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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:

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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.

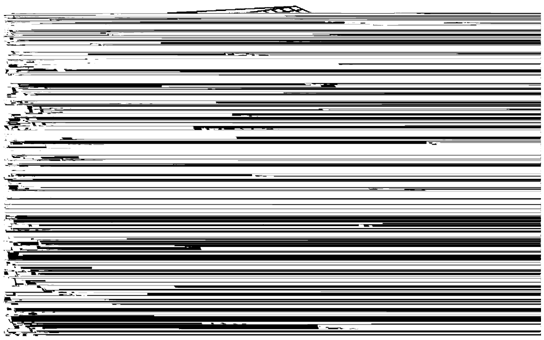

Fig. 3 FEM model for hanger fixing plate

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.

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]

5.3 All-Hinge Erection Method

5.3.1 Structure of erection hinge

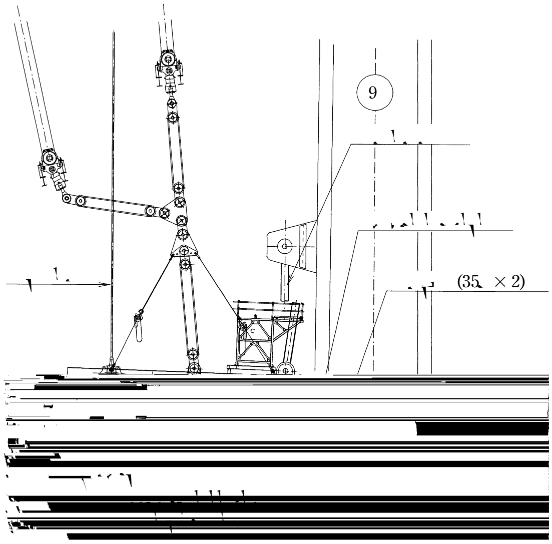

Fig. 7 Vertical adjustment of girder block at the tower

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 $\Delta = 12$ 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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