Design and Construction of Extradosed Bridge with Corrugated Steel Webs That Realized Substantial Recovery of Progress — Ikuno Bridge —

大幅な工程回復を実現した波形鋼板ウェブエクストラドーズド橋の設計・施工 — 生野大橋 —


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Keywords: extradosed bridge, launching method, extra-large traveler, progress recovery
DOI: 10.11474/JPCI.NR.2018.91

Synopsis

Ikuno Bridge is a 7-span continuous extradosed prestressed-concrete box-girder bridge with corrugated-steel webs. It is located in Kobe City of Hyogo Prefecture, between the Kobe and Takatsuki junctions of the Shin-Meishin Expressway. The total length of the bridge is 606m, with a 188m main span, which is among the longest in Japan (**Fig.1**).

The bridge was constructed using various special methods such as incremental launching of the intermediate support at pier 6, the cantilever method using an extra-large traveler, and a parapet made of precast concrete.

Two stay cables of 37S15.2 steel are arranged in a twin-plane configuration and meet at the center of the deck. Because of dimensional limitations, a saddle structure was adopted to route the cables through the pylons. In addition, as countermeasures against cable vibration due to wake galloping, wind-tunnel tests were conducted and a vibration-control damper was installed.

Fig.1 Ikuno Bridge

Fig.2 Crossing over an existing railway

Structural Data

Structure: 7-span continuous extradosed prestressed-concrete box-girder bridge with corrugated-steel webs
Bridge Length: 606.0m
Span: 96.2m + 188.0m + 103.0m + 39.0m + 39.0m + 71.0m + 66.2m
Width: 25.15m (at the start of servicing)

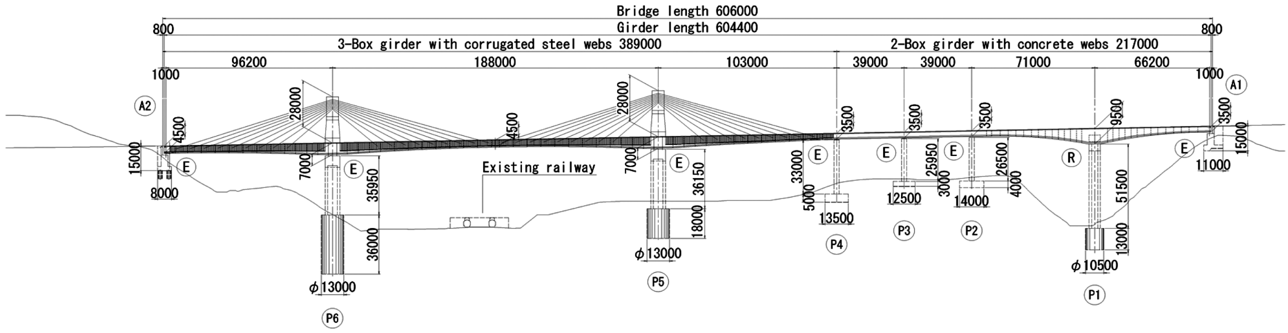


Fig.3 General view

Tower Height: 28.0m
Owner: West Nippon Expressway Co., Ltd.
Designer: Taisei – P.S. Mitsubishi JV
Contractor: Taisei – P.S. Mitsubishi JV
Construction Period: Sep. 2012 – May 2018
Location: Hyogo Prefecture, Japan

1. Introduction

A 188-m-long main span was required for the bridge to cross over an existing railway at an intersection angle of approximately 15°. To achieve this, an extradosed bridge constructed by the cantilever method with corrugated-steel plate webs was adopted (**Fig.2**). The bridge is a 7-span continuous extradosed bridge. Four spans near the beginning have a two-box girder with concrete webs, and three spans near the end have a three-box girder with corrugated-steel webs (**Fig.3**).

The structure's original design of a concrete-web box-girder structure with a 4-m-long block was changed to an 8-m-long block (to be constructed by an extra-large traveler) because substantial recovery of progress was required for superstructure construction. Furthermore, the structure type was changed from a concrete box girder to a corrugated-steel web structure to reduce the superstructure weight. High-strength prestressing steel cables coated with epoxy resin were used as cantilever cables, leading to fewer cables.

The precast intermediate support at pier 6 on the critical path was constructed simultaneously with the pier head. After constructing the latter, the above precast intermediate support was transferred onto the pier head. To reduce the weight of the precast intermediate support during launching, only part of it was constructed in advance; the rest was cast *in situ* after the support was transferred onto the pier head.

The prestressing steel cables could be arranged and stressed one by one in the saddle structure used for the pylons of this bridge. This arrangement improved workability using a compact mono jack and is robust against fretting corrosion fatigue. The fact that each strand can be exchanged individually during the service period is excellent from a maintenance perspective.

