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Design and Construction of a Cable-Stayed Bridge with Mixed Structure

Conception et construction d'une structure mixte pour un pont à haubans

Bemessung und Bau eines Verbundträges für eine Schrägkabelbrücke

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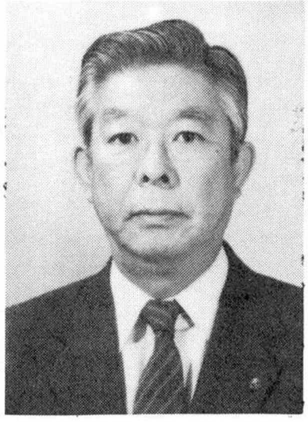
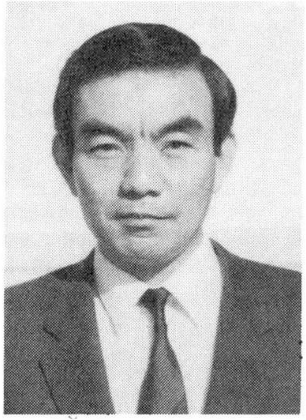
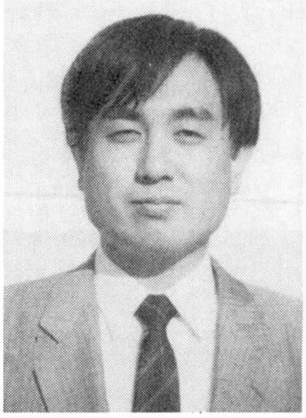

SUMMARY

Ikuchi Bridge is a cable-stayed bridge with a 490 m main span. The main girder of this bridge is a steel concrete mixed structure consisting of a steel girder for the center span, and prestressed concrete girders for both side spans. The connected parts between them were designed by taking account of the transmission mechanism of the stress, steel fabrication, actual construction methods etc. This report deals mainly with the design and construction of the connected parts between two types of girders.

RÉSUMÉ

Le Ikuchi Bridge est un pont à haubans avec une travée principale de 490 m. La poutre principale de ce pont est une structure mixte acier-béton, comportant une poutre en acier pour la travée centrale et des poutres en béton précontraint pour les deux travées latérales. Les éléments de jonction entre les deux types de poutres ont été conçues en tenant compte de la mécanique de transmission de la charge, de la fabrication en usine, des méthodes actuelles de construction, etc. Ce rapport traite essentiellement de la conception et de la construction des éléments de jonction entre les deux types de poutres.

ZUSAMMENFASSUNG

Die Ikuchi Brücke ist eine Schrägkabelbrücke mit einer Hauptspannweite von 490 m. Der Hauptträger dieser Brücke ist eine Verbundkonstruktion aus Stahl und Beton, die aus einem Stahlträger für den mittleren und aus Spannbetonträgern für die beiden Seitenspannweiten besteht. Die Verbindungsteile zwischen ihnen wurden unter Berücksichtigung der Belastungsübertragung, der Stahlherstellung, der aktuellen Bauverfahren, usw. entworfen. Dieser Bericht behandelt in erster Reihe den Entwurf der Verbindungsteile zwischen den zwei Trägertypen.

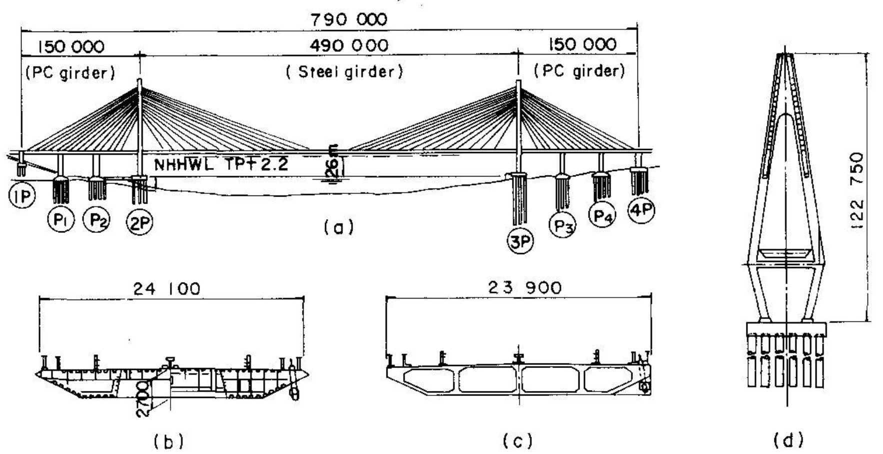

Fig. 1 Ikuchi Bridge : (a) Side view (b) Steel girder section,
(c) Concrete girder section,
(d) Side view of tower (2P)

