbuilding Co., Ltd.-Haltec Corp. joint venture in charge of work on the opposite side of the strait). The

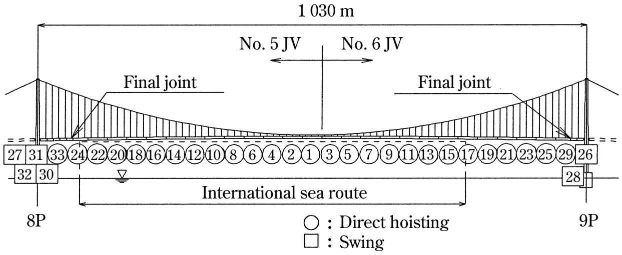

Fig. 5 Sequence for block erection

blocks for the swing erection method were erected in such a sequence that they would overhang from the main tower toward the center.

Days on which the tidal current velocity 50 min after a tidal current change was not more than 3 knots were selected as the days of erection and one block was erected each day for the 3rd bridge.

5.2.4 Major erection equipment

The main erection equipment in this work are as described below.

(1) Self-propelled Barge

The self-propelled barge sails at a speed of 7 knots with a stiffening girder block onboard and has a capability of staying still at a designated point without the need for mooring ropes within a 2.5 m radius of a target position at a tidal current speed of not more than 3 knots due to the automatic control of omni-revolving propellers at four corners of the barge[33–39] and the main engine. The actual time required from the start of approach to an erection point to the staying still at a designated point and stabilization was 7 min on the average.

(2) Lifting Beam

The lifting beam is a transportable stiffening girder block hoisting equipment capable of being installed in an arbitrary position on the main cable. This lifting beam, which was designed and fabricated for use in this project,[40,41] was assembled and disassembled using a tower-top crane. The designed load was set at 175 t × 4 hooks to respond to the increased size of stiffening girder blocks and the lifting speed was raised to 5 m/min maximum during the hoisting of the designed load. Furthermore, to achieve the design concept of a double safeguard system, the lifting beam was designed so that two ropes were connected to each hook. Furthermore, in order to reduce section deformation of main cable, the main hoist winch was arranged under the main tower in the conventional way, although the traveling system employed a method of sliding on a rail girder like a looper in place of the conventional roll-on method.

(3) Quick Joint

Quick Joint, which is a connecting device[2] developed for use in the mooring of steel caissons of Akashi Kaikyo Bridge, was further improved for use in this project. The system is composed of a male device and a female device, each attached to a girder-side metal hanger on the lifting beam hooks. As shown in **Photo 5**, the system is of such a structure that mechanical engagement is performed using a lead wire as a guide.[23,24,42] Manual work was limited to light work involving shackling the lead wire and the leading end of the male device. Therefore the four hooks could be safely and positively connected in 5 min on average even on the oscillating self-propelled barge.

Photo 5 Junction of the quick joint

5.3 All-Hinge Erection Method

5.3.1 Structure of erection hinge

Due to the use of the all-hinge erection method, the stiffening girder blocks during the erection period were temporarily connected by erection hinges. Under these circumstances, there were concerns about the occurrence of flutters resulting from a decrease in the torsional stiffness of the girder at a hinged portion.[20,43,44] There were also concerns about an adverse effect of an increase in the bending angle at a hinged portion and the working stresses caused by the large block design. Accordingly, the erection hinge structure of this bridge was planned in such a manner that stiffening was not performed for the in-plane bending between blocks and the interference by connections was prevented. At the same time, in regard to the load distribution after partial splicing, which will be described later, the erection hinge could handle all working stresses during the erection period.[23] **Figure 6** shows an overview of the arrangement and structure of the pin-plate and shear-key adopted in the project.

5.3.2 Partial splicing

The stiffening girders in this bridge were designed based on the existence of no stress at the time of completion and negative bending moment was generated in girders in the late stage of erection. However because the traveling scaffolds for field splicing were built after the erection of closing blocks within the work district, lower flanges would interfere with each other. Therefore, it was decided that when the lower-flange intervals of a stiffening girder block had decreased, the girders would be forcedly drawn nearer using jacks and the longitudinal rib of lower flange where work could be carried out on the inner surfaces of the girders would be completely spliced before the lower flange. Furthermore, in order to ensure that erection hinges for the bending moment would function perfectly during a storm, an area equivalent to two-thirds of the lower side of the side web and the center web were also spliced at the same time.

