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橋

【 *h a s h i* 】

“橋 (*hashi*)” in Japanese, or bridge in English

Bridges constitute the most important element of social infrastructure. This issue, No. 42, discusses the emerging trends in the design standards for bridges, the maintenance of steel highway and railway bridges, and the retrofitting of RC slab bridges to steel deck bridges.

In this issue.....

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Design Standards for Civil Engineering Structures

—Performance- and Reliability-based Designs, and Future Directions—

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This article discusses the design standards for civil engineering structures in Japan with the following two objectives: to briefly introduce the trends of the last 15 years and to prospect future directions. The key terms of this discussion are performance-based design and reliability-based design.

Here, the term "performance-based design" conforms to the definition provided in JGS4001-2004[1], which states, "Performance-based design: a design concept in which structures are not designed based on descriptive characteristics, but are designed by specifying the performance required by society."

Details of Reliability- and Performance-based Designs

• Introduction of Design Standards Based on Reliability-based Design

The introduction of reliability-based design, or the limit state design method, dates back to the 1970s. At that time, efforts were launched in Europe to prepare design standards for buildings and infrastructure facilities that were commonly used throughout the European community of nations. The standard thus prepared is the currently prevailing Structural Eurocode. Based on the concept of "putting new wine into fresh wineskins," the Eurocode introduced a verification method based on reliability-based design, which had just been compiled. It is well known that the Eurocode was completed and put into effect in 2010 after nearly 40 years of preparation.

Established in 1983 in North America, the Ontario Highway Bridge Design Code (OHBDC) is a full-scale design standard based on reliability-based design. Triggered by the establishment of this code, preparation began in the latter part of the 1980s on Bridge Design Specifications (BDS), which was issued by the American Association of State and Highway Transportation Officials (AASHTO) and is the most influential design standard in the field of civil engineering structures in North America. Then in 1995 the LRFD (load and resistance factor design)-version of AASHTO BDS was issued and has since undergone regular revision.

In spite of steady movement in Europe and North America, Japan trailed far behind in the establishment of design standards that rely on partial factor-type verification formulae based on the reliability-based design method. It was the *Technical Standards for Port and Harbor Facilities* in Japan issued in 2007 that was Japan's first design standard for civil engineering structures clearly based on reliability-based design.

Specifications for Highway Bridges, the most influential reference for the design of civil engineering structures in Japan, currently does not adopt the partial factor design method. However, it is now under vigorous revision that will likely be completed in the near future and will widely incorporate verification formulae based on the LRFD approach.

Japan lags behind Europe and North America in the introduction of reliability-based design in design standards, but in its introduction in Japan there is a feature that differs distinctly from other countries. That is, in Japan's design standards, great importance is attached to the framework called performance-based design, and then the reliability-based design is positioned as the performance verification method and incorporated into the standards. Introduction of performance-based design is explained in the following:

• Details in Introduction of Performance-based Design

When examining design standards worldwide, there are at least two basic roots in performance-based design. One originates in the Nordic 5 Level System[2] (Fig. 1). In order to promote international harmonization of building standards, in its concept, the required performances are stratified and divided into two layers: mandatory requirements and supporting documents (guidance). This concept is closely related to a statement in the Agreement on Technical Barriers to Trade of World Trade Organization (WTO/TBT Agreement): "technical regulations based on product requirements in terms of performance rather than design or descriptive characteristics (Article 2.8)." Another concept originates in the proposal for a required performance matrix (Fig. 2) that is shown in Vision 2000 of the Structural Engineers Association of Califor-