1. OUTLINE OF IKUCHI BRIDGE

The Ikuchi Bridge is a cable-stayed bridge, connecting Innoshima island and Ikuchi island, on the Onomichi-Imabari, west side route of the Honshu-Shikoku Bridge Project in Japan.
Characteristics of this cable-stayed bridge are that the main girder is a steel-concrete mixed structure which consists of steel girder for the center span and prestressed concrete (PC) girders for both side spans.
This type is adopted, because as the side span length is short in comparison with the center span length from the condition of horizontal road alignment, it is favorable that the dead load of side spans is heavy in order to eliminate the negative reaction force. For this reason the main girder in the side spans is made of concrete, which is supported by the two middle piers as shown in Fig. 1.

2. DESIGN OF CONNECTION

2.1 OUTLINE

The steel girder is firmly jointed to PC girder over an entire section at the point out of the support on the lateral beam of the tower to the center slightly. On the connection between the steel girder and PC girders, where the structural and material characters change. It was necessary to fully investigate the mechanism of load transmission and design to be safe and rational.
For the structure type of connection, after studies about different kinds of types through large scale experiments, the type transmitting load from the steel cells to the PC girder through the filled concrete in the cells was adopted ([1], [2]).
Perspective of the connection and the typical crosssection are shown in Fig. 2 and Fig. 3 respectively.

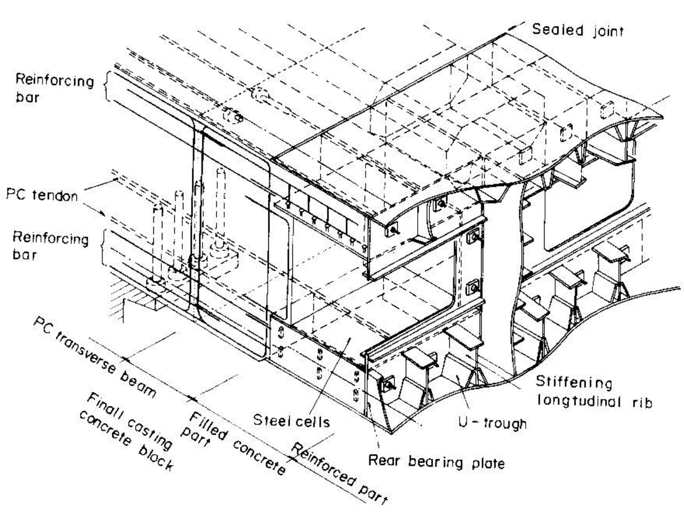

Fig. 2 Perspective of connected part

2.2 DESIGN PROCEDURE

The connected part was designed by the following procedure.
(a) Determined the shape and general proportions of the connection considering the mechanism of the stress transmission and the ease of fabrication and erection.
(b) Determined the design section forces at the connected part.
(c) Proportioned the sections of the main girder : PC section, steel section and composite section.
(d) Designed the elements of the stress transmission part : shear connectors, rear bearing plate and checked the stress in the filled concrete.
(e) Designed the structual detailes : the reinforced part in the steel girder, the tip of the steel plate and the arrangement of the reinforcing bars in the filled concrete.

2.3 DETAILS OF CONNECTION

The details of the connection was decided as follows.
(1) Steel cells
The width of the steel cells equals to the distance of the U-shaped ribs and the height was decided considering ease of fabrication of the steel cells, arrangement of the prestressing tendons and reinforcing bars, casting the filled concrete and required area across the filled concrete to distribute the load to the PC girder.
The length of them was decided considering how to transmit the load from the steel cells into the filled concrete smoothly and to arrange the required shear connectors.
(2) Reinforced part in the steel girder
In the reinforced part longitudinal stiffening ribs were welded to the deck in order to distribute the load smoothly from the deck plate, inclined flange and lower flange into the filled concrete part. This stiffening ribs were inserted in the the U-shaped ribs of the deck plate.
The length of the reinforced part was determined by consideration of the stress distribution characteristics and the distance of cross beams.

