# Japan's Longest-Span Continuous Rigid Frame Box Girder Bridge — Shintabisoko Bridge —

# 日本最長の PC 連続ラーメン箱桁橋 — 新旅足橋 —

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## Synopsis

The Shintabisoko Bridge is one of the bridges on National Route 418, which was rerouted as part of the Shin-Maruyama Dam project. The bridge is in a mountainous area and spans a steep V-shaped ravine, with the top of the bridge deck about 200 m above the riverbed (**Fig.1**). As a result, the pier height is 101 m, which is the third tallest, and a span is 220 m which is the longest span of any of Japan's continuous prestressed concrete rigid frame bridges[1] (**Fig.2**).
During execution, the high piers and long span resulted in substantial deflection during cantilevering, and careful camber control was required. And to reduce the construction time, ultra-large travelers were used.

## Structural Data

*Structure*: 3-span prestressed concrete continuous rigid frame box girder bridge
*Bridge Length*: 462.0m
*Span*: 119.0m + 220.0m + 119.0m
*Width*: 10.750m
*Pier Height*: 101.0m (P1), 93.5m (P2)
*Owner*: Ministry of Land, Infrastructure, Transport and Tourism
*Designer*: CHODAI Co., Ltd.
*Contractor (Superstructure)*: Sumitomo Mitsui, Showa Concrete Industry JV
*Construction Period*: Mar. 2007 – Jan. 2010
*Location*: Gifu Prefecture, Japan

**Fig. 1 Shintabisoko Bridge**

## 1. Introduction

Shintabisoko Bridge is a 3-span continuous rigid frame prestressed concrete box girder bridge with 220m long main span. The main span is the longest as prestressed concrete continuous rigid frame box girder bridge in Japan. The 101 m pier height is Japan's third tallest. To secure the seismic performance, high strength concrete with specified concrete strength of 50 MPa and high strength steel reinforcements having yield strength of 490 MPa were used.
Hereinafter, the summary of the project is described.

## 2. Outline of superstructure execution

As the concrete used in the bridge superstructure is a high strength concrete with specified strength of 50

