Replacement of Existing Steel Trestle Bridge with Concrete Bridge
— New Amarube Bridge —

鋼トレッスル橋からコンクリート橋への架替え
— 新余部橋りょう —

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Keywords: extradosed bridge, seismic design, wind protection, sliding, rotation, replacement
DOI: 10.11474/JPCI.NR.2014.125

Synopsis
New Amarube Bridge (**Fig.1**) is a railway bridge located between Yoroi and Amarube on Sanin line of West Japan Railway Company. This bridge adopted 5-span continuous prestressed concrete box girder extradosed bridge, with length 310m and maximum pier height 40m.
The previous Amarube Bridge (**Fig.2**) had endured severe winter climate for about 100 years owing to devout maintenance. But frequent service delay and cancel caused by the limitation of wind velocity had severe effect on daily commuting. Because of these circumstances, Amarube Bridge was rebuilt to improve safety and punctuality of transport.

Fig.1 New Amarube Bridge (2010-)

Fig.2 Amarube Bridge (1912-2010)

Structural Data
Structure: 5-span continuous prestressed concrete box girder extradosed bridge
Bridge Length: 310.6m
Span: 50.1 + 82.5 + 82.5 + 55.0 + 34.1m
Width: 7.25m
Tower Height: 5.0m
Owner: West Japan Railway Company
Designer: JR West Japan Consultants Company
Contractor: Shimizu/Zenitaka Joint Venture
Construction Period: Mar. 2007 – Feb. 2011
Location: Hyogo Prefecture, Japan

1. Introduction
The previous Amarube Bridge built on March 1912 was the biggest steel trestle bridge in the East then. But huge effort to maintain the bridge had been paid

because its location is near Sanin coast line (about 70m) which has very strong winter wind and the bridge's corrosion spreads very rapidly. And after the train tumbling accident caused by the unique blast in winter 1986, the limitation of wind velocity lowered from 25m/s to 20m/s in 1988 and the service cancel and delay increased a lot. On this occasion, "The Council of Amarube bridge" was established in 1991 and decided to reconstruct the bridge in 2002. The reconstruction work started in March 2007 and the operation started in August 2010.
Hereinafter, the summary of project is described.

2. Design

(1) Basic condition

The new bridge was planned to construct beside the previous bridge which was still in service. The alignment of new bridge has 220m long straight line from Amarube side and connected to existing rail position by S-curve because it linked to tunnel in Yoroi side. The bridge span length was determined as 82.5m from the condition of cross road and to avoid the existing pier foundation. Regarding superstructure, priority was given by aesthetic appearance of simplicity and straightness of previous bridge, extradosed bridge was adopted. The height of girder is 3.5m along the whole span, and the height of main tower is 5.0m considering continuous appearance of wind protection wall and main tower, and maintenance ability (**Fig.3 and Fig.4**).

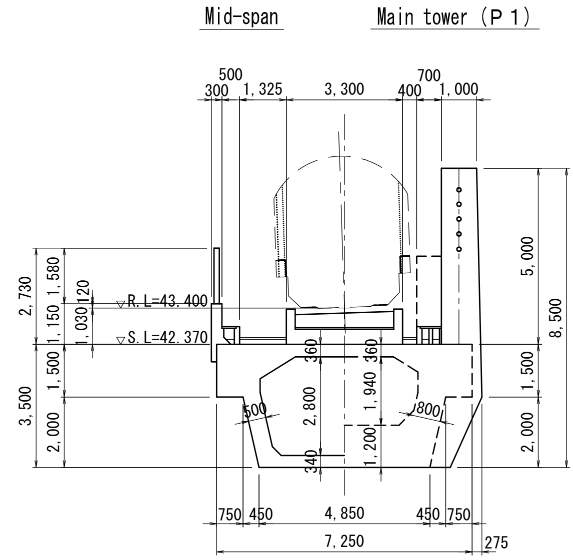

Fig.4 Typical cross section of girder

(2) Seismic design

About joint condition between pier and girder, ramen structure can't satisfy the required damage level (L2 seismic motion) because of low girder height, thus continuous girder bridge with elastic bearing was adopted, and motion limiters were set to both abutments. Structure of motion limiter is a reaction wall set in-situ rock bed and the wall is separated from abutment to avoid seismic momentum force for abutment.