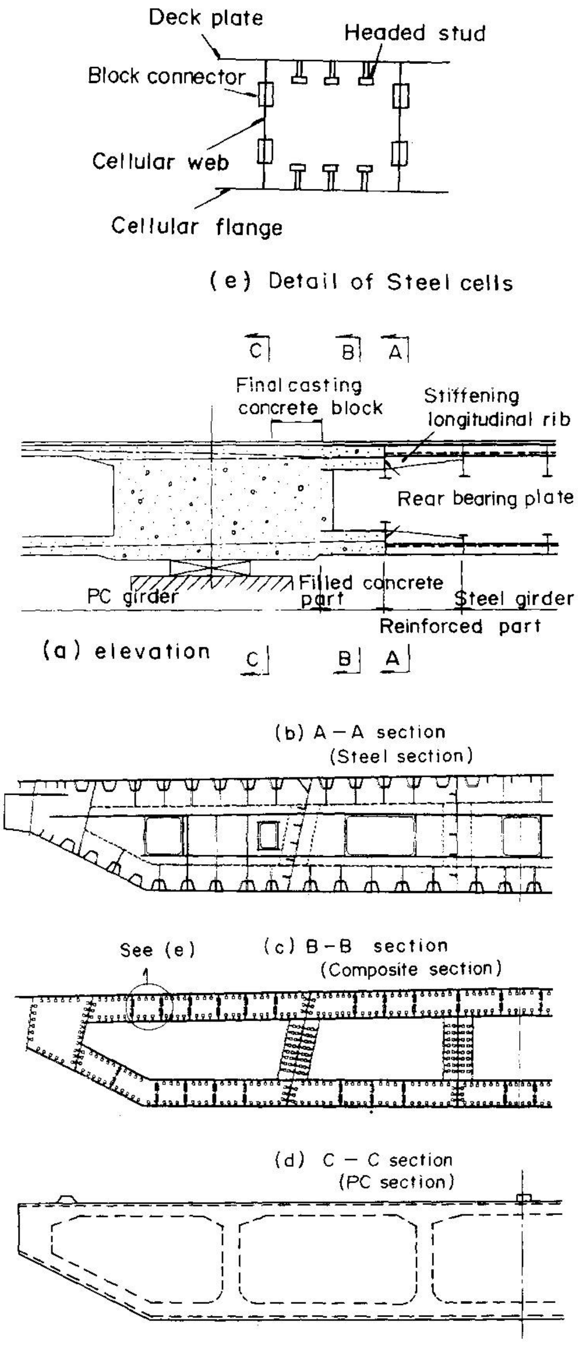

Fig. 3 Cross-section of connected part

(3) Selection of shear connector
Headed studs were adopted for shear connectors welded to deck plates, inclined flanges, lower flanges, flanges in the steel celles and webs, which resist the tensile stress as well as the shearing stress and protect to separate the filled concrete from the steel cells. However the rectangular blocks were welded to the webs of the steel cells between the filled concrete, for there is no problem of the separation and headed studs disturb the assembly of this part.

2.4 DESIGN SECTION FORCES

An example of the design diagrams for the section forces acting on the main girder under service loading conditions is shown in Fig. 4. The bending moments distribution in the center span are relatively smooth, whereas those for both side spans distribute with some peaks at the supports of the middle piers and towers.

These bending moments were adjusted mainly by varying the tensile force of stay cables in center span, because the flexural stiffness of steel girder is smaller than that for the prestressed concrete girder.

The maximum axial forces in the girder occur at the location of each tower due to the type of cable-stayed girder. The shearing forces for side spans are extremely larger than those for center span.

The typical values for section forces which have been used for design of the connected part of main girder are shown in Table 1.

2.5 TRANSMISSION PROCESS OF INTERNAL FORCES

Table 2 shows the working stresses acting on the section of steel-concrete connected part under service loading conditions. The tensile stresses in the composite section at the filled concrete part does not occur due to the action of large axial compressive force.

The transmission mechanism of axial forces at the connected part between steel beam and prestressed concrete girder, was considered that the stresses acting on the standard portion of steel beam are dispersed at the reinforced part of the beam and transferred to filled concrete part.

At the filled concrete part, the stress acting on the steel multi-cells will be transferred to the filled concrete by the following mechanism, namely, ① rear bearing plate, ② shear connectors and ③ friction at the interface between steel and concrete. The stresses at the filled concrete are uniformly transmitted to the prestressed concrete transverse beam on the supports at the tower.

As the results of the static loading tests in which the scale model specimens for one steel cellular box were used, it was recognized that the rate of the stresses transferred by the friction was considerably large ([3]). However, the transmission factor by the friction was neglected in designing because of its uncertainties, and this would be considered as a safety margin.

