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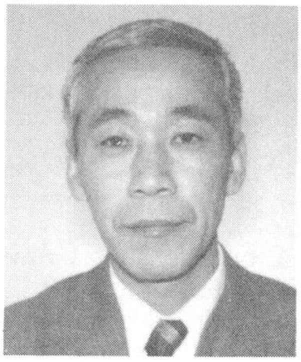
Erection of the Tatara Bridge's Superstructure



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Summary

The Tatara Bridge (hereafter referred to as 'the Bridge') is located at the middle of the Onomichi-Imabari Route, the most westerly route among the three routes of the Honshu-Shikoku Bridge Project. It is a cable-stayed bridge connecting Ikuchijima Island (Hiroshima Prefecture) and Ohmishima Island (Ehime Prefecture), and the Bridge has a total length of 1,480 meters and a center span length of 890 meters. When completed, it will be the world's longest bridge in terms of scale, excelling its sister bridge Normandie Bridge (located in France with a center span length of 856 meters).
This paper reports on the design and construction of the superstructure of the Tatara Bridge.

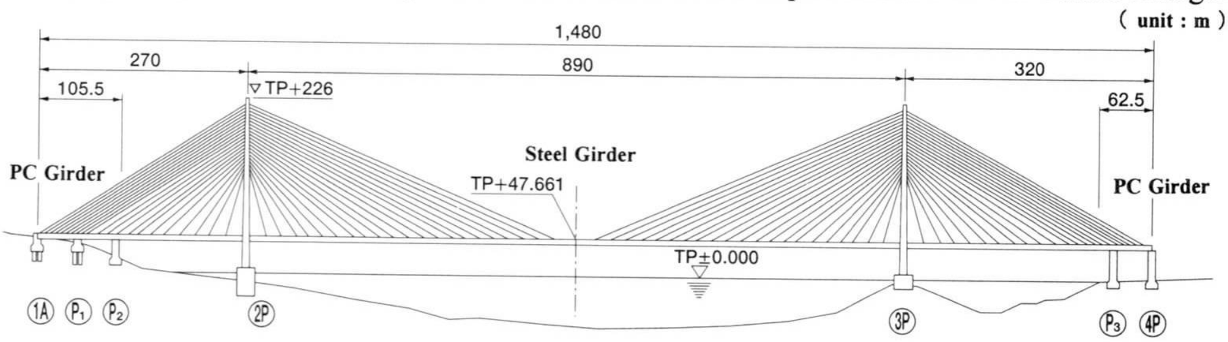

Fig. 1 General View of the Tatara Bridge

1. Design of the Bridge

In the initial design, the Bridge was planned to be a suspension bridge. Under the plan, great impact of topography was anticipated because the anchorage point on the Ikuchijima Island side was proximal to the mountain ridge at its back. However, a cable-stayed bridge design was eventually selected because it does not need anchorage, resulting in less topographical influence. Furthermore, we could assure sufficient design and installation, with subsequent technological advances, and the fact that a cable-stayed bridge was no more costly than a suspension bridge was in our favor.

Since the center span of the Bridge is too long compared with the side span, countermeasures against negative reaction occurring at the ends of the side spans were necessary as was the enhancement of overall rigidity. Prestressed concrete girders were placed at the ends of each side spans (for 1A-side: 105.5m, for 4P-side: 62.5m), and a steel girder was adopted for the entire remaining parts of the bridge to make it a composite cable-stayed bridge.

Table 1 Volume

Tower	Tower	12,110 t
	Accessory Facilities	450 t
	Anchor Frame	480 t
Girder	Steel Girder	15,860 t
	PC Girder	6,610 m3
	Accessory Facilities	450 t
Cable	Cable	3,640 t
	Socket	310 t

The cable system is double-plane with a multi-fan of 21 rows of cables. The girder is a flat box type, and its height and width are 2.7 meters and 30.6 meters respectively. Bicycle and pedestrian ways of 2.5 meters in width are extended on both sides of the 4-lane roadways of 20.0 meters in width.

2. Designing the superstructure

2.1 The tower

The tower shape was initially designed to be A-shape. However, harmful out-plane vibration occurred not only when the tower stood alone but also at time of completion of the bridge. Vibration characteristics and mechanism of vibration were analyzed, and the reverse Y-shaped tower was adopted because of its improved aerodynamic properties and aesthetic appearance. As a result of many wind tunnel tests, the corner-cut shape was adopted to the tower section for reducing the amplitude of the vortex-induced oscillations. Cross section measurements are 12 ~ 5.6 m × 8.5 ~ 5.9m, and the tower is the largest class ever of monocell structures.

