Seismic Retrofit of Hamana Bridge
— Seismic Retrofit Technology for Large Bridges —

浜名大橋耐震補強の施工
— 大規模橋梁の耐震補強技術 —

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Synopsis
The Hamana Bridge on the Hamana Bypass of National Highway Route 1 is a five-span continuous rigid-frame bridge with a hinge at the center of midspan (hereinafter,hinged rigid-frame bridge). As one of the major bridges in Japan, the Hamana Bridge was designed and constructed from 1973 to 1976[1][2]. The bridge can be seen in full view from the Tokaido Shinkansen passing near Lake Hamana in Shizuoka Prefecture. Due to the level of structural design analysis technology at the time when the bridge was built, many bridges of this type were designed and constructed in the 1960s to the 1980s. Because of the peculiar structural characteristics of hinged sections, deterioration of hinge restrainers has occurred. Furthermore, there are some cases that excessive deformation of bridges happened for same type of bridges.

Structural Data
Structure: 5-span continuous hinged rigid-frame bridge
Bridge Length: 631.80m
Span: 55.0m + 140.0m + 240.0m + 140.0m + 55.0m
Width: 9.00m
Owner: Ministry of Land, Infrastructure, Transport and Tourism
Designer: IDEA Consultants, Inc.
Contractor: Oriental Shiraishi Corp.
Renewal Construction Period: Mar. 2010 – Mar. 2011
Location: Shizuoka Prefecture, Japan

1. Introduction
The Hamana Bridge was completed in 1976, and about 35 years have passed since then. A recent survey has revealed that the central-hinge section has sagged more than expected because of creep, and the sagging is likely to continue in the coming years. Because the bridge was designed in accordance with the 1973 Specifications for Highway Bridges, it is also necessary to take “B live loads” into consideration. In view of these, in order to meet performance requirements under B live loads, it was decided to structurally integrate the central-hinge section and carry out reinforcement by external tendon. Rigid connection of the hinged segments can also be expected to enhance earthquake resistance.
The seismic design of the Hamana Bridge is based on the Recommendations for Seismic Design of Highway Bridges (1972)[3]. The seismic performance of the integrated central-hinge section was checked against the Level 2 earthquake ground motion specified in the current Specifications for Highway Bridges and the estimated ground motions of the anticipated Tokai, Tonankai and Nankai earthquakes. As a result, it was found that the superstructure and piers of the bridge would not satisfy the specified seismic performance requirements. It was thus decided to reinforce the superstructure with carbon fiber sheets and strengthen

Fig.1 Current view of the Hamana Bridge

the piers by steel plate or concrete jacketing (**Fig.1**).

2. Seismic Retrofit

The seismic retrofit included two items. One is girder integration consisting of external tendon reinforcement and central-section integration, and the other is girder reinforcement by carbon fiber sheet. The following sections describe these in detail.

(1) External tendon reinforcement

1) Drilling holes for prestressing tendons

In order to rigidly connected protruding anchor blocks to the existing girder webs, holes for prestressing tendons were drilled by using a core boring machine. Hole locations were determined by using a reinforcing bar detector and an X-ray scanner so that re-bar and PC tendons of the existing girder would not be damaged. In cases that holes could not be drilled at design locations, hole locations were altered as appropriate in view of factors such as hole spacing, the distance from the edge, and the locations of the steel members of the protruding external-tendon anchor blocks to be installed later (**Fig.2**).