Fig. 5 shows an example of the results of finite element analysis on the model for the filled concrete part with neglecting the friction. Though the stress concentration occurs locally at the welded corner between rear bearing plate and cellular plate, the maximum stress in concrete is less than its allowable bearing stress. It shows that the most of the concentrated axial force induced from steel beam is uniformly dispersed to the filled concrete by rear bearing plate and shear connectors, and the stress concentration does not occur at the tip of steel plate, as shown in Fig. 5 (a).

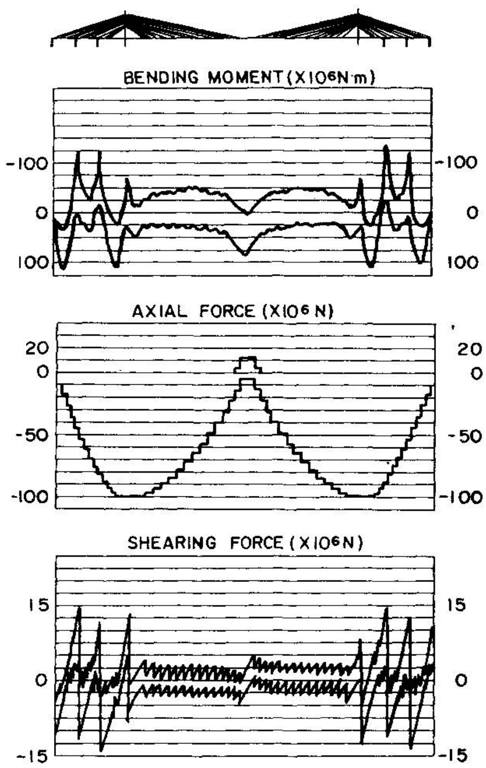

Fig. 4 Section forces

Category	Mzmin (X10⁶N m)	N (X10⁶ N)	Sy (X10⁶N)
2P side	−72.8	−90.3	− 8.2
3P side	−69.7	−90.6	8.1

Table 1 Section forces at the connected part
(At service loading condition)

(MPa)

Category			Top fiver	Bottom fiver
PC section	2P side		− 0.7	−7.0
	3P side		− 0.9	−8.8
Steel section	2P side		13.0	−155.8
	3P side		8.3	−152.0
Composite section	Steel plate	2P side	− 1.9	−38.5
		3P side	− 2.8	−37.2
	Filled concrete	2P side	− 0.3	−5.6
		3P side	− 0.4	−5.4

(Compressive stress: negative)

Table 2 Working stress at the connection part

Also, it is seen that the forces resisted by shear connectors reduce step by step according to a part from the bearing plate as shown in Fig. 5 (b). From these results, the transmission rate by headed studs was assumed to be 35 % and in the same manner that by block connectors was obtained to be 50 % by another result.
As the compressive forces are locally transferred to the prestressed concrete transverse beam through the filled concrete of the top and bottom flanges, the stress condition in concrete at connected part was examined in detail by finite element analysis. The following stresses were under consideration as shown in Fig. 6. ① Bearing stress at the root of the filled concrete ② Bursting stress at that point ③ Tearing stress in the front of the transverse beam. The reinforcing bars were arranged in proportion to the magnitude of these working stresses.
If the concrete of the transverse beam is in contact with the tip of steel plates, it may crush locally due to large stress concentration. Therefore the sealed joints were provided around the external periphery of steel girder.