The overall height of the tower is 220 meters and it consists of blocks divided into 23 levels in height. High-strength friction-grip bolts were adopted to all of the connection in the tower. Two vertical girders were installed in the tower shafts and cast anchor blocks were secured with the bolts to the girders, to which structure the cables are to be anchored.

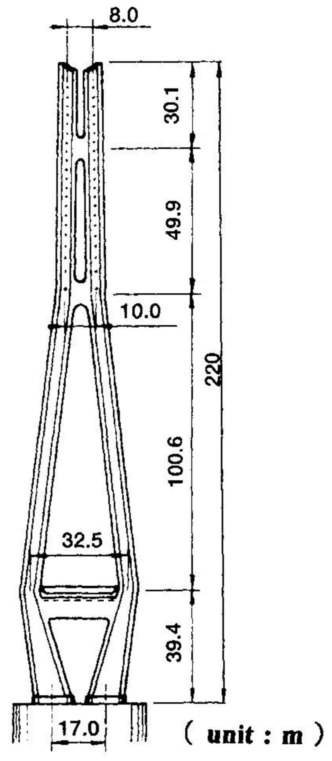

Fig. 2 Front View of Tower

2.2 The cables

Galvanized steel wires of 7mm in diameter were bundled in factories in a semi-parallel form with slight torsion. Then, they were directly coated with extruded high-density polyethylene. With long-spanning, the natural frequency of the cables is remarkably lower compared with those of another cable-stayed bridges (the longest cable: outside diameter 170 mm[ϕ 7 × 379], cable length ≒ 460 meters, natural frequency = 0.225 Hz). For this reason, the properties of rain vibration in the low vibration area were investigated by a wind tunnel test using full-scale cables, and so were the effects of anti-vibration measures by applying incremental damping addition and aerodynamic-oriented cross section. Indent application to the surface of cables in a discrete manner serves as a valid measure since it enhances aerodynamic stability of the cables without substantially exceeding drag force coefficients of conventional cylindrical cables.

2.3 The girder

As for the steel girders, box girder cross sections including fairing were selected for securing aerodynamic stability based on wind tunnel tests. The girders are wide and long flat 3-chamber deck slabs. The girder height is 2.7 meters, and the beam span depth ratio is about 1/330, being of remarkably slender shape.

As a result, the stiffness of the girder was relatively low and axial compressive stress was dominant for determination of the area. Therefore, trough ribs for deck slabs and lower flanges were used, and flat ribs were used for webs. Thus, all of them were designed as compressive stiffened plates.

In consideration of aesthetic and maintenance aspects, the cable-fixing parts were stored inside the fairing so that the fixing structure is not exposed outside of the girders.

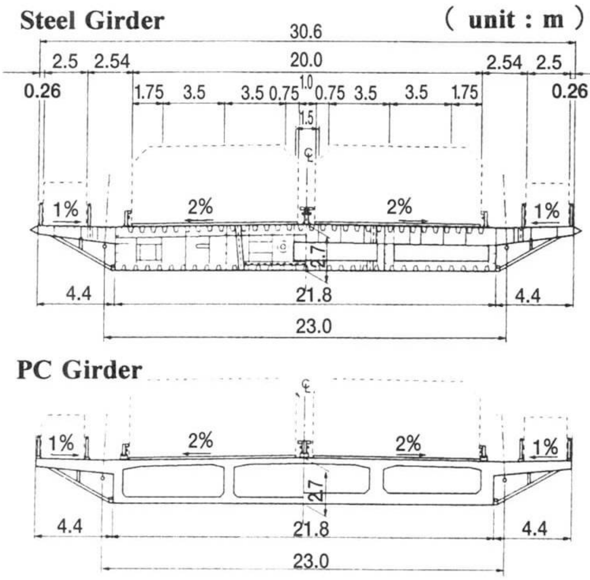

Fig. 3 Cross Section of Girder

In the steel girder manufacture, taking into account that the steel girder is designed as a compressive stiffened plate, careful sizing of members was carried out to minimize initial strain on deck slabs and lower flanges to consequently avoid deterioration in ultimate buckling strength. Based on the results of various fatigue tests of the deck slabs, the following measures were taken: specifying penetration amount of welding trough ribs to deck slabs (about 80% of board thickness), backfilling of scallops of lateral beam webs that trough ribs are installed through, reducing scallop dimensions of the reverse welding part of deck slabs (30 × 75 mm), grinder finishing of close welding which is right below the wheels, etc.

