PLANNING AND DESIGN OF THE SHIN-MEISEI BRIDGE

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Summary

The Shin-Meisei Bridge is an extradossed prestressed concrete bridge with a length 294.32m and a main span 122.34m. The effective width varies from 18.6m to 22.6m and the height of the girder is constant at 3.50m.

This paper presents the main characteristics of the design and construction of this bridge and the main technical features of the bridge such as cross-section, pylons, anti-seismic design and construction.

Keywords: extradossed, precast segment, composite structure

1. Introduction

The Shin-Meisei Bridge is located on the No. 3 Nagoya expressway, which connects the Nagoya city center with the east Nagoya-Osaka road and the Ichinomiya interchange of the Meishin expressway. As the bridge crosses over the Shonai River at a high level and is parallel to the national road No. 22, the architectural design was an important aspect and was taken into consideration so as to avoid any feeling of oppression.

Among several steel or concrete structure alternatives selected, four types were studied in more detail and a three-span extradossed prestressed concrete bridge was finally chosen.

The design of this bridge was aimed at harmonizing the bridge with the surrounding area, give an impression of rhythm, lightness and opening from the national road 22, and reduce the size of the foundations, as they are adjacent to those of the existing bridge. In order to use an extradossed prestressed concrete bridge, it was so necessary to reduce the weight of the superstructure cross-section.

This bridge contains many innovations in comparison with other similar bridges build in Japan such as a trapezoidal-shaped three-box cross-section comprising external webs with a low slope and internal webs sloped towards the cables anchorage, a precast concrete core placed in cantilever to realize the side span’s extremities without scaffolding, the lateral parts being built by a form traveler, the use of composite steel-concrete solution for the main tower at the top zone where the anchors of stay-cables are located, could reduce the required erection equipment and improve the facility of work for the use of FUT-H cable stay system.

Figure 1 - Panoramic view of the bridge

This paper presents the construction of the Shin-Meisei Bridge with the impact of these innovations.

2. Conditions of design

The three-span continuous extradossed prestressed concrete bridge has a total length of 294.32m with the spans of 89.63m + 122.34m + 82.35m. The effective width varies from 18.6m to 22.6m.

The longitudinal slope varies from 2.4% to -0.675% and, due to a horizontal alignment varying from a straight line to a curvature with radius 420m, the transverse slope varies from 2.0% to 5.0%.

The river flow is Q = 4200m^3/s and the execution period without flooding extend from October to May. As the rate of waterbed obstruction should be less than 7% including the existing bridge, the total width of one pier should be smaller than 4.0m.

The soils characteristics at bridge site are from top to bottom: a cohesive and alluvial sandy soil to depth 15m, a sandy, gravel soil to depth 45m and a sandy, becoming gravel soil under 45m. The resisting soil could be, at a depth greater than 25m, the firm sandy soil.

The seismic design of the bridge had to be carried out with a non-linear dynamic analysis. The seismic zone is A-area, which corresponds to the most aggressive seismic action. This region covers a great part of Japan including the biggest cities. The priority classification is B, which corresponds to the most important safety coefficient against seismic action. The classification of the soil for earthquake resistance is Soil type III, which correspond to worse soil, and the earthquake scale is level I (frequent occurrence of earthquake but small level) with a equivalent static acceleration 0.3 g.

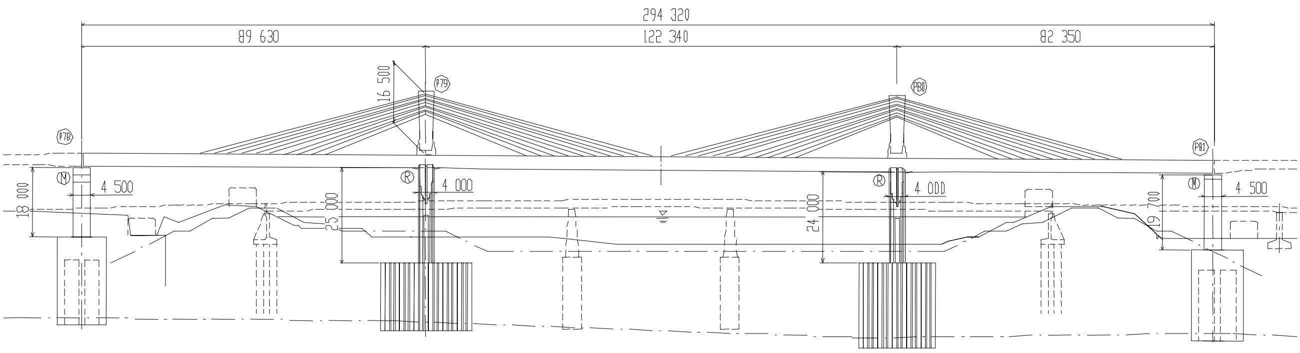
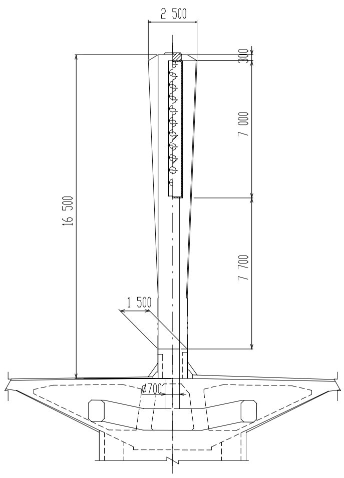