Fig. 1 Two Basic Roots in Performance-based Design

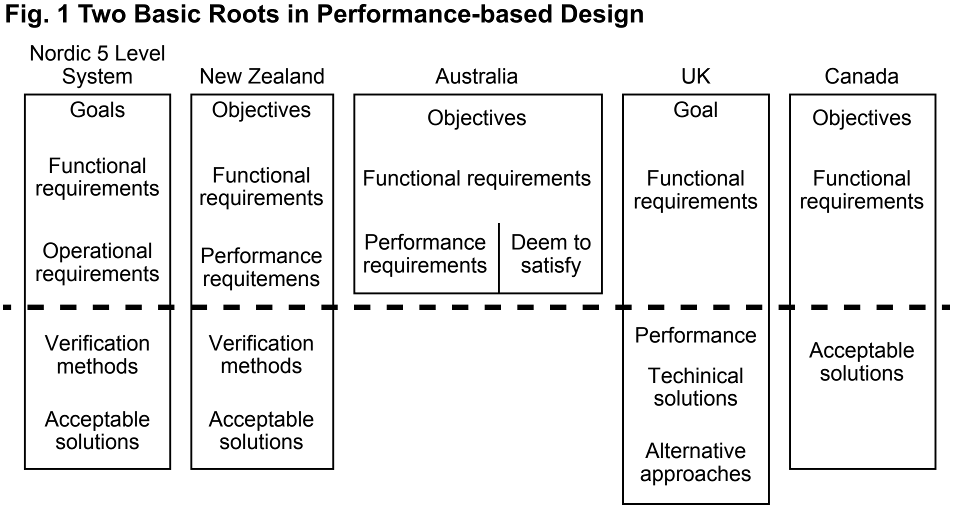


Fig. 2 Performance Matrix in Vision 2000 of SEAOC

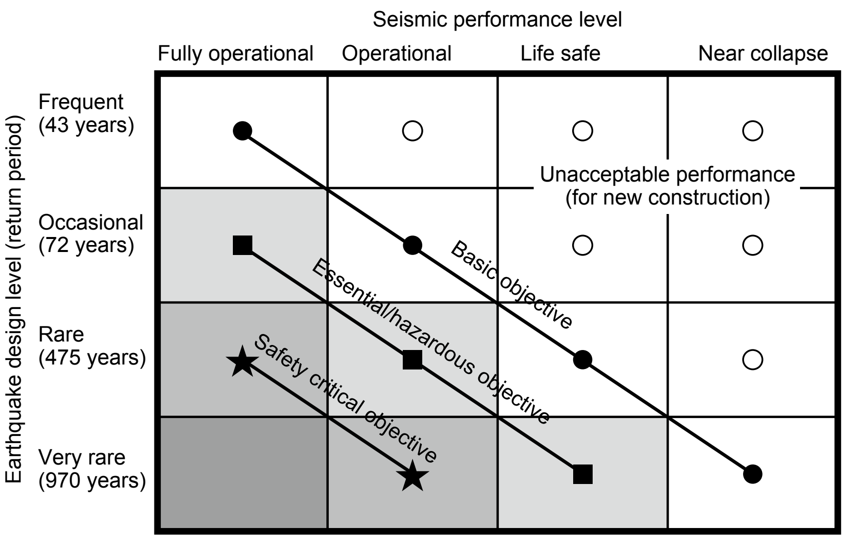
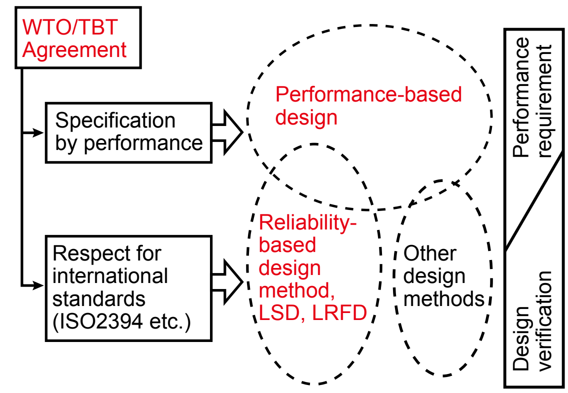


Fig. 3 WTO/TBT Agreement and Performance- and Reliability-based Designs



nia (SEAOC, 1995)[3]. Triggered by the damage experienced in both the Northridge and Loma Prieta Earthquakes, the proposal was worked out as a means to promote dialogue between building owners and structural engineers regarding seismic resistance.