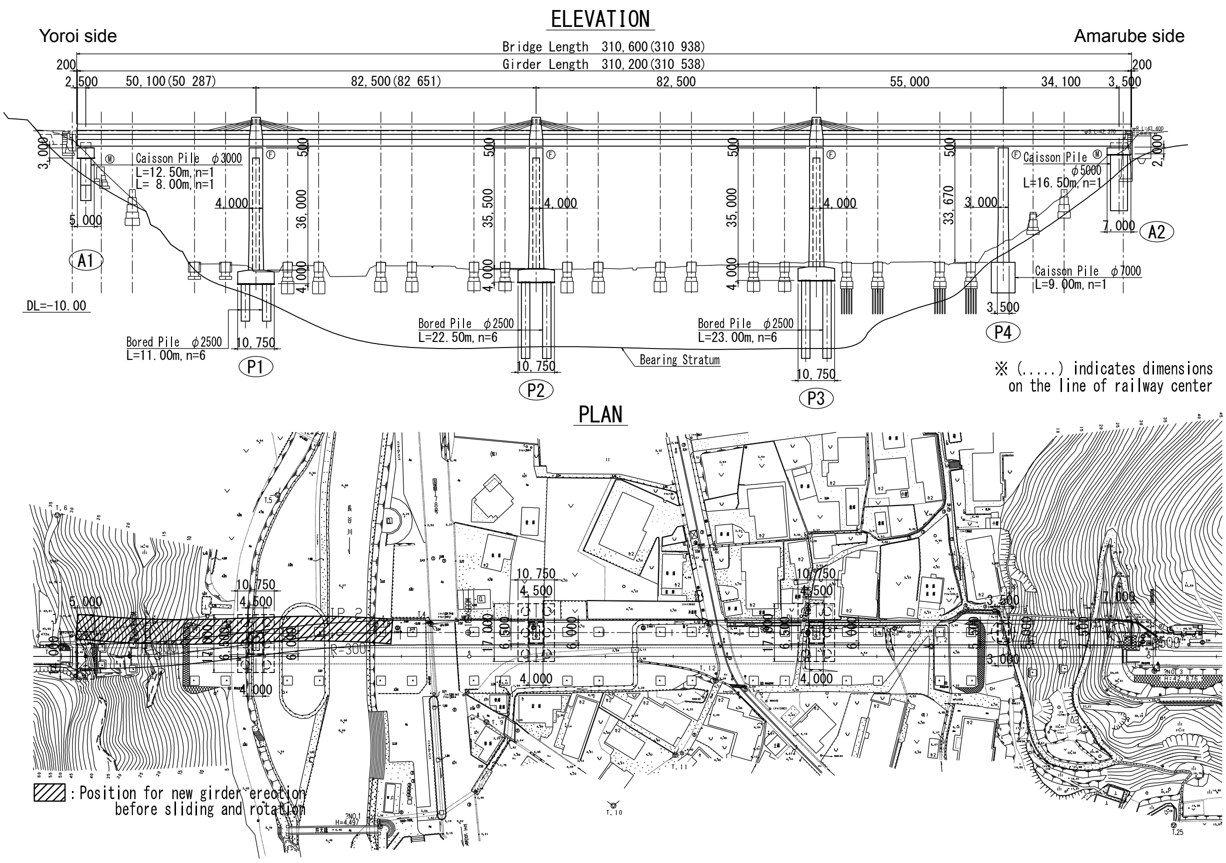

Fig.3 Elevation and plan of New Amarube Bridge

For seismic design, motion of long-period structure as this bridge can't ignore higher mode effect, thus 3D non-linear time history response analysis was conducted[1].

(3) Protection against corrosion by chloride ions

Since Amarube Bridge faces on the Sea of Japan and is exposed to severe corrosive condition, two types of corrosion protection methods were adopted as increasing concrete cover and control of water-cement ratio.
From design work stage, investigation of chloride ion density and exposure test had been secured.
The design concrete cover was determined based on "Design Standards for Railway Structures and Commentary (Concrete Structures)[2]" considering the test's results. In design of mixture of concrete, unit cement volume was reduced and water-cement ratio was 10% lower than usual mix proportion by using low-heat cement and air entraining and water reducing agent.

(4) Countermeasures to strong wind

As it had been difficult to assure punctuality of train schedule because of strong wind, wind protection wall was set to increase the limitation of wind velocity of service 20m/s to 30m/s. To decide the wall height, wind tunnel experiment (**Fig.5**) was conducted at RTRI (Railway Technical Research Institute).

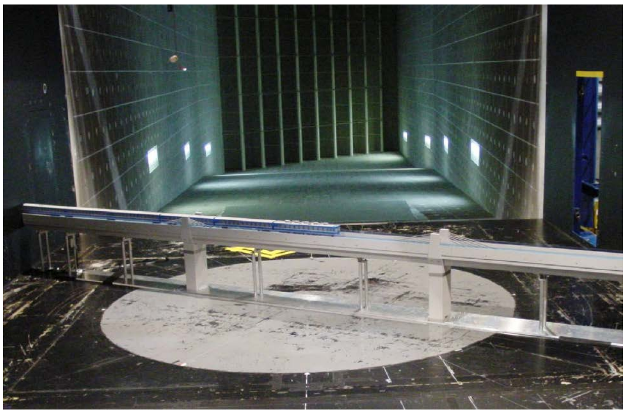

Fig.5 Wind tunnel experiment

3. Construction

(1) Execution of superstructure

Execution method of superstructure was cantilever erection method at P1 to P3 (**Fig.6**). The closure pour between P1 and P2 was executed by suspended scaffolding method. At A2 side span, staging erection with temporary bent supports and truss beams was adopted.
The concrete cover of main girder and main tower is 80mm and synthetic short fibers were admixed to concrete (0.05vol %) to prevent flaking off.
The new bridge and the previous one apart only 7m at center line, while cantilever erection, the distance of form traveler and the existing bridge is just 30cm.
To avoid the collision between form traveler and existing pier by strong wind, analysis of stableness of form traveler against wind was done. The condition of analysis of maximum instantaneous wind velocity was 58m/s.