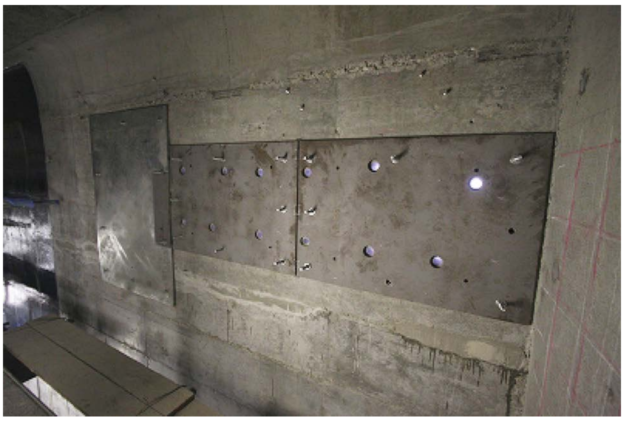

Fig.2 Locations of drilled holes in girder web

2) Installation of reinforcing steel plates

Steel plates were installed for reinforcement against the tensile stress acting on the girder due to the tension of the external tendons. After these reinforcing steel plates were brought into the girder, they were installed at the specified locations on the existing webs cleaned in advance and were secured in place so as to leave a 5 mm thick space for epoxy resin grouting. Then, the space between the steel plate and the existing web surface was grouted with epoxy resin to bond the reinforcing steel plate to the existing web. **Fig.3** shows a reinforcing steel plate used for protruding anchor block installation. The reinforcing steel plates for protruding anchor block installation were provided with deformed stud dowels to ensure strong connection between the protruding anchor block concrete and the reinforcing steel plate.

Fig.3 Epoxy resin grouting

3) Fabrication of prestressing tendons

The reinforcement pattern for a protruding anchor block is shown in **Fig.4**. One end of each prestressing bars (hereinafter, PC bars) is embedded in the concrete block as a dead anchor, and the tendon is tensioned from outside the girder through sheathing. The tension from the prestressing tendons is used to join together the anchor block and the girder web.

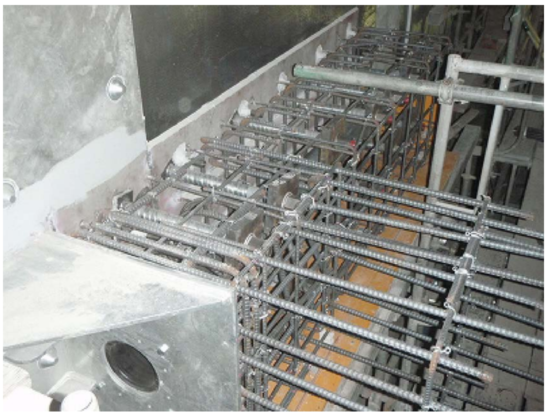

Fig.4 Reinforcement in anchor block

4) Concreting
Each external tendon anchor block has anchorage elements, anchorage reinforcing bars, prestressing tendons by which to join together the anchor block and the girder web, and reinforcing bars for struts for reducing girder web deformation due to the tension from the external tendons. The anchor block is heavily reinforced as shown in **Fig.4**. Accordingly it was judged that complete concreting by use of normal slump concrete would be difficult, and it was decided to use 50 cm slump flow concrete[4]. **Fig.5** shows the slump flow of the concrete used.

Fig.5 Concrete with slump flow of 50 cm

5) Installation external tendons
External tendons (19 × ϕ 15.2mm) were inserted into the girder by using a turn table. Temporary cable supports were provided with felt sheets and a roller conveyor to protect the external tendons. **Fig.6** shows external tendons inserting into the box girder.

Fig.6 Insertion of external tendons into box girder

6) Tensioning external tendons
The external tendons were tensioned from both ends. One tensioning team consisted of hydraulic jacks, hydraulic pumps and tensioning crews, the tensioning took longer time than in normal tensioning work because it had to be carried out in a confined space. **Fig.7** shows the condition of tension work in confined space.

Fig.7 Tensioning work in confined space

7) Integrating the central-hinge section
The central hinge had a gap of 100 mm and consisted of restrainers to transfer shear force (**Fig.8**) and rolling leaf type expansion joints. In the central-hinge integration work, the lanes were closed to traffic, and, after the expansion joints were removed, concrete was placed and prestress was transferred by means of external tendons to connect the hinge section rigidly.