Currently in Japan, there is a movement to establish a design standard that primarily incorporates the verification of structures by means of reliability-based design that is based on the performance-based design concept. It can be said that the first step in this movement was attributable to the WTO/TBT Agreement that went into effect in 1995. Taking this opportunity, the Japanese government implemented a wider deregulation policy. As a link in deregulation, several policies were promoted such as the international conformance of various industrial standards in Japan, specification of performances and elimination of repetitive inspections.

To meet such trends, the Japanese engineering societies quickly responded to the movement to introduce the performance-based design concept proposed in the WTO/TBT Agreement into Japanese design standards. Among the practical attainments in this field were Geo-code 21 that was established by the Japanese Geotechnical Society in 2004 and the code PLATFORM[4] by the Japanese Society of Civil Engineers in 2003, both of which affected subsequent revisions of various design standards.

Fig. 3 conceptually shows the position of performance- and reliability-based designs in the design standards and the relation between them and the WTO/TBT Agreement and ISO standards. The performance-based design framework is based on the agreement in terms of international commerce policies. Accordingly, it is believed that, in the future preparation of design codes, the performance requirements of structures will be indicated by performance criteria, and that the verification of performance criteria will use the reliability-based design approach prescribed in ISO2394 and other international standards.

Current Design Standards for Civil Engineering Structures in Japan

As stated above, the *Technical Standards for Port and Harbor Facilities in Japan* was revised in 2007 and enforced, which basically adopts performance verification by means of the partial factor method based on the reliability-based design approach. An outline of the revision has already been introduced, for example, in an English technical paper by Nagao et al. (2009)[5] and is therefore not discussed in this article.

Revision work is underway on *Specifications for Highway Bridges in Japan*, a technical standard for the highway structures that account for the largest share of civil engineering structures in Japan. In this connection, the major concepts in the revision of the *Specifications* are introduced here[6]:

Fig. 4 shows the concept of performance *specifications* for highway bridges, shown in the draft for the current revision of the Specifications. While the figure shows the concept that specifies the performance pertaining to the loading capacity of bridges, the performance requirements of bridges are shown using a performance matrix composed mainly of two conditions: the bridge limit state (description of the performance at the limit state) and the design situation (i.e. the loading conditions to be taken into account in the design work: combination of permanent, variable and accidental loading conditions). Then, it is required that the performance requirement be satisfied by means of "specified assurance" within the design service period (normally 100 years). The specified assurance described here means the reliability, and it is understood that the performance specification in the *Specifications* under revision is based on the concept of reliability-based design.

The lower section of Fig. 4 shows the concept of performance verification by means of the partial factor method, and the combination of design situation and limit state specified in the matrix is verified by means of the verification formulae using the partial factor method. The format used in the verification formulae is prepared based on load and resistance factor design (LRFD).

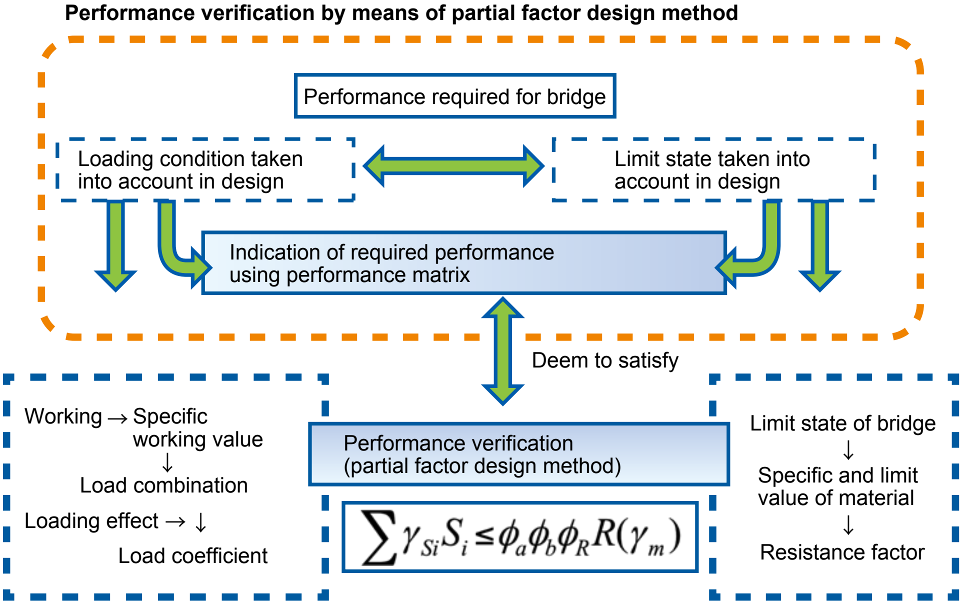
Currently, revision of the *Specifications for Highway Bridges* is being promoted based on the concept shown in Fig. 4. Specifically, the work is underway to develop the practical verification criteria of the required performance defined in the matrix in Fig. 4 pertaining to each structural member. Diverse engineering judgments are required in the process. To this end, the concept "deem to satisfy" is emphasized in the course of the revision. The term refers to whether or not a specifically required per-