The support condition of the girder is as follows: with an aim to control dispersion of horizontal force and excessive displacement in the longitudinal direction according to earthquakes, elastic support was adopted for the bearing at the tower, and movable bearings were adopted for the other supports. For the elastic support method at the tower part, what was adopted is horizontal shear spring bearing support using non-damping type rubber, which also serves as vertical reaction bearing. The spring value was set at 4,000 t/m/Br.

3. Erection of the superstructure

3.1 Outline of the erection

Erection procedures of the superstructure are shown in Fig. 4 .

The tower is divided into a total of 23 blocks. First, base plates (w ≒ 120t × 2), the first blocks of the tower (w ≒ 240t × 2) and large block of the lower part of the tower (w ≒ 1,500t), were erected by floating cranes. Following, the large block of the steel girder near the tower (2P-side: l=123m, w ≒ 2,000t,

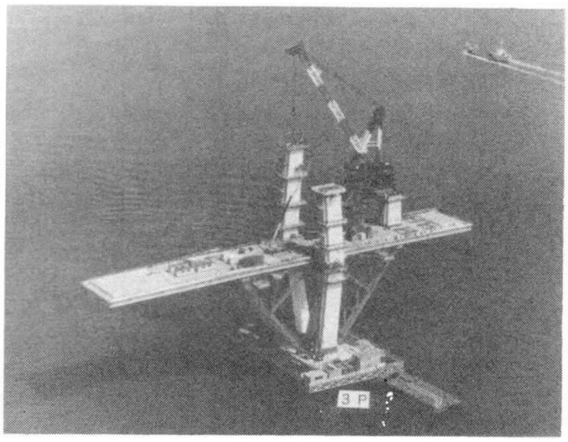

Photo. 1 Erection by Tower Crane

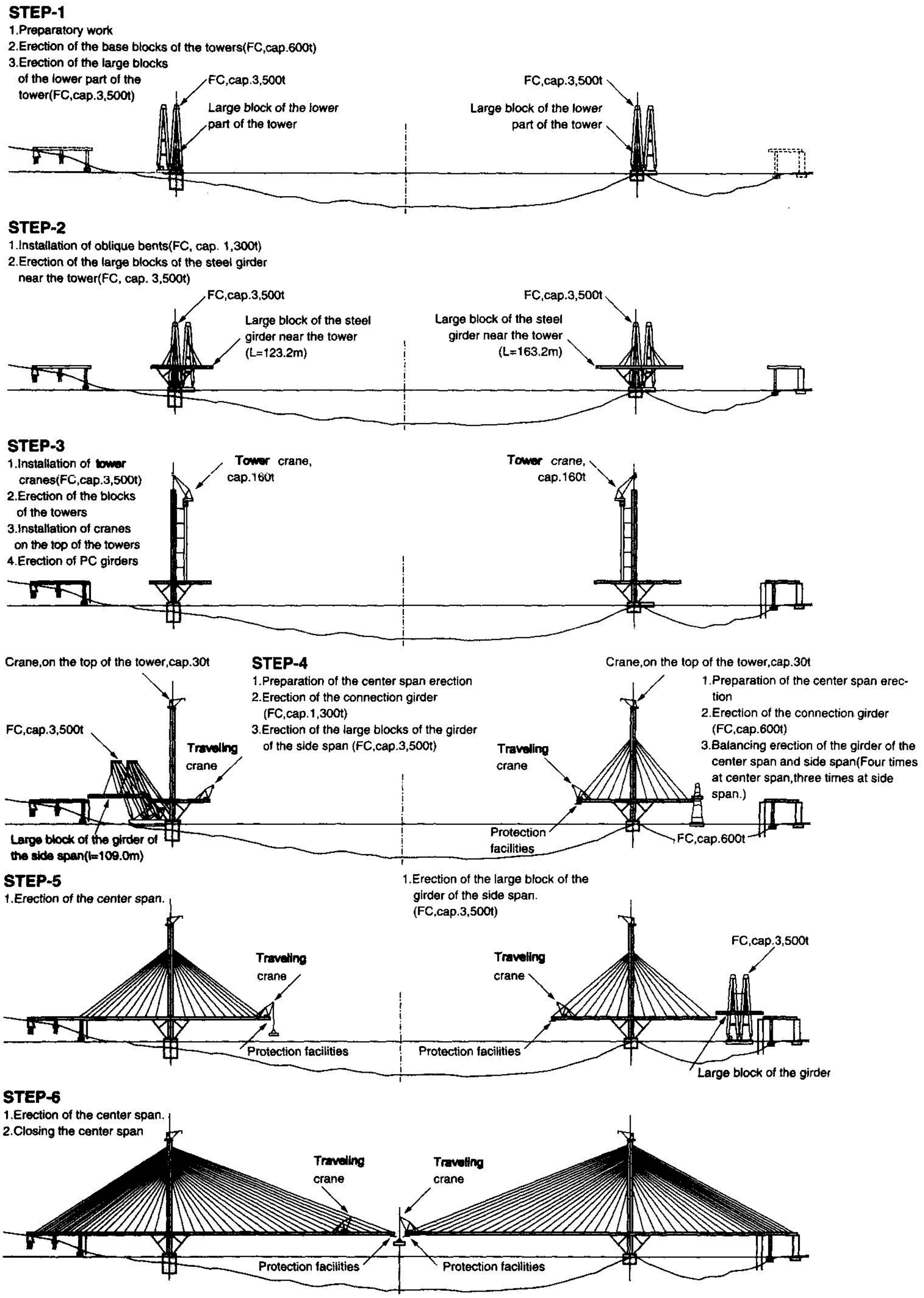

Fig. 4 Erection procedures of the Superstructure

3P-side: l=163m, w ≒ 2,500t) was erected by a floating crane (cap.3,600t). Making use of this large block as the work yard, upper blocks of the tower were erected one by one with a tower crane (lifting cap. 160t) set on the girder. The erection precision of the tower (slant quantity of the tower) was high at the level of around 1/7,000 which was made possible by sufficient manufacturing precision control at factories and careful erecting operations on the site. It successfully met the required erection precision (1/2,000 of the tower height).