Fig.8 Hinge restrainer

8) Removing expansion joints
To remove an expansion joint, a cut was made, with a concrete cutter, in the transverse direction along the external edge of the removal zone. Then, by using two concrete wire saws, the expansion joint was cut about 1 m long pieces in the bridge axis direction and the cut pieces were removed by crane.

9) Concreting of the central-hinge section
The concrete for the central-hinge section was placed from the upper deck slab by using a pumper truck located on the bridge deck.

(2) Carbon fiber sheet reinforcement
Carbon fiber sheet reinforcement was provided to the inside surfaces of the box girder webs. Internal reinforcement of the box girder with carbon fiber sheets eliminates the need for a suspended scaffold that would require traffic control and for the coating of epoxy resin contained in the carbon fibers for

protection from ultraviolet radiation. Although the construction site is located in a windy coastal area and the construction work continued until winter, the work was successfully completed without being affected by weather conditions.

Three types of carbon fiber sheets having weights per unit area of 200 g/m^2 (guaranteed tensile strength = 377 kN/m), 300 g/m^2 (guaranteed tensile strength = 566 kN/m) and 400 g/m^2 (guaranteed tensile strength = 755 kN/m) were used, and carbon fiber sheets of each type were bonded in one layer or two layers. Thus, in total, six types of carbon fiber sheet arrangements were used for reinforcement.

The reinforcement was carried out in the following steps: (1) cleaning the surface with a sander, (2) applying epoxy resin primer, (3) leveling the surface by applying epoxy resin putty, (4) applying a subcoat of impregnating resin, (5) bonding a carbon fiber sheet and (6) applying an overcoat of impregnating resin. In the case of two-layer reinforcement, Steps 1 to 6 were repeated again.

To anchor the carbon fiber sheets to the existing structure, embedment-type special anchors for carbon fiber were used. An embedment-type anchor is a bundle of 40 to 130 carbon fiber strands; one end of an embedment-type anchor is embedded in concrete, and the other end is fanned out and bonded to the carbon fiber sheet surface. **Fig.9** shows carbon fiber sheet reinforcement.

Fig.9 Carbon fiber sheet reinforcement inside box girder

3. Conclusion

One of the aims of this seismic retrofit project was to reduce maintenance cost structural integrating the hinge-section, which was more prone to deterioration and damage than the other parts of the bridge. Another aim was to ensure safety from the long-expected Tokai earthquake by means of carbon fiber reinforcement. Preventive maintenance and appropriate maintenance to extend the service life of infrastructure are essential in infrastructure projects in the coming years. The authors hope that this report will contribute, even in a small way, to similar projects in the coming years.

In closing, the authors would like to thank Mr. Tachikawa, Director of Hamamatsu Maintenance Branch Office, Hamamatsu National Highway Office, Chubu Regional Bureau, Ministry of Land, Infrastructure, Transport and Tourism, and all other people concerned for the kind cooperation in connection with the seismic retrofit work.

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概　要

国道１号線浜名バイパスに架かる浜名大橋は，昭和48年（1973年）から昭和51年（1976年）にかけて設計・施工された我が国有数の５径間連続有ヒンジラーメン橋である。その全体像は，静岡県浜名湖付近を通過する東海道新幹線の車窓からもよく望むことができる。この構造形式の橋梁は，当時の構造設計における解析レベルから1960年～1980年代にかけて数多くが設計・施工されている。しかし，有ヒンジ部がもつ特有の構造から，現在では多くの同種橋梁で，中央ヒンジ沓の経年的な劣化や，あるいは当初想定されている値を超える橋体の垂れ下がりが生じるなど，メンテナンス性や走行性への問題がクローズアップされている。そのため浜名大橋においては，対応策として中央ヒンジ部の連結や構造系変更に伴う耐震補強を行う必要があった。本稿は，浜名大橋における耐震補強工事の施工報告を行うものである。