A 3-span Continuous Box Girder Bridge with a Maximum Span Length of 173 m — Egawa Bridge —

最大支間長173m を有する 3 径間連続箱桁橋 — 江川大橋 —




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Synopsis[1]

As part of the Koishiwaragawa Dam construction project, the Egawa Bridge is constructed on the new route for National Route 500 located at the center of Fukuoka Prefecture and spanning over the Egawa Dam Lake (**Fig. 1**). It is a 3-span continuous prestressed concrete (PC) box girder bridge with two piers supporting a 173-m span crossing the dam lake. A spread foundation and a pneumatic caisson foundation are used for piers P1 and P2, respectively.
National Route 500 is the access road between Asakura City and Toho Village, which had been cut off by the dam construction project that takes place in these areas. Early opening of the bridge was mandatory to meet the high demand of local residents using bypass roads as alternative route; thus, the authors reviewed and revised the structure and construction method of both the bridge substructure and superstructure to shorten the overall construction period.

Fig. 1 Egawa Bridge

Structural Data

Structure: *3-span continuous PC box girder bridge*
Bridge Length: 339.0 m
Span: 87 m ＋ 173 m ＋ 79 m
Width: 8.2 m
Owner: Japan Water Agency
Designer: Chodai Co., Ltd.
Contractor: Sumitomo Mitsui Construction Co., Ltd.
Construction Period: Sep. 2016 – Mar. 2020
Location: Fukuoka Prefecture, Japan

1. Introduction

Fig. 2 shows general views of the entire bridge. The most significant feature of this bridge is its central span length of 173 m, which is longer than the 170 m of the Heigen Bridge, which until then was the longest continuous PC box-girder bridge in Japan. It also has a maximum girder depth of 10 m. For its foundations, P1 is a spread foundation while P2 is a pneumatic caisson, and the bridge piers are made of solid reinforced concrete cross sections. To shorten the construction period, the P2 backfill was changed to lightweight embankment, and prefabricated reinforcing bars and headed bars were used for the substructure.