Fig. 4 Concept of Performance Specification for Highway Bridges Shown in Revision of Specifications for Highway Bridges in Japan

Performance matrix of loading capacity

Bridge limit state / Loading condition to be taken into account in design	No occurrence of deterioration and damage even in partial section of entire bridge; and no impairment of function as a bridge (service limit state)	Occurrence of deterioration and damage even in partial section of entire bridge, but limited effect on function as a bridge and quickly recoverable in function (recovery limit state)	Considerable impairment of function as a bridge and difficult in quick recovery of function, but not fatal in bridge collapsing (ultimate limit state)
Permanently governing	To be satisfied with specified assurance at optional time in design service period		
Variably governing			
Accidentally governing			

Performance verification by means of partial factor design method



formance that is satisfactorily verified by the use of a specified verification method is deemed to satisfy the performance requirement. At the same time, it is important that there remains some allowance in the meaning of "deem to satisfy." That is, the specified verification method is not necessarily the only verification method, but, rather, other verification methods that are deemed to be appropriate by structural designers can also be applied.

Future Directions

A widely prevailing trend in the design standards for major civil engineering structures in Japan is that performance-based design is basically incorporated in the design standards and that performance verification is conducted employing the partial factor method that is based on the reliability-based design approach and the load and resistance factor design (LRFD) method. The concept "design standards based on performance-based design" is peculiar to Japan and is not found in the Eurocodes or the Standard Specifications for Highway Bridges of AASHTO.

As discussed above, this concept was initially introduced as a link in the deregulation policies taken by the government as a result of the enactment of the WTO/TBT Agreement and has continued to develop since then. However, when examining the revision of the *Specifications for Highway Bridges in Japan* now underway, it seems that the framework of the concept is developing in new directions.

Like the United States and other advanced nations, Japan possesses a huge stock of social infrastructure. Their construction accelerated from the 1960s and reached massive numbers in the period of rapid economic growth from the 1970s to 1980s. Currently, it is widely accepted that the maintenance of these massive social infrastructure facilities will be of great concern in the future.

Given this situation, it seems that those who are responsible for maintenance at the Ministry of Infrastructure, Land, Transport and Tourism and at other government agencies foresee difficulty in uniformly maintaining these superannuating infrastructures at identical levels. In this connection, it is considered that the concept of performance-based design will potentially allow the formation of a social consensus in discriminating the performance of various structures, which should not be overlooked when prospecting the future development of performance-based design. ■

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Report on Repair of Fatigue Cracks of the Orthotropic Steel Decks in the United Kingdom

by Shigeyuki Hirayama, Highway Technology Research Center; Susumu Inokuchi, Yokogawa Bridge Corporation; Daisuke Uchida, Mitsui Engineering & Shipbuilding; and Atsunori Kawabata, JFE Engineering Corporation

Fatigue cracks around the welded joints between deck plate and trough rib of orthotropic steel bridges which initiate at weld root and propagate through weld bead can be seen in Japan. Recently, these cracks which are called as bead-through cracks occurred at the bridges suffered from severe traffic condition. Sometimes they can propagate toward trough rib or deck plate and make damage to bridge function.