2. Design

(1) Design of Main Girder

With a single-plane cable-stayed structure (**Fig.4**), the total width of the bridge was 25.15m at the start of service and will be expanded to 35.40m in the future. In addition to an investigation by sequential analysis that reflected normal construction steps, the design considered (i) the effective transmission length of the axial force of the stay-cable tension calculated by three-dimensional (3D) finite-element analysis (FEA) and (ii) allotment of the shear force of the stay-cable tension calculated by 3D frame analysis. Furthermore, a 3D FEA that reflected the construction sequence was performed to investigate local stresses caused by erection with an extra-large traveler (**Fig.5**).

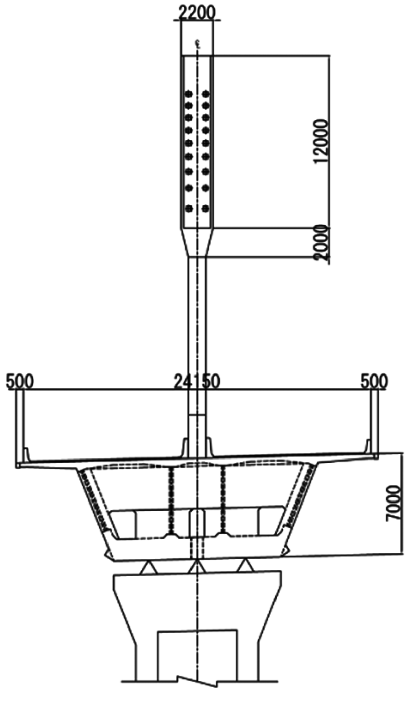


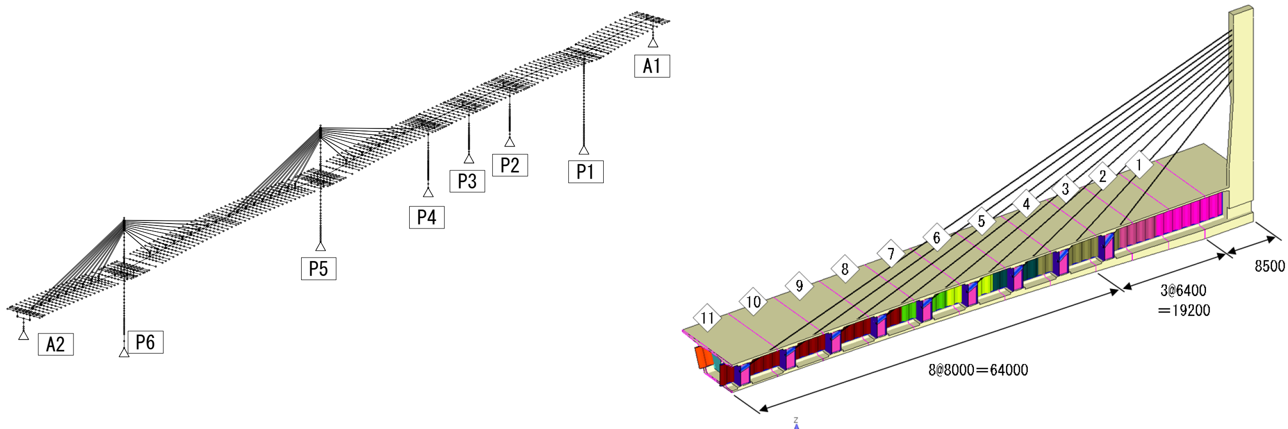
Fig.4 Cross section (at start of service)

(2) Seismic Design

To consider the phase difference due to the long bridge length and the influence of a shattered zone near pier 6, in addition to seismic design based on the specifications for a highway bridge, an examination was conducted by earthquake response analysis using a ground model that reflected the shattered zone (**Fig.6**).

(3) Investigation of Incremental Launching Method for Intermediate Support

Investigation of the incremental launching method for the intermediate support was executed by a 3D FEA



(a) Frame analysis model (b) FEM analysis model

Fig.5 Three-dimensional analysis model

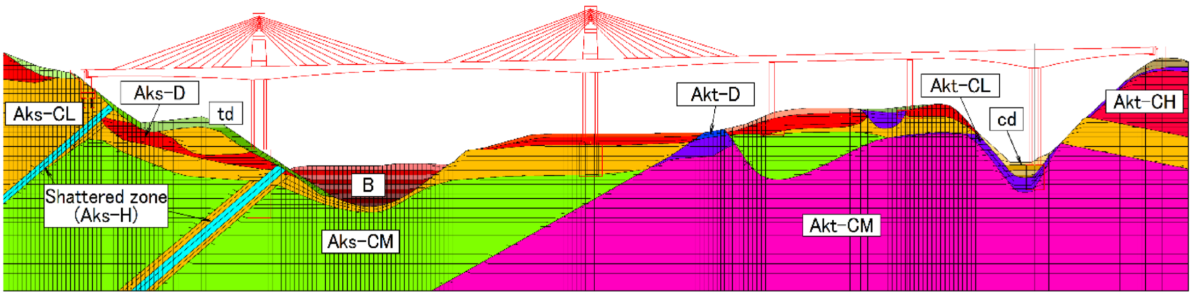


Fig.6 Earthquake response analysis using ground model

to reflect the changes in the structural system upon (i) completing the preceding construction part of the intermediate support, (ii) transferring the intermediate support, and (iii) completing the intermediate support (**Fig.7**).

