Design and Construction of Shallow T-Shaped Girder Bridge
— Tagajo Daiichi Bridge —

桁高制限を受けるＴ型ラーメン橋の設計・施工
— 多賀城第一橋りょう —

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Synopsis
The governments of Miyagi Prefecture and Tagajo City and East Japan Railway Company (JR East) worked on a project to construct an elevated railway along a 1.8 km section near Tagajo station to eliminate some level crossings. The bridge is a rigid frame with two 40.5 m spans. The cross section of the superstructure varies its shape from box girder around interior pier to T-shaped girder toward side span ends. The bridge crosses two lanes of municipal roads. At the intersection with the road, it is necessary to ensure an overhead clearance of 4.7 m from the ground. However, raising the level of the rails gives a significant impact to the project's cost. Therefore, the girder height of the bridge is limited up to 1.5 m. In the superstructure, the main girder is a partially prestressed concrete (PPC) structure, and the slab and cross beam are reinforced concrete (RC) structures. The piers are RC structures, and the foundations are cast-in-place piles with preloading systems at the tips of the piles.

Structural Data
Structure: 2-span continuous box and T-shaped girder rigid frame bridge
Bridge Length: 81.0m
Span: 40.5m
Width: 10.2m
Location: Miyagi Prefecture, Japan

1. Structure outline
In the heart of Tagajo City in Miyagi Prefecture, level crossings along the JR Senseki Line have caused problems in the development of the city due to traffic congestion and the splitting of the communities. Therefore, Miyagi Prefectural government and Tagajo City municipality worked with JR East on a project to construct an elevated railway along a 1.8 km section encompassing Tagajo Station in order to eliminate some level crossings on the Senseki Line. An overall view of the project is presented in **Fig. 1**. The Tagajo Daiichi Bridge is a part of the Senseki Line, which was elevated in this project. The bridge is a rigid frame with two 40.5 m spans. The cross section of the superstructure varies its shape from box girder around interior pier to T-shaped girder toward side span ends. The bridge crosses two lanes of municipal roads.

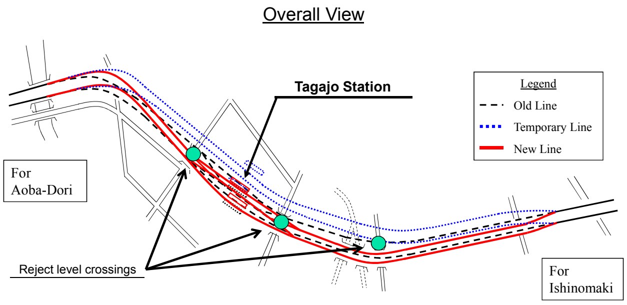

Fig. 1 Overall view of the project

2. Dimensioning conditions
At the intersection with the road, it was necessary to ensure overhead clearance of 4.7 m from the ground. However, raising the level of the rails would impose a significant cost impact on the project. Therefore, the girder height of the bridge is limited up to 1.5 m. Furthermore, the elastic ballast track was adopted as the track structure to reduce maintenance work.

3. Ground conditions
The construction zone covers two deep valleys in the north-south direction. The railway crosses these valleys. A topographic map of the project area is presented in **Fig. 2**.
The strata are as follows: The first stratum is earth