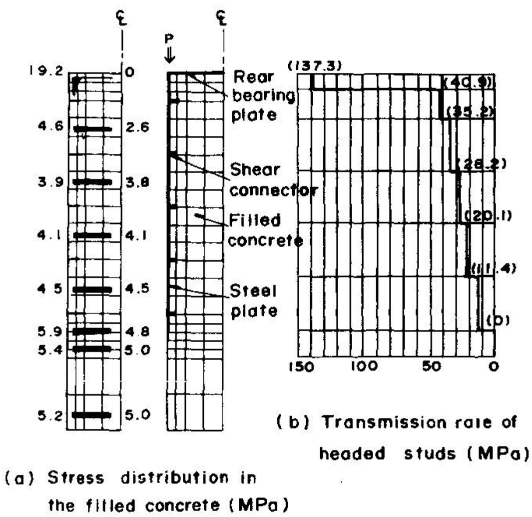

Fig. 5 Transmission mechanism of the filled concrete

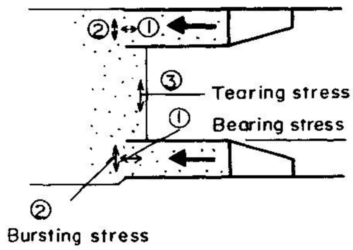

Fig. 6 Local stress at the connected part

3. CONSTRUCTION OF THE CONNECTION

Prestressed concrete girder was constructed by a cantilever erection with traveling forms and cast-in-place construction on false works, to complete 3-span continuous girders. The connection segment of the steel girder was temporarily fixed on the inclined bent as the construction of the prestressed concrete girder approaches the completion, after which the connection concrete was placed so as to minimize, if any, the effect of the construction errors of the prestressed concrete girder. Erection procedure is shown in Fig. 7.
The filled concrete inside the connection segment of the steel girder was placed in advance at the manufacturing of the girder. Non-shrinkage concrete was used considering the restraint condition of the concrete by the steel cells, and to reduce the effect on shear connectors due to bleeding.
Furthermore, the concrete was placed in such a way that the bearing area of the studs faces upward so as to avoid relative or residual slippages due to bleeding which otherwise may develop on the bearing area, and to facilitate the concrete placement.

4. STAY CABLE ANCHORAGES OF THE PRESTRESSED CONCRETE GIRDER

Each cable anchorage of the prestressed concrete girder in the side span consists of a thick steel bearing plate and a steel casing pipe. In the anchorage zone anchoring of a stay cable with the maximum design force of 5800 KN as well as transverse prestressing tendons with that of 4200 KN induces, in the concrete, considerable stresses due to bearing, bursting and tearing, which requires congested arrangement of reinforcing bars. The form of the reinforcing bars to be arranged behind a bearing plate was, to facilitate construction, a grid instead of a spiral which has been commonly used. Grid type of reinforcing bars improves the bearing capacity of the concrete by dispersion of the bearing pressure whereas spiral type of reinforcing bars does so by triaxially confining the concrete.
A static loading test, conducted using 1/3 scale model of the anchorage zone, showed no specific difference of properties in the stresses, deformations or cracks between the grid type and the spiral type.

5. CREEP AND SHRINKAGE

Changes in resultant forces due to creep and shrinkage of the concrete induced as the progress of construction of the prestressed concrete girder and following cantilever erection of the steel girder after its completion were calculated by a plane-frame analysis, allocating appropriate values of coefficients of creep and shrinkage to each construction segment.
The most remarkable result shown by the analysis was 50 mm of toppling of the tower toward the center span and 70 mm of downward deflection of the steel girder resulting from axial shortening, which is also 50 mm, of the prestressed concrete girder in the side span due to the creep and shrinkage.

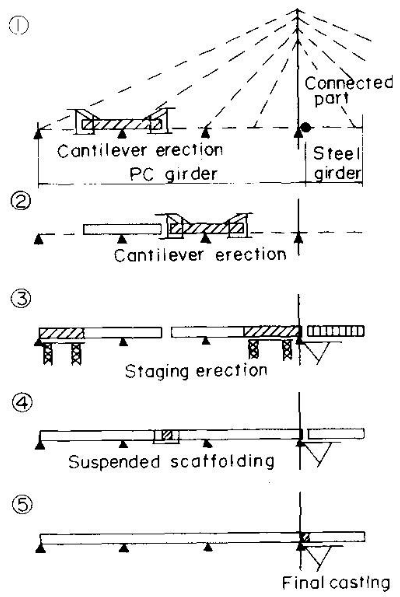

Fig. 7 Constructing sequence of the connected part

6. AERODYNAMIC STABILITY DURING CONSTRUCTION

The aerodynamic stability during construction was investigated by wind tunnel tests using an elastic model of 1/65 scale. Not only free-standing of the tower but also just before the girder closure was examined. The test results are as follows :
(1) It was determined that the liquid-type damper, so-called Tuned Sloshing Damper, would be installed at the top of the tower while free-standing to suppress vortex-induced oscillation at the wind speed of about 10 m/s. The sloshing phenomena of water is applied using a rectangular tank by the size of 5m x 0.8m x 1.4m (length, width and height, respectively) with the water depth about 0.8m.
(2) It was observed that there was no oscillation in a smooth flow, however a buffeting oscillation was evoked in a turbulent flow during the erection of the center span girder. This vertical bending oscillation was judged to be not harmful to the structural safety, because the maximum amplitude at design wind speed (i. e., 41m/s) was only 120mm.

7. CONCLUDING REMARK

The construction of both PC girders in side spans has been already finished, and now the erection of steel girder in center span is under construction by the cantilever method. The design of Ikuchi Bridge has been proceeded under the guidance of the Technical Committee and the Mixed Structure Committee of Honshu-Shikoku Bridge Authority.

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