Figure 2 – General arrangement of the bridge



With the previously mentioned conditions, the main design considerations were for the environmental and economical aspect, as some bridge parts should be inside the river and as the bridge are close to the main national road No. 22.

The influence of the works adjacent to the National road No.22 Bridge and, during the works, the negative influence on this road traffic should be taken into account. In addition, architectural aspect should be carefully studied to reduce the feeling of oppression and heavy weight from the national road No.22.

3. Study of the extradossed bridge

3.1. Decision of span length

Due to the river condition, the location of the piers should be grouped with those of the existing bridge and a three-span continuous extradossed bridge with a main span of 122.34m was designed; in addition, for a better seismic resistance, the box-girder has been rigidly connected to piers below and pylon above.

3.2. Bearing conditions

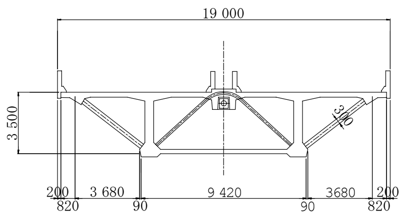
A comparison of the conditions of connection between pylon, girder and pier has been carried out. 3 alternatives were studied: rigid-frame structure, floating type and continuous girder on rubber bearings.

Although the forces in the pylons are smaller for continuous girder on rubber bearings, cost of the bearings shoes becomes larger and greatly reduces the advantage of this alternative. Furthermore, the seismic resistance of rigid-frame structures, its economy and structural aspect are excellent and this structural type appeared as being the most appropriate solution for the Shin-meisei Bridge.

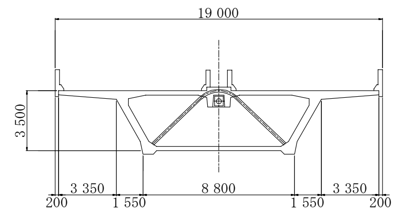
3.3. Cross-section of main beam

The height of extradossed bridge beam is generally variable with normal ratio 1/35 of span length on support and 1/55 at midspan. In the case of the Shin-Meisei Bridge, the height of the beam has been examined for several variants and, as a result, a height of 3.50m appeared to be advantageous for economical and aesthetic reasons.

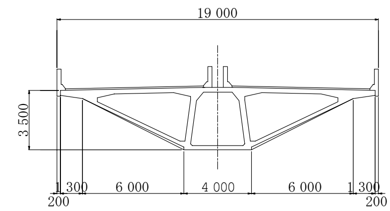
In designing the cross-section, one aim was to increase the efficiency of the force transmission between the beam and the inclined cables. With an axial cable-stay layer, all the forces should be transmitted transversely to the center of the beam and the three cross-section alternatives presented on figure 3 were analyzed.



a) Internal and external struts box - Cross-section area 12.62m^2



b) Internal struts box - Cross-section area 13.61m^2



c) 3-cellular box - Cross-section area 12.76m^2

Figure 3 – Cross-section shape alternatives

In the case a), steel rod struts (ϕ 160mm) at the center of the cross-section and steel pipe struts supporting the cantilever slab have been designed to transmit the stay-cables force to the lower slab, allowing the reduction of weight. The cross-section of the box beam and the position of the struts being fixed, the change of width would be done on the cantilever parts.

In the case b), struts are placed in one plane at the center part of the cross-section to minimize the weight, but inclined webs were preferred at struts for the cantilever. The traveler form would be more difficult for the construction with the widening of the box beam cross-section.

In the case c), the use of walls for the central core of the cross-section which should resist the tensile force of the stay-cables, with three cells of the cross-section and the use of 20cm thin inclined webs allowed the three-cellular girder cross-section to have a higher torsional rigidity. It has been chosen as the final cross-section. Additional ribs were added to support the form traveler loads during the construction period, reinforce the inclined webs and accent the tensile force at the rising of the inclined webs.

The lower slab, external web and internal web axes are concurrent, and the use of frame walls is not necessary to transmit the load between the girder and the stay-cables.