(4) Vibration Control of Stay Cables

Because two 37S15.2 stay cables were arranged in a twin-plane configuration, cable vibration due to wake galloping was an existing concern. Accordingly, the spacing between the two cables was increased and small sectional cables were adopted. In addition, after the required damping performance had been confirmed by wind-tunnel testing, a friction damper with a high damping effect was adopted for the first time in Japan (**Fig.8**).

3. Construction

The extradosed bridge section crossing over the existing railway was erected by means of the cantilever method, using an extra-large traveler starting from piers 5 and 6. Because all erection work for the center span was done above the railway, construction was performed under thorough safety management. The following countermeasures were taken for appreciable recovery of progress for superstructure construction, and as a result progress recovery of approximately eight months was achieved.

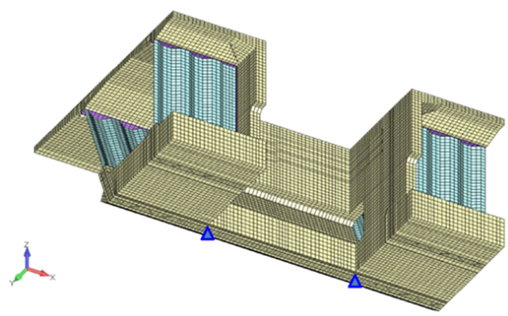
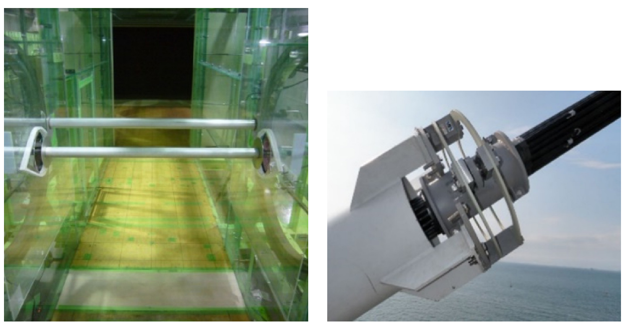


Fig.7 Three-dimensional FEM analysis on launching



(a) Wind tunnel test (b) Vibration damper

Fig.8 Vibration control of stay cables

(1) Incremental Launching Method for Intermediate Support

Because construction of the intermediate support for pier 6 was on the critical path in all steps, the intermediate support was constructed in advance and erected by incremental launching. As a result, progress recovery of approximately two and a half months was achieved. The precast intermediate support was constructed simultaneously with the pier head, and after the construction of the pier head the above precast intermediate support was transferred onto the pier head at a pace of approximately 20m in three days (**Fig.9**). To reduce the weight of the precast intermediate support during launching, part of the bottom slab and diaphragm were cast *in situ* after the support was transferred onto the pier head. This reduced the weight of the precast intermediate support during launching to approximately 15,000 kN.

(2) Cantilever Erection by Extra-large Traveler

The number of construction blocks of the main girder was reduced from 22 to 11 by adopting an extra-large traveler (20,000kN m) that enabled the block length to be 8m, which is twice the original length (**Fig.10**). A progress recovery of 3.5 months was achieved through this change in construction method. Because erection work by the extra-large traveler was carried out above the existing railway, the traveler could be moved for only two and a half hours each night (1:30–4:00 am) when the railway was closed.

(3) Precast Parapet

A progress recovery of one and a half months was achieved by using precast parapets. Parapets are usually constructed after completion of the main girder, using a mobile crane on the bridge deck. However, to reduce the risk of materials, tools, and equipment falling onto the railway, precast parapets were installed temporarily in the extra-large traveler during erection of the main girder.

4. Conclusion

Superstructure construction of the Ikuno Bridge was completed in December 2017. This was the first time that a corrugated-steel web structure with single-plane stay cables and incremental launching of the intermediate support had been used in Japan. It was also the first time that the saddle structure and friction damper adopted in this bridge were used in Japan. The many new technologies used in this project overcame difficult constraints such as a tight schedule and working above an existing railway.

(a) Before launching

(b) After launching

Fig.9 Incremental launching method

Fig.10 Cantilever erection by extra-large traveler

概　要

新名神生野大橋は，新名神高速道路の高槻 JCT・IC－神戸 JCT 間の兵庫県神戸市に位置している。本橋は，国内最大規模の中央径間長（188m）を有した橋長606m の波形鋼板ウェブエクストラドーズド橋である。

本橋は，上部工工事の大幅な工程回復のために，P6柱頭部の押出し架設，超大型移動作業車による架設，壁高欄のプレキャスト化などの対策を講じた。本橋の斜材は，37S15.2が 2 本並列配置されている。主塔部は寸法の制約からサドル構造としている。また，ウェイクギャロッピングによる斜材ケーブルの振動対策として，風洞実験を行い，制振ダンパーを設置した。本工事では，これらの新しい試みを取り入れることにより，厳しい工程，鉄道との交差などの困難な制約条件を克服することができた。

本橋は，2017年12月に上部工工事が完成した。