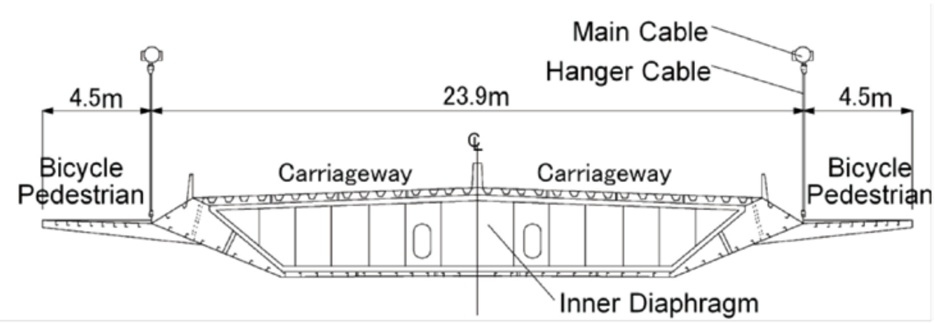
The first observation of bead-through cracks in Japan was reported by Hanshin Expressway in 1993. As the accuracy and technique of bridge inspection were improved, the number of bead-through cracks has been increased. Many organizations have conducted experimental or analytical studies for them. On the other hands, the first observation of bead-through cracks in the world was reported at Richemont Bridge, France, in 1970's. Also Severn Bridge in England is famous as the bridge where bead-through cracks occurred at the same term and they have been repaired by re-welding. We also have studied about the initiating mechanism or the repair method for bead-through cracks. Then ten-day field visit about the repair for fatigue cracks of orthotropic steel bridge such as Severn Bridge in the United Kingdom was carried out in April 2011. The field visit at Severn Bridge and Erskine Bridge was carried out. This paper reports field visit on the fatigue problem of orthotropic steel bridges in the U.K.

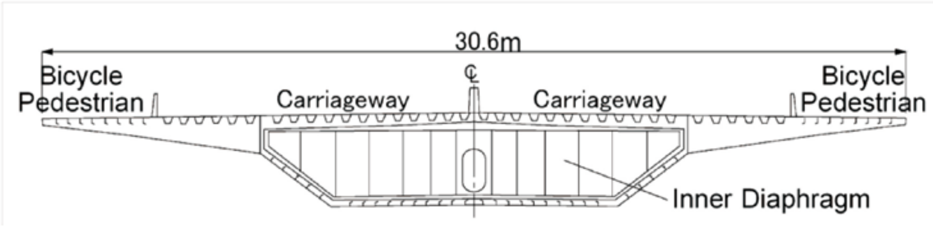
Outline of Severn Bridge and Wye Bridge

Severn Bridge is the suspension bridge crossing Severn River in southwest of U.K. Severn Bridge is called as Severn Crossing with Wye Bridge, Aust Viaduct and Beachley Viaduct which cross the boundary between England and Wales. Severn Bridge was opened in 1966. The bridge length is 1,600 m and main span length is 988 m (Photo 1). The box girder with stream-