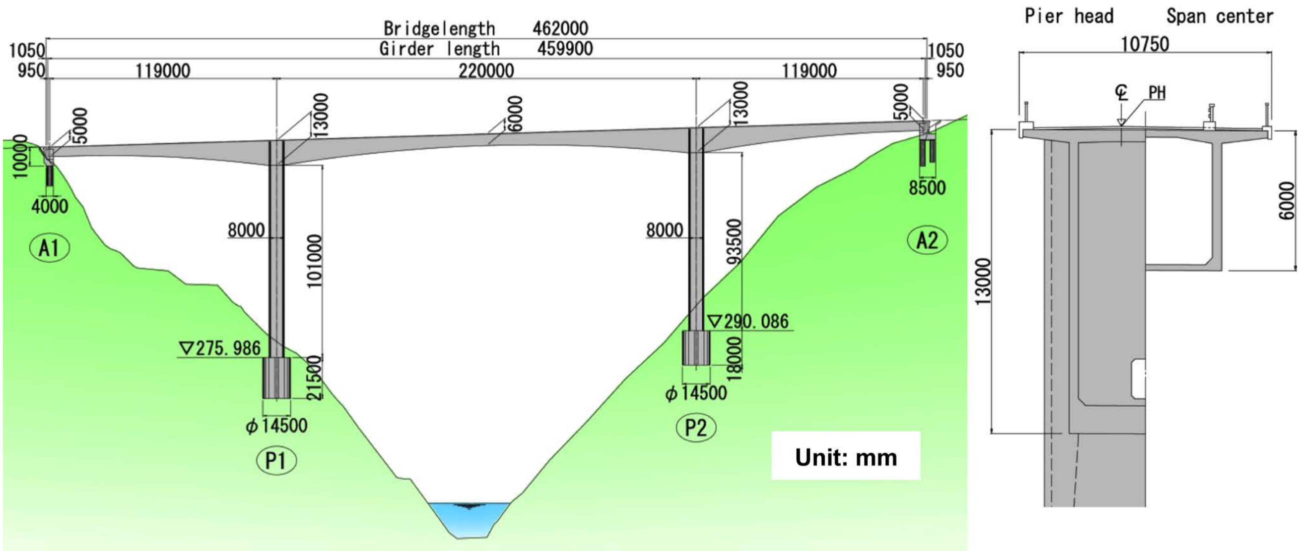


**Fig. 2 Gereral drawing of Shintabisoko Bridge**

MPa, substantial slump loss was expected when the fresh concrete is transported or pumped through the pipes. This led to concern about pump blockages and similar problems. For this reason, concrete placement plans were formulated with consideration for all the potential risks.

For the execution of this bridge, a girder bridge with a maximum girder depth of 13.0 m, we attempted to substantially reduce the construction period by using ultra-large form travelers, and we also introduced a number of other measures at the execution stage.

## (1) Concrete placement planning

The plans for concrete placement for this bridge involved using a concrete pump truck to pump the concrete to the point of placement. As the construction site is in a steep V-shaped valley, for Pier P2 the concrete has to be pumped through a vertical distance of about 50 m from the concrete pump truck location to reach the placement point (**Fig. 3**), and at maximum extension, the distance that the concrete has to be pumped is equivalent to horizontal pumping of 650 m. For this reason we used an ultra-high pressure concrete pump truck capable of pumping at 22 MPa, and 6-inch pipes. In order to be able to respond quickly to blockages and other problems, we installed a backup set of vertical pipes on each pier.

Since concrete slump declined by pumping, it was envisaged that there would be a notable reduction in slump when concrete was pumped at high pressures for long distances. As shown in **Table 1**, assuming a 4 cm slump loss in transportation and pumping, we set the slump level on leaving the plant to 19 cm, planning for a 15 cm slump on discharge from the tip of the pipe.

The mix for the main girder concrete is shown in **Table 2**. Early-strength cement and polycarboxylic acid-based superplasticizer were employed for the concrete of balanced cantilever sections of main girder.

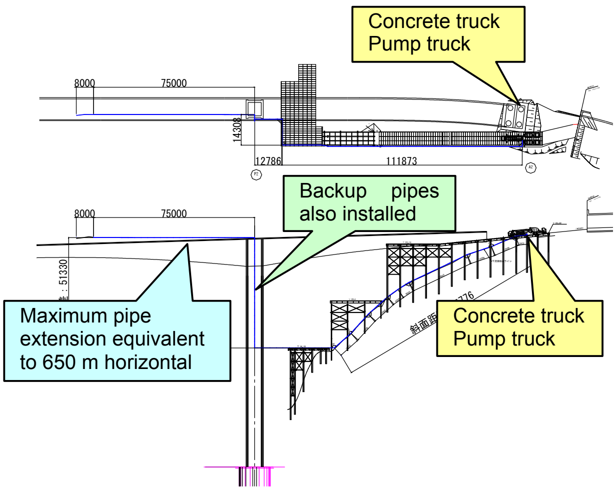


**Fig. 3 Concrete piping for Pier P2**

**Table 1 Slump settings for transportation**

| | Design | On leaving plant | On delivery to site | After pumping |
|---|---|---|---|---|
| Slump | 15.0 cm | 19.0 cm | 17.0 cm | 15.0 cm |
| Air | 4.5 % | 4.5 % | 4.5 % | 4.5 % |

**Table 2 Mixture proportion**

| W/C (%) | s/a (%) | Unit quantity ($kg/m^3$) | | | | |
|---|---|---|---|---|---|---|
| | | Water W | Cement C | Sand S | Gravel G | Admix. Ad |
| 36.8 | 42.0 | 462 | 170 | 687 | 969 | 4.160 |

## (2) Balanced cantilever construction using ultra -large travelers

For this bridge, it was decided to use ultra-large travelers to shorten the construction period. These special travelers provided performance of 10,000 kN·m, and had already been used for a small number of projects in Japan. They enabled a cantilevering segment length of up to 7.0 m, compared with the 4.5 m with ordinary available large travelers. That enabled to cut the number of segments down from 28 to 15, reducing the construction period (**Fig. 4**). **Fig. 5** shows the ultra-large traveler being assembled and its internal falsework.