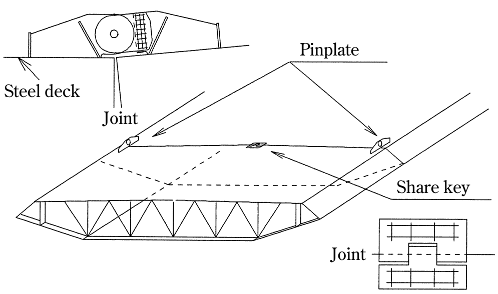

Fig. 6 Structure of erection hinge

5.4 Procedure for Erection of Special Portions

In this bridge, there are special sections where erection procedures different from those for general sections were adopted for reasons of construction restrictions. For example, because there were sections where steel rods were used[7,45] in the hangers near the center of the bridge and because lifting beams on the tower side during the erection of blocks attached to a tower could not be arranged immediately above an erection position. The erection processes in these special sections are described below.[46]

(1) Erection of the Rod-type Hanger Section

A stiffening block installed after the anchorage of a hanger was in a lower position than existing girders due to the effect of the lifting beam on the main cable. Although in general sections it was possible to adjust the lifting height with a lifting beam in order to connect erection hinges, the upward displacement of a stiffening girder block was constrained in the rod-type hanger section. For this reason, the erection procedure was modified so that erection hinges were connected after the adjustment of a level difference with existing girders by rotating a stiffening girder block in a vertical plane after a hanger was installed on the side of the erection hinges. Incidentally, although compressive forces were acting on the hangers erected beforehand during the fixing of remaining hangers, it was ascertained by structural analysis that the values of the compressive forces were small and did not exceed the lifting capacity of the lifting beams.

(2) Passing of Lifting Beam over the Rod-type Hanger

When a lifting beam passed over a steel-rod hanger in the middle of a stiffening girder block, compressive forces acted on the hanger. Even though the torsional stiffness of the main cable was small, there was no problem in the buckling strength of the rod itself. Nevertheless, it was assumed that instability could be caused by, for example, a universal joint on the cable band side being displaced in the out-of-plane direction of the cable. Therefore, in order to prevent damage to the hanger socket, pin-plate, etc. in a universal joint, it was decided to constrain movable parts by

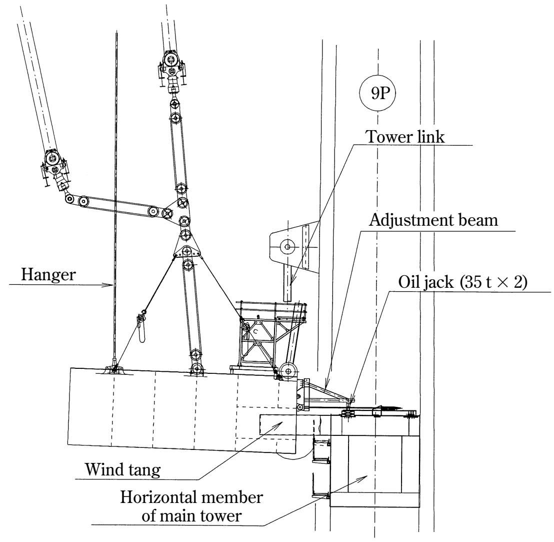

Fig. 7 Vertical adjustment of girder block at the tower

inserting wooden wedges.

(3) Erection of Block Attached to the Tower

At the tower top there was a balcony-type staging to be used in the later period of wrapping and the lifting beam on the main tower side could not be installed immediately above the erection place of a block attached to the tower. Therefore, pulling-in from the main tower side had to be done at the time of swing erection. This work was carried out by use of a winch and set-back equipment for the stiffening girder blocks (PC steel bar and center-hole-jack).

The erection plan was formulated so that the pulling-in force could be reduced by moving the lifting beam on the center side to the tower side. Furthermore, the block attached to the tower included a tower link and a wind shoe, which are structures of connection to the tower, and the height of these parts was adjusted using an adjustment jig overhanging from the girder end to the tower side and hydraulic jacks on the horizontal member of the tower as shown in **Fig. 7**.

5.5 Wind-Resistant Measures during Erection

5.5.1 Measures against out-of plane bending moment

Out-of-plane bending moment acting on a stiffening girder due to storm loads was smaller during the period of erection than upon completion because the stiffening girder was not constrained in the direction perpendicular to the bridge axis except by the hanger. After the erection of the final block, however, the out-of-plane displacement of the end of a stiffening girder was constrained by a wind shoe; thus, the out-of-plane bending moment working on a stiffening girder increased greatly from the level before the erection of the final block.

Because joining by erection hinges and partial splicing alone had insufficient strength, it was decided to cut the edge of the center side of the closing block to reduce the out-of-plane moment during the erection period and to splice the final block after the completion of the full splicing of blocks in general portions.