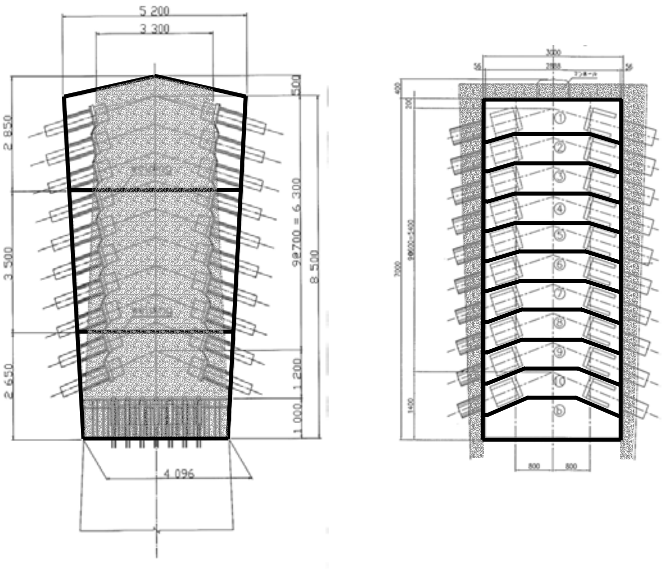
The cross-section of the box beam (lower slab defined) being fixed, the change of width would be done on the cantilever part.

3.4. Pylon

The normal height of the pylon for prestressed concrete extradossed bridge is 1/10 of the main span, but in the case of this bridge as the width of the deck is large, the height 1/8 of the main span, say 16m, has been also chosen and appeared to be more advantageous.

A comparison has also been done for the stay-cable spacing between one cable per each block, 3.60m, and one cable every two blocks, 7.20m. As the width of the main beam is 25m, the weight of the beam is large and the spacing 3.60m, corresponding to 10 stay-cable units, appeared to be advantageous.

The width of the central band is narrow, 2.0m, and the pylon should be inserted inside this band. The design of the head of the pylon, supporting the cables, should be aimed to reduce their width and the two alternatives presented on figure 4 have been compared. A saddle alternative was studied in addition, but the dynamic analysis under seismic motion shown that the bonding of cables is deficient in case of earthquake and this alternative should be excluded.



a) 3 steel units type pylon b) 11 steel units type pylon

Figure 4 – Composite steel-concrete pylon head alternatives

In the case a), the anchorages of the stay-cables are placed inside the steel box of a steel-concrete composite structure. With the use of a steel box for the anchorage of the stay-cables, this alternative solution is safe but many reinforcement bars should be used for the cross-section to resist the heavy bending moment.

As the steel shell is cut in three blocks (15t/block), the welding of the blocks is difficult due to the heavy weight and the thickness of the plates. As the crane should be located far from the pylon, heavy lifting equipment was needed with this alternative.

In the case b), the anchorages of the stay-cables are also placed inside the steel box of a steel-concrete composite structure, but the steel shell is cut in eleven blocks placed without welding.

No heavy equipment is required for construction as the weight of each unit could be adapted to the lifting capacity. With the use of a steel box for the anchorage of the stay-cables, this alternative is seemed as safe. High-strength-steel should be used for the cross-section to resist the heavy bending moment.

For both cases, as we have to place many stay-cable anchorages in a narrow space, the works are difficult, but the box shape allows the replacement of stay-cables.

As a result of the study, we chose the alternative type b with eleven steel units. The design of the steel concrete composite structure was done with a transmission of the vertical load of the stay-cables to the concrete by studs and by designing the steel only to support the horizontal force of the stay-cables. With these concepts, a comparison with the traditional method, where only the steel shell supports the forces of the stay-cables, showed that the reduction of the steel weight could be important.

As it was impossible to use heavy lifting equipment for the construction of the steel shell members, it was divided in eleven units, each unit having a weight smaller than 50 kN. Without welding between the units, the geometrical tolerances for the production of each unit were very small and the main difficulties for the welding of each unit was to avoid the deformations over the full width during construction, guarantee the quality of the welding and avoid thermal stresses, and guarantee the vertical precision of the pylon. For these reasons, a machine cutting manufacturing and a metal touch process was used in the factory for the production of each unit.

A non-linear FEM analysis was carried out and confirmed that stresses in the steel shell members under the load of the stay-cables tension were lower than the allowable stresses.

3.5. Stay-cables

The FUT-H cable-stay system manufactured by SE Corporation was used for the stay-cables of the extradossed bridge. This system is made of individual 15.6mm diameter galvanized prestressing strands, injected with grease and coated in HDPE. The strands are then located within a Fiber Reinforced Polymer (FRP) sheath to form the stay-cable. Each stay-cable comprises 37 or 43 prestressing strands, but the strand-by-strand installation technique of this system allows the erection without any special and heavy equipment.