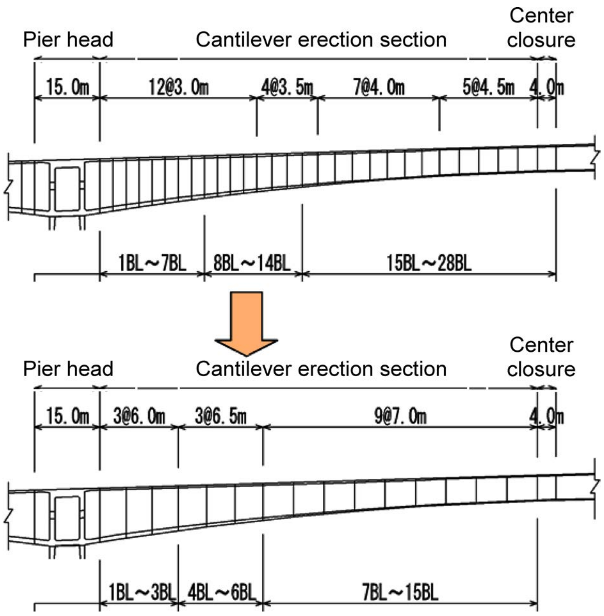


**Fig. 4 Change to segment divisions in balanced cantilevering**

**Fig. 5 Ultra-large traveler: Assembly and Internal falsework**

**Fig. 6 Balanced cantilevering using ultra-large travelers**

Because this bridge has a large girder depth (13.0 m) at the pier head but only a thin web (40 cm), and the quality of the section has to be ensured when concrete is placed, placement holes and openings for compaction were incorporated into the internal forms for the box, enabling concrete drop height to be kept down to a maximum of 1.5 m. Consequently, despite the web depth exceeding 10 m, there were no compaction problems, and concrete placement quality was good. **Fig. 6** shows balanced cantilever construction using the ultra-large travelers.

### (3) Camber control

This bridge has a central span of 220 m, cantilevered to a maximum length of 104 m from each pier, making it one of the largest girder bridge structures. Besides, in order to shorten the construction period, ultra-large travelers with a capacity of 10,000 kNm were used, enabling a maximum segment length of 7 m. Moreover, because the girder is very deep, ranging from 13 m to 6 m (5 m at end supports), and a large amount of concrete is required for each segment, it became necessary to place the concrete for each pier's left and right segments on different days. Besides, the piers are very tall, with Pier P1 reaching 101 m, whereas Pier P2 reaching 93.5 m. The unbalance moment produced by placing the concrete for each side of the cantilevered segment on different days was expected to result in substantial deflection. These led to rotational deformation of the main girder in addition to bending deformation of the pier. The prior calculations suggested a maximum deformation of 350 mm at the tip of the main girder.

#### 1) Calculating the amount of camber

For the main girder we used concrete with a static modulus of elasticity of 31.3 GPa (at 28 days, standard curing), determined by material tests. In contrast, for the piers we used concrete with a reference design value of 3.10 GPa, but the piers incorporate substantial amounts of reinforcement, so the rigidity of the steel reinforcements is likely to influence the overall rigidity of the piers. Consequently, we used a value for section rigidity converted to take into account the rigidity of reinforcements in the pier. The equivalent flexural rigidity for the bridge piers was approximately 1.3 times the value of rigidity calculated by only taking the concrete into account.