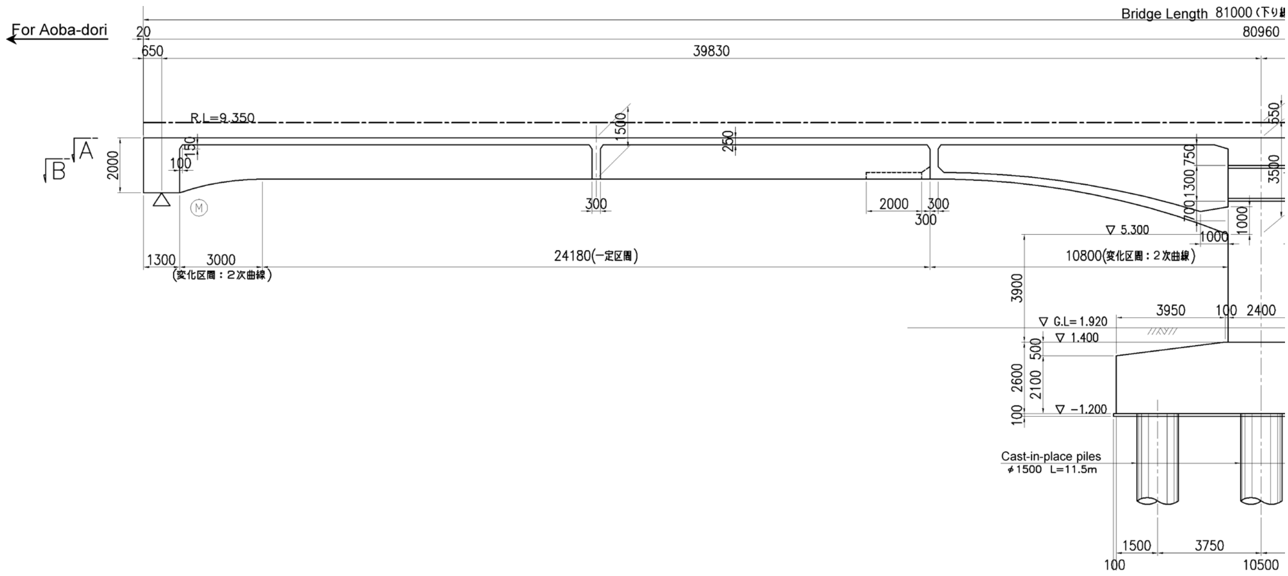

Fig. 3 Drawing of the superstructure (Units: mm)

and sand from the Holocene epoch, and alluvium. The second stratum is Pliocene sandstone, gravel, and tuffaceous Miocene sandstone. The third stratum is shale from the Middle Triassic epoch. The support layer (11.92 m below the ground level) is mainly comprised of this shale.

4. Design outline

(1) Design conditions

The Tagajo Daiichi Bridge was designed in conformity to "Railway Structure Design Standards" by the Railway Technical Research Institute in Japan, and the design manual of JR East. As the design conditions, M-18 (for only operating Electric Multiple Units) was adopted as live load with 110 km/h design speed. In terms of environmental conditions, the location is categorized as a cold area (although not subject to heavy snow).

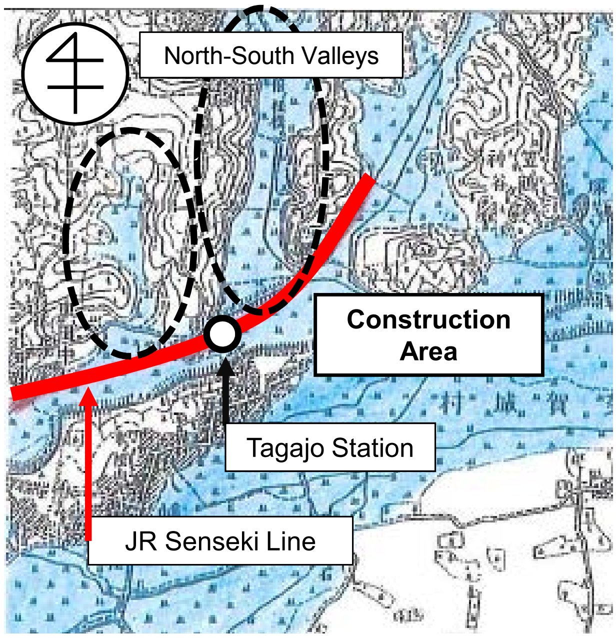

Fig. 2 Topographic map of project area

Since the construction site is a narrow strip of land, a temporary line was necessary, and construction of the inbound and outbound lines was separated. The inbound line was elevated in Phase 1 and the outbound line was elevated in Phase 2. Also in Phase 2, the inbound and outbound line structures were united. Therefore, out of consideration for the risk of an earthquake during Phase 1, the structure was designed by elastic static analysis to behave within elastic range under the seismic force with the seismic coefficient of 0.25. In Phase 2, it was designed to satisfy the seismic capacity as predefined.

(2) Superstructure

The main girders are PPC structures, and the slabs and cross beams are RC structures. The prestressing tendons are arranged to be stressed from both sides. The minimum depth of cover concrete was specified as 50 mm considering construction error.

As mentioned above, the height of the main girder is limited up to 1.5 m. Also, the central part of the span has a T-shaped cross section considering its workability. The supports at both ends of the superstructures are movable with rubber bearings. The superstructure is monolithically connected to the interior pier. The superstructure at the top of the interior pier is box girder with 3.5 m high. It varies continuously to T-shaped girder with 1.5 m high in the central part of the spans (**Fig. 3**).