Furthermore, a stiffening girder overhanging from the main tower side, particularly in the wind shoe, undergoes great lateral rotational deformation due to loads during a storm, causing it to interfere with the girder adjoining the edge-cut portion and the tower. Therefore, these blocks were set back and rotation was constrained by ensuring load transmission between the end of the stiffening girder and the tower wall via a buffer material.

5.5.2 Measures against buffeting

Buffeting during erection may cause damage to bridge members due to the opening and closing of the lower flange before partial splicing. Therefore, it was decided to prevent this phenomenon by carrying out partial splicing beforehand. To prepare for interference, a buffer jig was installed in the longitudinal rib of the lower flange. The timing of partial splicing was planned in consideration of the amount of opening and closing of the lower flange that was supposed to be caused by buffeting, the number of joints formed per day, and the capacity of the draw-in equipment. This was done by calculating the lower-flange intervals and the forced draw-in force in each step of erection.

5.5.3 Measures against secondary stress of hanger cable

It was assumed that during storms in the initial to the middle stage of erection, in a hanger for a stiffening girder block in the middle of the span, hanger cables would be bent sharply at hanger fixtures, with the result that the hanger stress (including the secondary stress of steel wires forming the hanger cable) would exceed the allowable value. Therefore, it was decided to reduce the force working on the hanger by lifting existing girders fixed to this hanger by means of a lifting beam. Furthermore, it was decided to make provisions against an unexpected situation by estimating a critical wind velocity causing an excess of stress by an analysis.

During the actual construction, no storm with a wind velocity exceeding the critical value occured, so it was not necessary to take the above measures including those against fluttering.

5.6 Measurement of Camber on Stiffening Girder during Erection

Because the pin fixing method was adopted as the hanger fixing structure of this bridge, it was impossible to adjust the girder shape during erection. However, in order to understand the role of girder shape and check the accuracy of alignment, a site survey was conducted each time a stiffening girder block was installed.[23] A comparison between the measurements of vertical displacement of stiffening girders and the results of analysis revealed that the maximum value of differentials was $\delta_v = 247$ mm (upon completion of erection: $\delta_v = 138$ mm) and that the maximum differential between the right and left cross sections of a stiffening girder upon completion of erection was $\varDelta = 12$ mm. Although it is likely these differentials were greatly affected by errors in measured temperature, there is a high degree of correspondence between the two in terms of the scale of structure.

6 Conclusions

The design, fabrication and erection technologies related to 3rd Kurushima Kaikyo Bridge were described. The main results obtained in this topics were as follows:

(1) By forming the center stringers and diaphragm of a stiffening girder as a truss structure, it was possible to formulate a design with excellent maintenability and cost-effectiveness.

(2) The adoption of PWS (parallel wire strands) in hanger cables was more economical compared with conventional structures. When designing fixation, it is important to consider the stress concentration and fatigue of a pin-plate.

(3) On the bases of the results of structural analysis, shop cambers field-welded in a steel deck plate were not taken into consideration. A site survey taken after erection confirmed that there is no problem at all in terms of the accuracy of alignment.

(4) The severe connection accuracy at the end of a stiffening girder (connection to a wind-tongue: 2 mm, connection to a tower-link: 5 mm) could be ensured by shop-fabrication precision which reflects the results of field measurements.

(5) Analysis by the finite element method showed that the orthotropic deck used as the steel deck plate had structural features that would help to prevent fatigue. Furthermore in the fabrication of this steel deck plate, measures were taken not to reduce fatigue strength.

(6) The actual erection time per block of stiffening girders under the direct hoisting method was 27 min on the average vs. 50 min in the plan. Thus, the conventional erection time could be substantially shortened.

(7) Owing to the adoption of the all-hinge erection method, the period of sea blockage was about 2 months. The erection work period until the erection of the final block was about 4.5 months. The average erection cycle of stiffening girders for the third bridge was 1.9 d.

This bridge construction work was carried out under sometimes severe natural conditions and aesthetic regulations. The project was executed under the guidance of Honshu Shikoku Bridge Authority in conjunction with the Yokogawa Bridge Corp.-Mitsui Engineering & Shipbuilding Co., Ltd.-Haltec Corp. joint venture in charge of Kurushima Kaikyo Bridge stiffening girder (Part 5) work. The authors would like to extend their sincere thanks to the persons involved with this work.

Kurushima Kaikyo Bridge represents a collection of modern suspension bridge technologies and is expected to contribute greatly to the development of future long-span bridge technology. In recognition of this, it received the prestigious Tanaka Award from the Japan Society of Civil Engineers in 1988 (Works section). The authors sincerely hope that this bridge will make a major contribution to the formation of a regional traffic network and the economic growth of the Nishiseto district, and that the advanced technologies developed in connection with this bridge will be employed in the next strait-related bridge project.

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