#### 2) Measurements on site

As described above this bridge was expected to have a substantial degree of deformation at each step in the execution process, and it was considered that there were many factors that could produce execution error, leading to a large impact on the overall geometry control. Among these factors, the fact that this bridge was a long-span structure with high piers means that deformation of piers has a substantial influence on main girder deformation. Since any deformation in foundation work leads to pier deformation, we surmised that it was necessary to identify any inclination of the foundations. To measure any such inclination, we installed biaxial inclinometers on the top of the wide caisson piles for piers P1 and P2 to take measurements parallel to and perpendicular to the axis of the bridge. Also, in order to measure actual deflection at the top of the piers two inclinometers were similarly installed on the tops of the pier heads. Moreover, in order to identify the effect of temperature variations, thermometers were embedded in the upper and lower deck slabs of girder.

From the results of these measurements, it was discovered that inclination of the foundations was less than the result from calculating deflection, confirming that actual pier deformation was somewhat smaller than the calculated value. Also, measurements of main girder temperature revealed the possibility that temperature differences between upper and lower deck slabs potentially affected main girder deformation. For that reason we decided to take the measurements for geometry control early in the morning when the temperature difference between upper and lower deck slabs was smallest.

### 3) Results of geometry control

The side span construction and central span construction were finished in November 2008. The bridge had a maximum camber of about 500 mm, which is large for a girder bridge, but the prior investigations described above and the survey made at the time of closure revealed that the geometry achieved was very close to the planned values, the differences were less than 30 mm. Further deflection is expected after construction finish due to creep and drying shrinkage so an eventual camber at the center of the span of 200 mm is anticipated.

### (4) Execution of central closure section

The central closure section was constructed after the completion of balanced cantilevering, with the travelers moving to the central closure section and used to execute the closure.

In order to prevent differences in deflection due to daily variations in temperature during central closure, connecting girders (Steel H-beams) are installed to eliminate deflection. There is also a risk of the temperature falling while the concrete is hardening, causing the existing girder to shrink as the temperature falls, thereby putting tensile stress on the central connecting section. To suppress this effect, concrete placement was performed early in the morning.

## 3. Conclusion

The Shintabisoko Bridge successfully met the final geometry standards for the deck surface, and execution of the superstructure was completed in May 2009 after a construction period of just 26 months (**Fig. 7**). This report has outlined the construction of superstructure, for the longest span continuous rigid frame bridge in Japan. In particular, it has described camber control specific to prestressed concrete box girder bridges with long spans and high piers, considerations when making connections continuous, and rapid construction methods using ultra-large traveler. These all have the potential to be useful when erecting similar scale bridges.

**Fig. 7 Shintabisoko Bridge (after completion)**

## References

[1] Hirata, Y. (2009), "Examples of quality and final geometry control for the Shintabisoko Bridge", paper presented at the 2009 MLIT Technology & Research Conference, http://www.mlit.go.jp/chosahokoku/h21giken/program/kadai/pdf/ippan/ippan2-01.pdf, Oct. 2009 (in Japanese)

## 概　要

新旅足橋は，新丸山ダム建設事業で付け替えた一般国道418号の橋梁の一つで，橋長462m の PC3径間連続ラーメン箱桁橋である。河床より橋面まで約200m の高低差を有する急峻なＶ字渓谷の山岳地に位置するため，橋脚高さ，支間長共に日本有数の長大橋で，橋脚の高さ101m は現在我が国第３位，支間長220m は PC 連続ラーメン橋では第１位となる。このように本橋は高橋脚・長支間 PC 橋であるため，上部工の施工においては片持ち張出し架設時のたわみ量が大きく，上げ越し管理に細心の注意が必要となった。そこで，本橋の上部工施工においては各部材の剛性評価などを詳細に管理し，所定の形状を確保した。

また，Ｖ字谷での施工であり，設計基準強度50MPa のコンクリートを水平換算距離650m 程度ポンプ圧送する必要があり，コンクリート打設時のトラブルが懸念されたため，事前のポンプ圧送試験によってコンクリート打設時のスランプロスの調査や必要な圧送能力の設定を実施した。

さらに，超大型移動作業車により，工期短縮を図った。