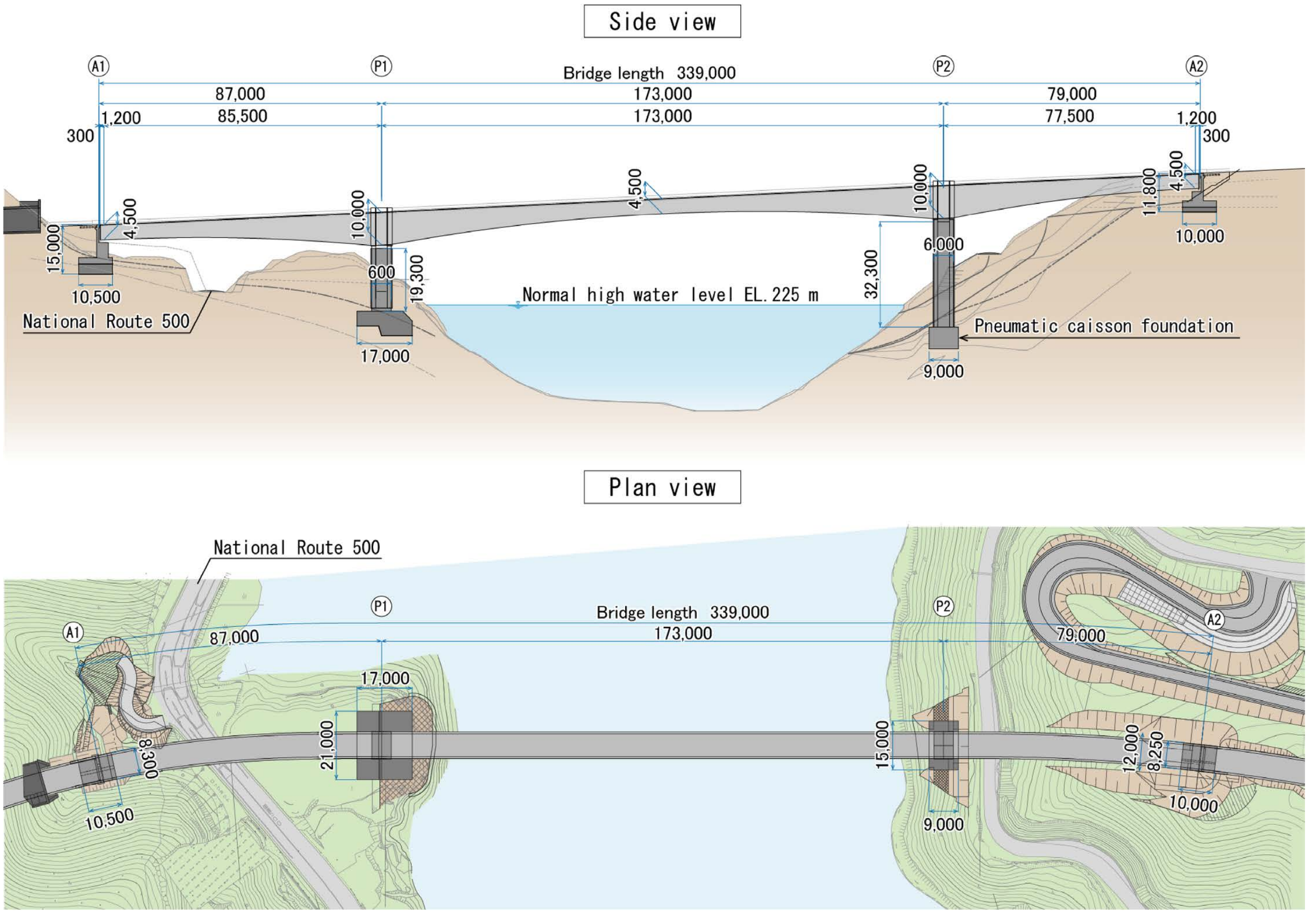


Fig. 2 General views of Egawa bridge

For the superstructure, the internal cables were changed to high-strength prestressing steel and some of the continuous cables were changed to external cables, thereby changing from an internal cable system to a combination of internal and external cable system. These modifications allowed the pneumatic caisson length to be reduced from 18.0 m to 11.0 m. In addition, the superstructure was slimmed down, reducing its weight by approximately 10%.

2. Design

(1) Design Study for Shortening the Construction Period

In order to shorten the construction period, trial design was conducted with a view to structural modification of the substructure and superstructure. The pneumatic caisson foundation work and cantilever construction work on the superstructure covered a large part of the construction period, accounting for a third and a quarter of the total, respectively.

Design optimization of the pneumatic caisson foundation and superstructure was conducted with a view to reducing the duration of the overall construction schedule by shortening the construction period of these structures. The results of design optimization is described in detail below.

(2) Foundation and Pier Design

The length of the pneumatic caisson foundation was initially 18.0 m, but it was reduced to 11.0 m to shorten the construction period. Also, it was planned to backfill the back of pier P2 with a reinforced embankment for landscaping purposes after completion of the pier. Therefore, the pneumatic caisson foundation of pier P2 was designed considering the effects of eccentric earth pressure in addition to superstructure reaction forces. To reduce the moment and lateral forces caused by eccentric earth pressure from the backfill, the pier backfilling was changed to a lightweight embankment using EPS (expanded polystyrene) construction (**Fig. 3**). In addition, the superstructure reaction force was reduced by optimizing the superstructure, as described below. Compared with the cross-sectional forces in the initial design, the moment acting on the pneumatic caisson foundation was reduced by about 66% and the lateral force by about 60%, and ultimately the caisson foundation length was reduced from 18.0 m to 11.0 m (**Table-1**).

Various other structural details were modified to save labor and increase efficiency in the construction. Since caisson excavation expected more time and labor to excavate especially at the corners, the efficiency of excavation work could be improved by rounding and chamfering the corners of the caisson cross section.

Furthermore, to simplify the rebar assembly work, the double J hooks lap splices rebars were replaced with headed bars. For the P1 spread foundation, the ends of the shear rebars, which were similarly lap splices, were changed to headed bars.
On the bridge piers, the main bar joints were changed from welded joints to mechanical joints, and tie hoop anchorage was also changed to headed bars, mainly to save labor in construction. Formwork and reinforcement work was reduced by changing the cross section of the pier from the original hollow cross section to a solid cross section (**Fig. 4**).

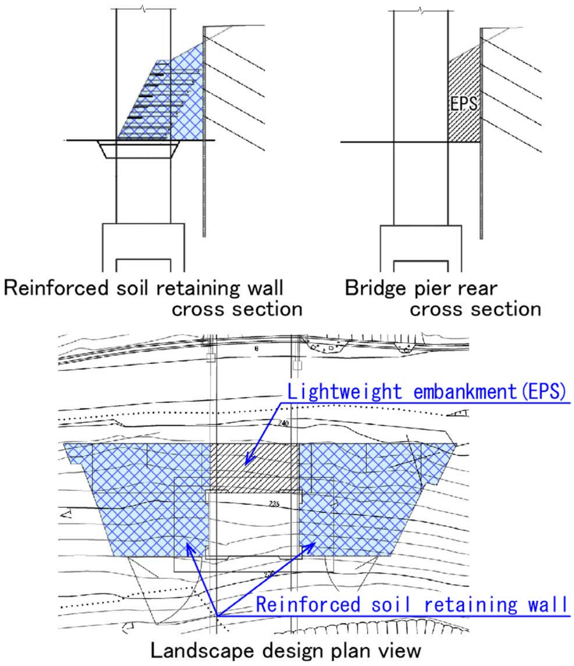
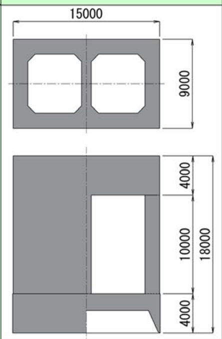
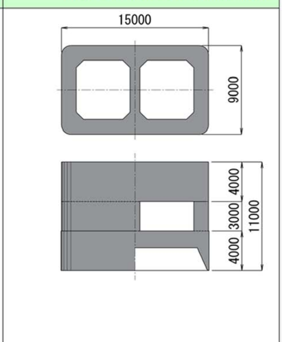