Photo 1 Severn Bridge

Fig. 1 Cross Section of Severn Crossing

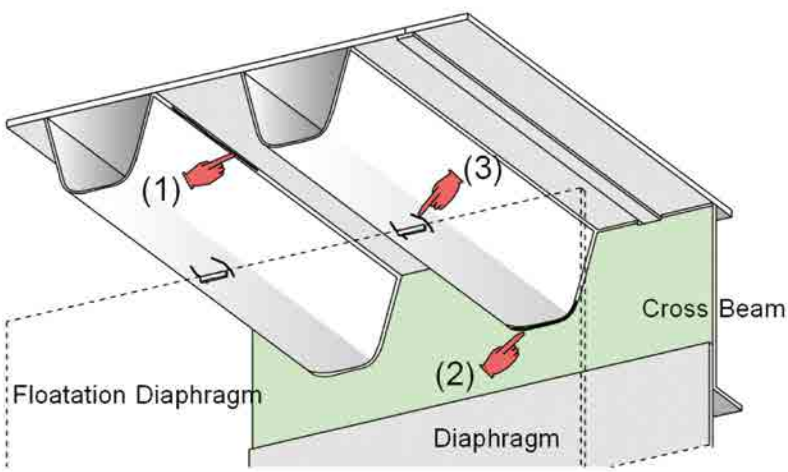




lined section was adopted as stiffening girders based on results of the wind tunnel testing. Pedestrian and bicycle lanes are located each side of the main girder and they are also used as lanes for maintenance vehicles (Fig. 1). Thickness of deck plate is 11.5 mm and shape of trough rib is (B) 305 mm × (t) 6 mm × (H) 230 mm. Inner diaphragms are located every 4,600 mm and trough ribs are welded round with inner diaphragms. Field joints of deck plate are welded joints in the each longitudinal and transverse direction. Surfacing is mastic asphalt of 35 mm thickness. Wye Bridge is the single-plane cable stayed bridge with twin towers and their structural dimension is same as Severn Bridge.

In 1977 when eleven years have gone after opening of Severn Bridge, fatigue cracks were observed at orthotropic steel deck. These cracks could be classified into three by its locations, (1) welded joints between deck plate and trough rib, (2) cross section between trough rib and cross beam, and (3) welded joints between bottom of trough rib and floatation diaphragm (Fig. 2). Floatation diaphragms were attached each end of box girder block in order to prevent water leakage into it when floated to transport on the water of Severn River. Because they were not removed away after construction, they caused fatigue cracks around welded joints between bottom of trough rib and floatation diaphragm. At first, fifteen fatigue cracks at welded joints between deck plate and trough rib were observed, and then over 160 cracks were observed by 1985. All cracks were located under the first traffic lane where almost large vehicles pass.

Fig. 2 Fatigue Cracks Observed at Severn Crossing



(1) Welded joint between deck plate & trough rib
(2) Cross section between trough rib & cross beam
(3) Welded joint between trough rib & floatation diaphragm

Re-Welding Repair at Severn Bridge and Wye Bridge

Many studies were conducted by TRRL (Transport and Road Research Laboratory) to decide the method to repair fatigue cracks. These studies indicated that fatigue strength of the welded joints between deck plates and trough ribs was improved by increasing throat thickness. Then re-welding repair at Severn Bridge started in the latter half of 1980's in order to keep enough throat thickness.

When fabricated, the edges of trough rib were cut to be parallel to the under face of deck plate. In re-welding process, they were cut (not grooved as we did usually) again to be vertical to it in order to make partial penetrate welding and keep enough throat thickness. The original cutting machine was developed (Fig. 3). This cutting machine could move longitudinally every one meter and the cutter supported by two independent guide flames was located in order to follow to deformation of deck plate and web of trough rib. The welding conditions were decided by the field trial tests of fifty one times with thirteen conditions and re-welding repair was conduct-