The cross-section of the main girder was determined considering the distribution of bending moment in ultimate limit state; it was decided such that 10.8 m long both sides of the interior pier are box-shaped, whereas the rests of the parts are T-shaped.

The specified design strength of the superstructure concrete is 40 N/mm^2 (cylindrical strength). The required compressive strength of the concrete for prestressing is specified as 34 N/mm^2. The compressive

stress limits were specified as 20 N/mm^2 for the state immediately after prestressing and 16 N/mm^2 for permanent loads. The tensile stress limits were specified as -3.1 N/mm^2 for the state immediately after prestressing and -2.7 N/mm^2 for permanent loads.
Six prestressing tendons ($12 \times \phi 15.2$mm) with a design tensile strength of 1880 N/mm^2 were arranged.
The cross section of the slab was determined by the fatigue limit state of the wing deck. The cross section of the crossbeam was determined by the shear of the ultimate limit state, because it was a low shear span member.
The deformation was calculated considering the reduction in cross sectional stiffness due to cracking.

(3) Substructure

Since the ground has heavy relief in the support layer, the support layer of the interior pier is inclined (**Fig. 4**). The foundation of interior pier comprises nine 11.5 m long cast-in-place piles with diameters of 1.5 m (the piles are arranged in a 3×3 layout). The consideration was given to the variation of the pile lengths by ± 1 m due to the unevenness or inclination of the support layer in the design. The special piling method, "Cast-in-place pile with preloading systems at the tip of the pile" was employed. The piles were constructed in following procedure.

1. Install cast-in-place bored piles as usual.
2. Inject cement milk into an infusion bag attached to a rebar cage tip beforehand from the ground after the concrete of the pile is hardened.
3. Remove any slime at the tip of the pile.
4. Reinforce the ground to minimize any subsidence of the pile and improve the bearing capacity by preloading.

This piling system is particularly effective for the foundations of structures such as viaducts that lack pile cap beams and are easily affected by unequal settlement.

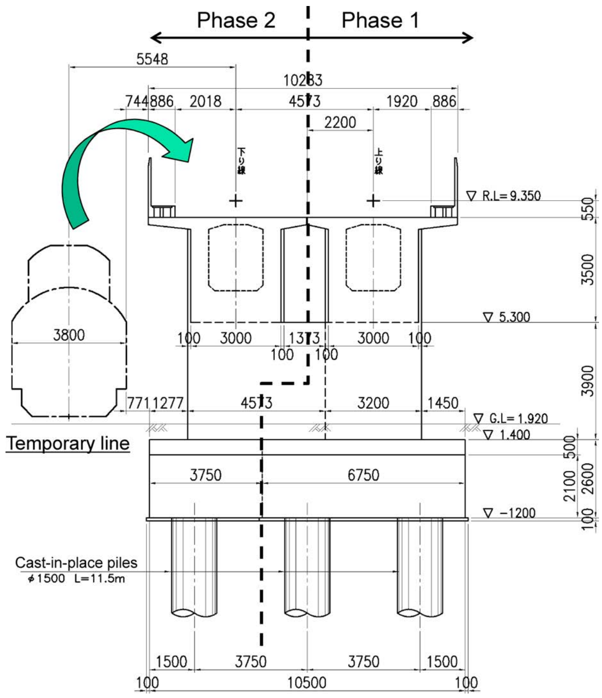

Fig. 5 Cross section at interior pier (Units: mm)

The bridge piers are an RC structure, with cross-sectional dimensions of 7.5 m × 2.4 m. The pier construction was carried out in two phases. In Phase 1, six piles (2×3) were constructed, while the other piles were constructed in Phase 2 (**Fig. 5**).