Fig. 3 Use of lightweight embankment

Table-1 Comparison of pneumatic caisson dimensions

		Initial design	Modified design (lightweight embankment +superstructure modification)
Schematic diagram			
Dimensions (m)		15.0×9.0×18.0^h	15.0×9.0×11.0^h
Acting forces	Lateral forces (kN)	38,300	23,100
	Moment (kN·m)	887,200	589,760
Maximum moment		1,104,640 < 1,243,171	698,543 < 1,243,171
Ductility factor		58.3 < 60.0	57.1 < 60.0
Uplift rate at bottom (%)		15.07 < 60.0	43.0 < 60.0

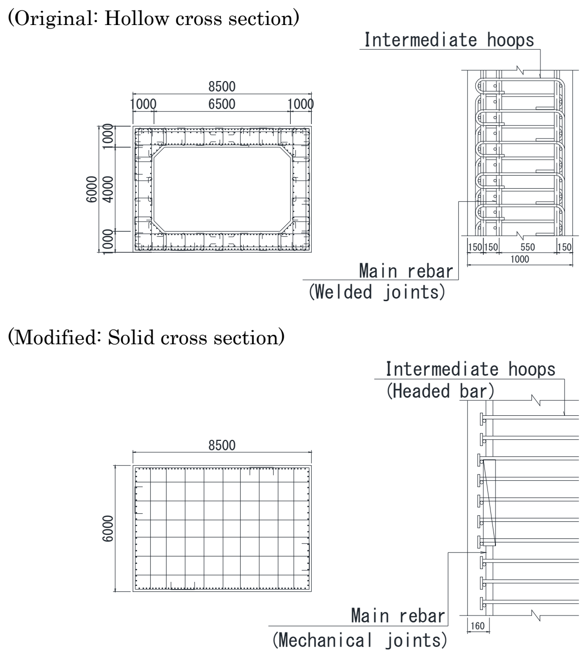


Fig. 4 Comparison of bridge pier sections dimensions

(3) Superstructure Design

As discussed previously, weight reduction of the superstructure could be achieved by reducing the thickness of the web and lower deck slab members of the superstructure. **Fig. 5** shows the cross section of the main girders.

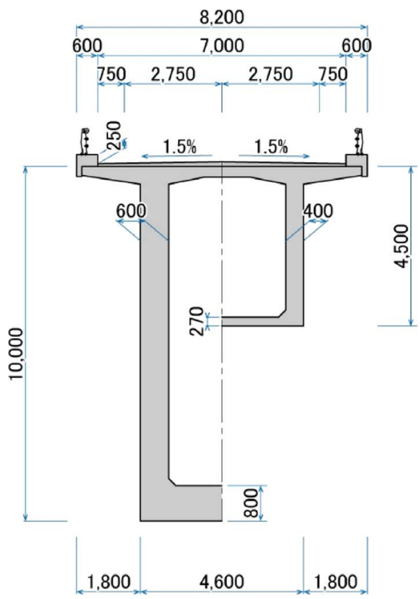


Fig. 5 Cross section of main girder

To reduce the weight of the superstructure, the arrangement of the prestressing tendons was first reviewed. In the initial design, a total of 100 tendons (12S15.2) were arranged in the upper deck slab and web, with a total of four columns within one side of the web (**Fig. 6**). To reduce the number of prestressing tendons, the prestressing steel for cantilever tendons were changed to high-strength prestressing steel

(12S15.7). The high-strength prestressing steel is about 1.28 times stronger than ordinary prestressing steel. With this material, the cantilever tendon arrangement can be modified from four columns to two columns in one side of the web, resulting roughly 15% reduction from about 209 tons to 178 tons in weight. Additionally, approximately 70% of the continuous tendons were changed from internal cables to external cables. These tendons were fixed to the web and lower deck slab using anchorages and thus the change made it possible to reduce both the number of anchorages and their total weight.