Fig. 3 Original Cutting Machine

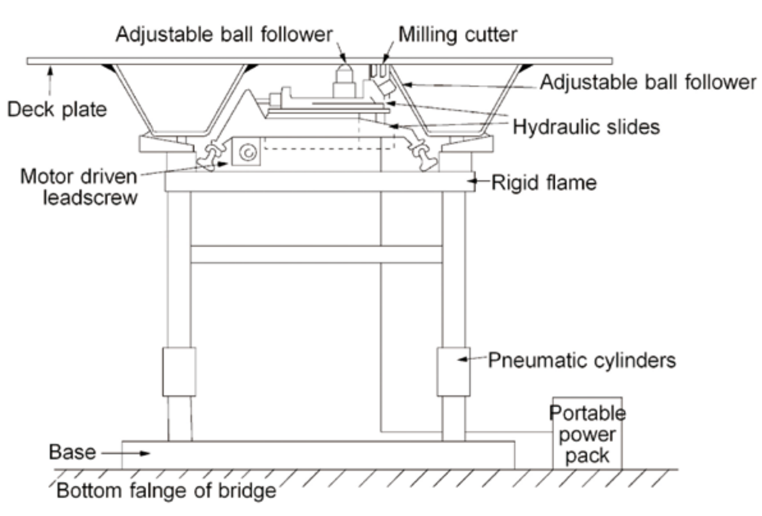
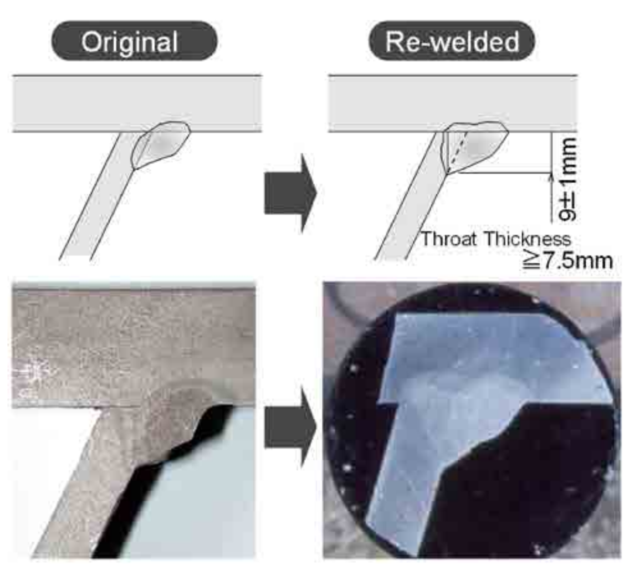


Fig. 4 Macro Etching Test for Welded Joints of Original and Re-Welded



ed to keep throat thickness of 7.5 mm with three welding passes according to the results of these tests (Fig. 4). The total length of re-welding became to be about 20 km, which had been conducted for the welded joints located under the position of vehicle's wheels. The repair was carried out while the traffic lanes were limited in order to avoid traffic inconvenience by traffic closing, then the period for repair became to be eighteen months.

Field Visit at Severn Bridge and Wye Bridge

In the field visit at Severn Crossing, we observed the condition of re-welded joints in the box girder and the condition of asphalt surfacing. We felt that the traffic volume was very little. The reason is that the traffic volume of about 70% may pass through Second Severn Bridge which was opened in 1996 at the downstream of Severn Bridge. Many patches by re-casting of asphalt surfacing which was re-cast in 1991 could be seen at the position of vehicle wheel (Photo 2). In the field visit inside the box girder, we observed the condition of re-welded joints between deck plate and trough rib, and also between trough rib and diaphragm, welded joints between bottom of trough rib and floatation diaphragm which had been repaired (Fig. 5).

For re-welding by three passes, it seemed that dispersion of weld size between deck plate and trough rib was smaller and their quality was better than the original. Welding repair with the metal backing plates had been carried out for the fatigue cracks at the cross section between trough rib and diaphragm. To repair the larger fatigue cracks at the welded joint between trough rib and floatation diaphragm, the bottom of trough rib was cut away partially and metal bypass materials were attached. The flotation diaphragms had been removed away when the fatigue problem came out.

The general inspection every two years and the principal inspection every six years are conducted for the long span bridges in the U.K. such as Severn Bridge. Fatigue cracks or defects found by general inspection are repaired in each occasion. Although number of fatigue cracks which are observed newly has decreased because of decrease of traffic volume by open of Second Severn Bridge, about fifty cracks or failures are still repaired every year.

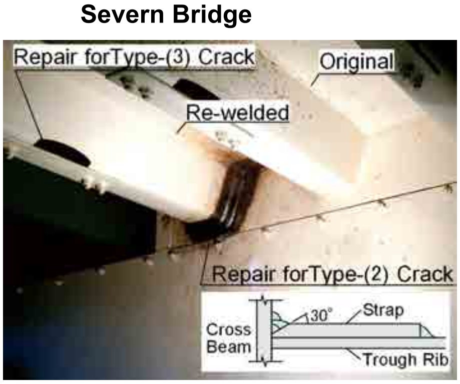
Outline of Erskine Bridge

Erskine Bridge is the single-plane cable stayed bridge which is located over Clyde River in West Glasgow of Scotland (Photo 3). The length of bridge is 1,321 m and it was opened in 1971. This bridge consists of the main route toward Scotland and then