The erection unit of the girder can be divided into: the large block deck near the tower, the large block deck at the side spans, and the ordinary block deck (l=20m, w ≒ 300t). Since the side span of the 2P-side is shorter than the 3P-side, large block deck (l=109m, w ≒ 1,800t) is erected at the side span by the floating crane (cap. 3,600t) immediately after the completion of the tower erection. The side span of the 3P-side is longer than the 2P-side, so it was not possible to erect the remaining side span parts as a single unit of the large block deck. While adjusting the balance between the center span and the side span, four blocks were erected at the center span and three blocks at the side span in the following way. Ordinary block deck girders were erected by the traveling crane (cap. 350t) on the girder for the center span, and the floating crane (cap. 600t) was used for the side span. The remaining side span was erected as a large block deck (l=102m, w ≒ 1,500t) by a floating crane (cap. 3,600t).

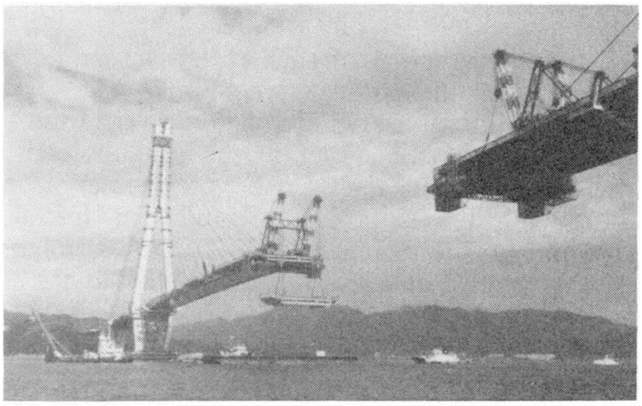

Photo. 2 Erection of Steel Girder

After completing the girder erection of the side span. At the center span part, the cantilever girder erection of ordinary block and cable erection using the traveling crane (lifting cap. 350t) placed on the girder and a truck crane (cap. 360t), were carried out as one process. This process was carried out 18 times for the 2P-side and 15 times for the 3P-side.

3.2 Remarkable features of the erection

Major characteristics of the superstructure erection of the Bridge are: cantilever method of the center span reaching as long as approximately 435 meters maximum due to the center span length of 890 meters, and erection of the girder-suspending cables with the maximum length of 460 metersr.