Fig.6 Cantilever construction at P1 to P3

(2) Replacement of bridge girder

In order to connect the main girder to existing tunnel line at A1 side, bridge girder sliding and rotation works were executed during 26 days of train service suspended period (**Fig.7**).
After removing the previous bridge along interference section, at 40m high from ground, bridge girder was slid horizontally and rotated 5.2° around P1 pier axis (**Fig.8, 9 and 10**). Then, closure part between P1 and P2 was poured to connect.

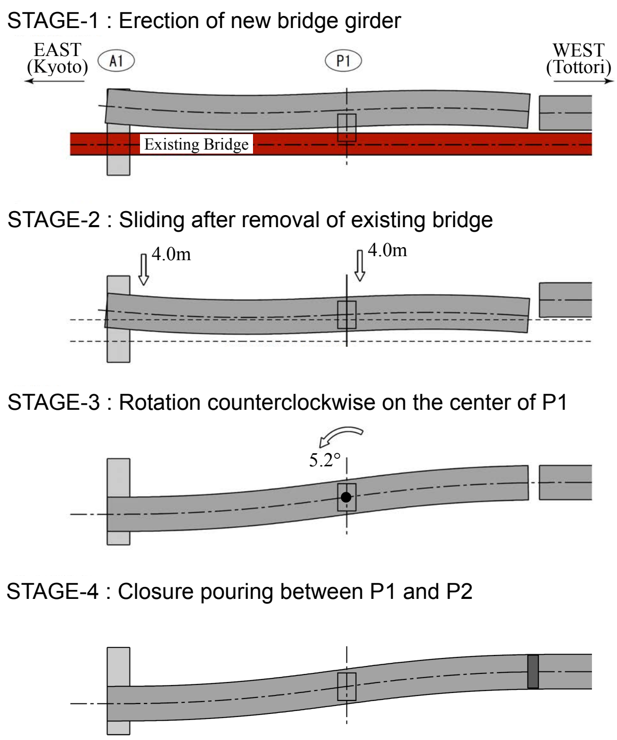

Fig.7 Sequence for sliding and rotation of girder

This girder was about 93m in length and 38,200kN in weight. Reaction forces to A1 abutment and P1 pier were 4,900kN and 33,300kN, respectively.
Sliding and rotation of box girder with such heavy weight and weight balance had never been executed before. To assure the construction reliability and safety, experimental model test and numerical analysis had been conducted beforehand to reflect to the in-situ execution plan[3].
Finally, girder sliding and rotation was succeeded and the gap of center line of bridge was less than 10mm.

Fig.8 Before sliding of girder

Fig.9 After sliding of girder

Fig.10 After rotation of girder

4. Conclusion

The new bridge is 5-span continuous box girder extradosed bridge with wind protection walls (**Fig.11**). Bridge replacement work beginning in Mar. 2007 was conducted under extreme condition with severe environment particular to this district where monsoons and heavy snowfall occur. Numerous difficulties were overcome during construction and in particular, thorough measures were taken prior to the final phase of sliding and rotation work of bridge girder. Consequently, the first train was able to cross the new bridge in the early morning of Aug. 12, 2010.

Fig.11 New Amarube Bridge (2010-)

References

[1] *Design Standards for Railway Structures and Commentary (Seismic Design)*, RTRI (Railway Technical Research Institute), Tokyo, Oct. 1999 (in Japanese)

[2] *Design Standards for Railway Structures and Commentary (Concrete Structures)*, RTRI (Railway Technical Research Institute), Tokyo, Apr. 2004 (in Japanese)

[3] Yoshitake, K. et al. : *Experimental and analytical studies of lateral yarding and rotation control of bridge girder for replacement of Existing Trestle Bridge with New Amarube Bridge*, Journal of Structural Engineering. A, Vol.57A, JSCE (Japan Society of Civil Engineers), Tokyo, pp.940-948, Aug. 2011 (in Japanese)

概　要

余部橋りょうはJR山陰本線鎧・餘部間に位置する橋長310m，高さ約40mの5径間連続PC箱桁エクストラドーズド橋である。旧余部鉄橋は献身的な維持管理により約100年間冬季の日本海の厳しい自然条件に耐え続けたが，風速規制により頻繁に発生する列車の遅延，運行休止が沿線地域の通勤，通学や経済活動に大きな影響を及ぼしていた。

このような状況のもと，列車運転の安全性向上と定時性確保を目的に余部橋りょうは架け替えられた。本稿では余部橋りょう架替え工事に関する設計・施工について報告する。