5. Construction

(1) Formwork

Although the cross section of the superstructure is box shaped, inner falsework was installed just after the arrangement of rebar of bottom slab and webs and

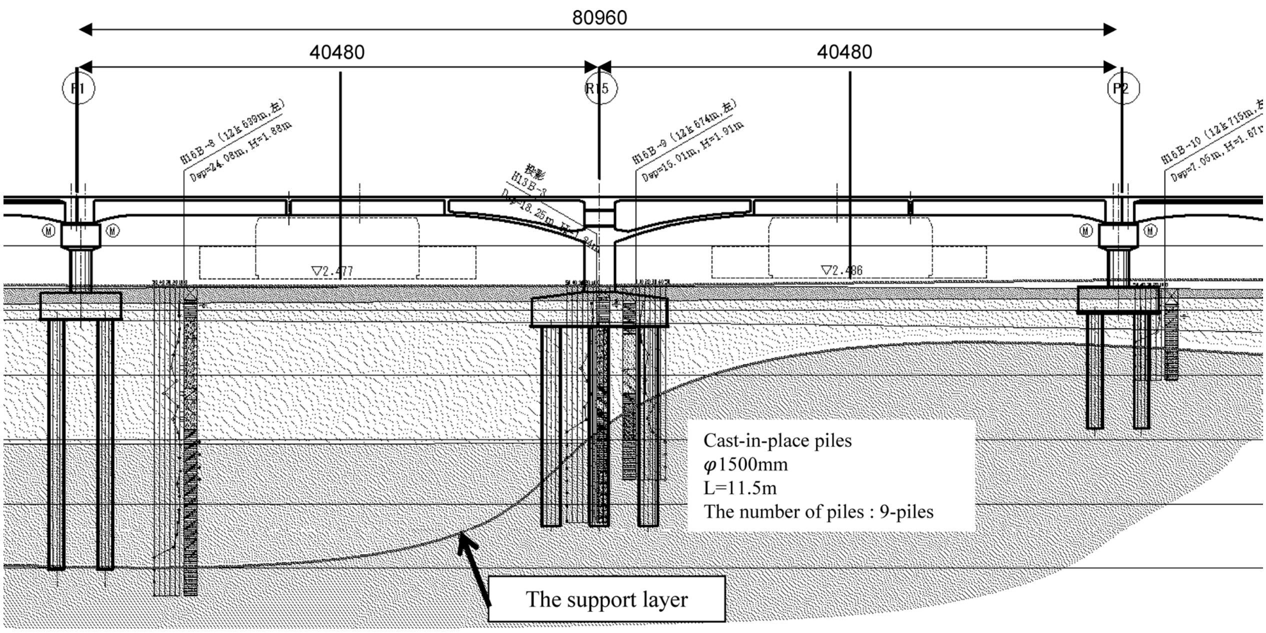

Fig. 4 Topographic map of the bridge (Units: mm)

sheaths of prestressing tendons to obtain monolithic concrete. The inner falsework had pillars to support H-beams integrated with the support receiving concrete (**Fig. 6**). The H-beams were removed and the spaces were filled with concrete after dismantlement of the formwork. The photographs of construction situation in Phase 2 and completed bridge are shown in **Fig.7 and 8**.

(2) Opening for work

Regarding access for the construction and dismantlement of framework, one opening was planned on the Ishinomaki side at first. However, difficulties were anticipated with the transportation of materials in the box girder and concrete placement. Therefore, an additional opening was added near Aoba-dori.

Fig. 6 Formwork in box girder

Fig. 7 Construction situation (Phase 2)

Fig. 8 Completed bridge

概　要

本橋りょうは２本の都市計画道路をまたぎ，道路面より4.7mの空頭を確保し，かつ，レールレベルを上げることによるプロジェクト全体工事費の引上げを避けることを目的として径間部の桁高を1.5mに抑えるため，構造形式にスパン40.5mの２径間単純箱形断面Ｔ型ラーメン橋を採用した。

スパン中央部の桁高を1.5mと低く抑えながら型枠・鉄筋組立・コンクリート打設などの施工性を確保するため，スパン中央部ではＴ形断面の梁形状とした。上部工両端支点は可動構造のゴム支承とし，中央部は橋脚とのラーメン構造とし，桁形状は主桁に発生する応力分布に配慮してスパン40.5mのうち，ラーメン橋脚側の10.8mを桁高3.5mの箱形断面とし，径間部から両端支点までをＴ形断面とした。

箱型断面の上部工コンクリートを一体施工とするため，内部支保工は下床版・ウェブ鉄筋，シース設置後に組立て，支持受コンクリートと一体化したＨ形鋼を支柱受とし，解体後Ｈ形鋼は切断・撤去し後埋めする構造とした。

以上のとおり，施工性に配慮しながら厳しい桁高制限をクリアして，併走する都市計画道路２本を跨ぐ，まさに立体交差化事業を象徴する橋りょうであり，今後の連続立体交差化事業における長大橋りょう構築技術の発展に寄与するところが大きい。