Regarding the cantilever erection, topographical constraint made it impossible to install temporary bent equipment. Therefore erection was carried out by completely employing the cantilever system. Prior to adopting this method, safety at the time of completion and erection was confirmed not only by examining numerical analyses but also by wind tunnel tests using a whole bridge model on a scale of 1/70.

During the erection of the girder outrigger (in 1997), as many as two typhoons hit the site (Typhoon eight at the end of June, and Typhoon nine at the end of July). Typhoon nine, which came immediately before the closure, especially worried us. However, no damage was caused at all, and the closure was able to be completed, thanks to previously planned countermeasures against typhoons. The countermeasures were: firm binding as anti-vibration measures of cables, retracting the traveling crane to lessen the projected area normal to wind direction, and rolling up the safety nets of the railing. The wind velocity of Typhoon nine was about 25m/sec at the maximum instantaneous wind speed. The relative displacement of the girder at this time was around 50cm.

With regard to the cable erection, pulling-in and anchoring at the girder side were first carried out after anchoring at the tower side. At anchorage fitting at the tower side, the highest cable reaches 170 meters above the girder, and the dead load alone is as heavy as 25t. The capacity of a tower-top crane which was no lift the cables needed to be upgraded, and a crane with a lifting capacity of 30t was necessitated. In pulling-in and anchoring efforts at the girder side, the maximum pulling-in length to the predetermined anchorage point reached about 150 meters. The pulling-in force at this time was as great as 800t maximum. Therefore, pulling-in of the cables to the girder side was divided into three phases: the first pulling-in was done with a winch (pulling-in cap. 30t), the second pulling-in with a wire clamp (cap. 70t), and the third with a center-hole jack (cap. 800t). Thus, cable pulling-in operations were carried out depending on the capacity of each equipment. The tension rod, at the highest cable (l=460m, w ≒ 60t) employed for the third pulling-in operation with the use of the center-hole jack, was ϕ 180mm × 8.5m.

Lastly, field observation of each part has been performed to primarily investigate the impact of winds during the construction of the superstructure of the Bridge. The following were executed: vibration observation of the tower at the time of erection, and vibration observation of the cables and the girder at the time of cantilever erection. After the girder closure limiting the duration up to one year, vibration observation of the cables and girders is on-going with the emphasis on study on spatial correlation of winds and the vibration observation of the cables. The spatial correlation has been conducted with ultrasonic aerovanes installed at five different points that are on longitudinal direction.

4. Postscript

The Bridge venture was decided on in 1989, and the construction ceremony was held in August of the following year, followed by starting the substructure construction in November 1992. The construction moved on to submarine drilling of the tower base, installing caissons, driving installation of underwater concrete and atmospheric concrete. In March 1995, the final driving installation of concrete was carried out, and the construction of the tower base (2P, 3P) was completed. Regarding the superstructure construction, the tower construction was initiated in August 1995. In June 1996, the erection of the final (23^{rd}) block was completed. After a period of preparatory works for erecting girders, erection of an outrigger between the center spans was started. In the summer of 1997, after a lapse of nine years since the venture was launched, the closure of the girder with a center span of 890 meters was observed.

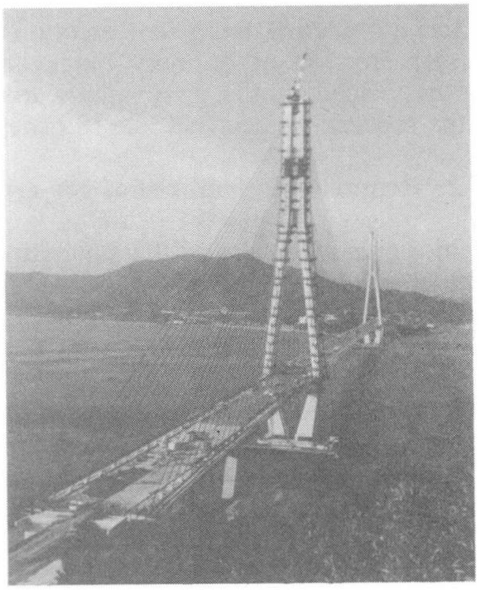

Photo. 3 Tatara Bridge

Lastly, we would like to mention that the superstructure of the Tatara Bridge is progressing without any accidents occurring since the start of construction. We would like to express our gratitude to the efforts of those concerned with the construction.