By using the two first strands as "pilot strands", the process allows to be sure to have the same calculated tension force in each individual strand. The FRP sheath is installed on the deck with the two pilot strands inside and hoisted up. The following strands are pushed through a guide-pipe toward the pylon by using a belt-type pushing machine on the deck. The hoisted strands pass through the FRP sheath through the deck and are tensioned by a monostrand jack.

At the end of erection of the bridge, the tension of cable-stays is adjusted strand-by-strand. Streamlined sheaths for the pylon and deck are mounted as soon as the stay-cables are in position, and every anchorage is injected with anti-corrosion material. Damping devices will be set on each stay cable in order to damp the vibration.

3.6. Piers

The width of the piers located in the river was limited at 4.0m to limit the river obstruction. As this width doesn't correspond with the Japanese standard, the use of reinforcement steel SD350 (yield strength 350 N/mm2) was impossible and both SRC structure (Concrete structure reinforced by H-shaped steel section) and high-strength-steel reinforcing bars SD490 (yield strength 490 N/mm2) alternatives were compared.

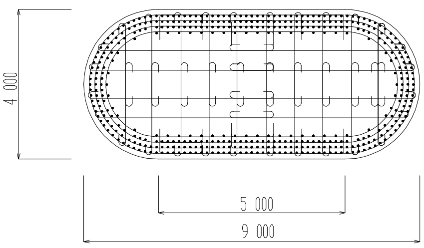


Figure 5 – Pier Cross-Section : High-strength-steel SD490 reinforcing bars structure

In case of the SRC structure, H-shaped steel sections (632×317×21×42) are used with concrete of characteristic compressive strength 40N/mm^2. Despite the H-shaped steel sections, the top of the column present a problem of insufficient strength. As H-shaped steel section as well as reinforcement bars are necessary, the construction of this alternative is difficult.

In case of the high-strength-steel reinforcing bars alternative shown on Figure 5, 4 layers of SD490 high-strength-steel rebar Ø51mm are used. A study of this alternative for seismic loads shows that the top of the column is safe. Even if a large reinforcement is necessary, this alternative is better than the previous one and was chosen as result of the comparison.

In addition to the theoretical study, a test on model of the high-strength-steel rebar top of column and beam connection was carried out to test the safety of the anchorage zone under alternative loading.

3.7. Foundations

As the loads applied on the river piers are large, they should have enough strength and bearing capacity. In addition, the new and the existing bridge are adjacent and some problems could appear. We decided to choose a foundation system reducing the interaction. A foundation caisson was clearly impossible with the adjacent existing foundations and the selected alternative was a steel-sheet-pipe foundation.

As the foundations of the end piers are close to the river banks, and as we have to guarantee the space for the management and road passage, we selected a continuous underground wall type foundation.

3.8. Seismic dynamic response analysis

In regard with the seismic analysis in Japan, it is necessary to check the safety of the structure with a level I and level II earthquake. The level I input motion corresponds to a large-scale plate boundary earthquake representing the earthquake motion observed in the Tokyo area during the Great Kanto Earthquake of 1923. The level II input motion is based on large acceleration records obtained at ground surface level during the 1995 Hyogo-ken Nanbu Earthquake. Three records of each earthquake level were used as input earthquake motion.

In this text, we present the critical points of the non-linear dynamic response analysis. We evaluated the non-linear time-history response of the structure taking in account the non-linearity of pylons, piers and girder. The elements were transversally modeled by fibers taking in account the materials non-linearity.

Element	Value
Pylon	3.0%
Girder	3.0%
Cables	3.0%
Piers	2.0%
Foundations	20.0%

Table 1 – Damping coefficients

The model is a plane-frame model with the damping coefficients for the various part of the model given on table 1 and for the initial value the permanent loads. For the first and forth modes, a Rayleigh modal damping coefficient was adopted in accordance to the Japanese regulation.

As the girder is made of prestressed concrete, the largest response moment should not exceed the yield bending moment of the cross-section. Large bending moments appear in the pylons under the level II earthquake and high-strength-steel reinforcing bars SD490 were required for the girder.

3.9. Construction method of side spans and analytical method

For the construction of the side spans, as it was impossible to use temporary supports due to limited conditions of river bank and as a construction by the cantilever method would have produced high bending moments and stresses in the box-girder and in the pylon, it was decided to erect a central core by the cantilever method from precast segments, and the remaining part of the cross-section has been cast in-situ with a form traveler.

The side span is thus made of a core part and of adjacent parts connected at different stages of construction. The concrete ages are different, while the center of gravity of the main girder section varies between the core part erection and the full section.

The model and the analysis of the side spans take into account these variations of the center of gravity and the difference of concrete age.

4. Conclusion

The extradossed girder construction started on March 2003 and is getting up to speed. Even in the case where the conditions of construction site and of execution presented many limitations, it was possible to resolve these difficult conditions and to construct this large-scale structure without using large-scale equipment.

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