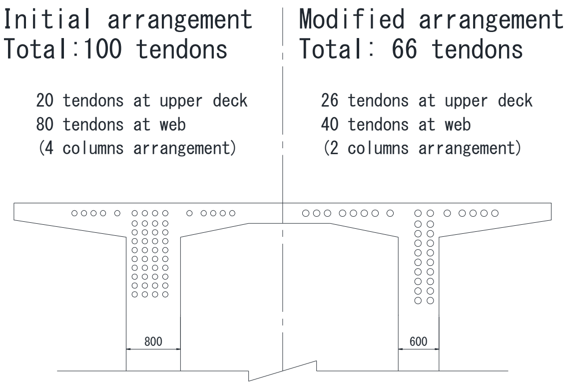


Fig. 6 Comparison of maximum tendon arrangements

The concrete strength σ_{ck} was increased from 40 N/mm^2 to 50 N/mm^2, the web thickness was reduced by 200 mm (from 800 mm to 600 mm), and the number of cantilever tendons was reduced. Increasing the concrete strength also helped to reduce the lower deck slab thickness near the pier caps—which are compression members—by 200 mm (from 1000 mm to 800 mm).
These modifications reduced the superstructure weight by roughly 10% or 5000 kN and made it possible to reduce the caisson foundation length considerably. Furthermore, because having fewer tendons and anchorages led directly to fewer work hours needed, It also improved the productivity of the superstructure and shortened the construction period.
Additionally, to reduce the number of work days for the cantilever construction, a change from the initial planned large form traveller to extra-large form traveller was considered. The initial design called for a maximum block length of 5 m for a total of 19 blocks. To reduce the number of days required for the cantilever construction, the total number of blocks was reduced to 12 blocks by making the block division with a maximum block length of 7.0 m (**Fig. 7**).

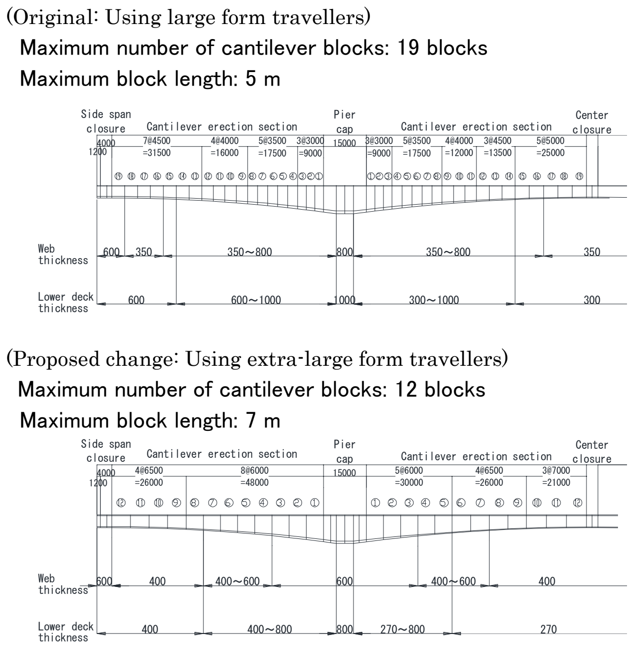


Fig. 7 Comparison of block segments

3. Conclusion

During its construction, the bridge was subjected to three natural disasters caused by torrential heavy rains, with one striking northern Kyushu in July 2017, another in July 2018, and the third in August 2019. In each of these events, the bridge suffered considerable damages, such as large deformations of the double cofferdam in pier P1 and collapse of the access road, resulting in considerable construction delays totaling over several months. However, the authors' efforts to shorten the construction process through structural and construction innovations minimized the impacts of these events and limited the construction delays.
The bridge was handed over in December 2019. With the forthcoming opening of the rerouted national highway, economic development and recovery from the disaster are expected. Finally, the authors would like to express their deep gratitude to all those who participated in this project, and they hope that this paper will be of some help in the planning, design, and construction of similar bridges.

Reference

[1] Nishimura, J., Inoue, E., Nakatsumi, K., Murao, M.: *Design and Construction of Egawa Bridge—A three-Span Continuous Box Girder Bridge with a Maximum Span Length of 173 m—*, Journal of prestressed concrete, Vol. 62 No. 6, JPCI, Tokyo, Nov.-Dec. 2020 (in Japanese).

概　要

江川大橋は，小石原川ダム建設事業に伴う付替国道の一部として，既設の江川ダム湖面上に位置する橋梁である。構造形式は，中央支間長が現日本一となる173m の PC3径間連続箱桁橋である。工事によって寸断されている地元住民のアクセス路を早期に確保するため，超大型ワーゲン，高強度 PC 鋼材，内外ケーブル方式などの採用により構造の合理化を図り，工程の短縮に取り組んだ。