Photo 2 Condition of surfacing at Severn Bridge

Photo 3 Erskine Bridge

Fig. 5 Repair of Fatigue Cracks at Severn Bridge



the traffic volume is over 4,000 per day. It was clear that this bridge did not satisfy the criteria shown later by Merrison Report and reinforcing and re-strain of the cables were carried out from 1970's to 1980's and in 2004 (Photo 4). As orthotropic deck system in Erskine Bridge, thickness of deck plate is 12.7 mm and V-shaped rib is used as longitudinal rib of 5 mm thick. Surfacing is mastic asphalt of 38 mm thick as same as Severn Bridge.

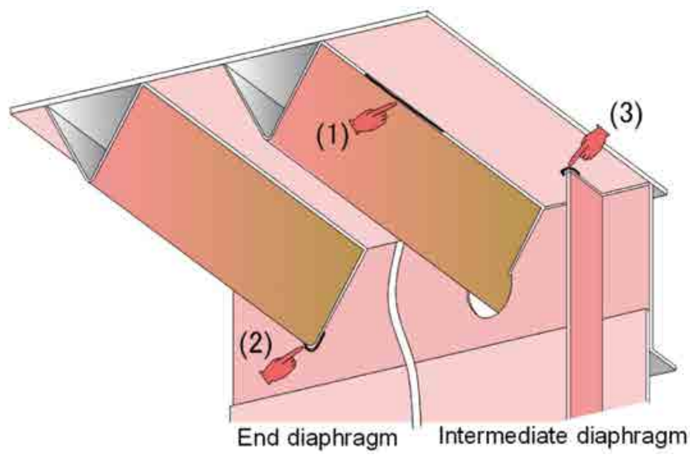
Photo 4 Reinforcement based on Merrison Report

Field Visit at Erskine Bridge

Fig. 6 shows the main three types of fatigue cracks which had been observed at Erskine Bridge; (1) welded joint between deck plate and trough rib, (2) welded joints between trough rib and end diaphragm, and (3) welded joint between deck plate and vertical stiffener. Over 1,200 cracks or defects including also ones observed without orthotropic deck have been repaired in Erskine Bridge. In the field visit, we observed these fatigue cracks of three types.

Photo 5 shows the sample of crack which occurred at the weld bead which we call "bead-through crack." About ten bead-through cracks have been observed in this bridge and some of them have been repaired by cutting the edge of trough rib and re-welding by three passes as same as Severn Bridge. Although the length of the fatigue crack shown in Photo 5 is over 700 mm, the crack does not propagate toward the web of trough rib as the case observed in Japan. The reason seems to be that welding condition and cutting at the welded joint between deck plate and trough rib and dimension or stiffness of trough rib are different between the U.K. and Japan and so on.

Fig. 6 Fatigue Cracks Observed at Erskine Bridge



(1) Welded joint between deck plate & trough rib
(2) Welded joint between trough rib & end diaphragm
(3) Welded joint between deck plate & vertical stiffener

Conclusions

We could know fatigue problem of orthotropic decks and stance for maintenance of them in the U.K. through this field visit. Especially for the long span bridges we visited, private companies have engaged in maintenance work for long periods, and we felt their stance that they kept watching the bridges intently in the future. It is meaningful that the same engineers can conduct usual observation, inspection, and repair work systematically, and we should refer them in Japan. ■

Photo 5 Bead-through crack at Erskine Bridge

Maintenance and Reinforcement of Steel Bridges on the Tokaido Shinkansen

by Hideki Kaji, Shinkansen Operations Division, Kazuya Takahashi, Chuo Shinkansen Promotion Division and Yuichi Ito, General Technology Division, Central Japan Railway Company

A total of 22.1 km of steel railway bridges are installed on the Tokaido Shinkansen (bullet train line). The construction of these bridges features the extensive use of fully welded steel structures, the first full-scale adoption of this method in Japan.

Because steel bridges respond more dynamically to train loads than concrete bridges and because the trains are now running at a greater frequency compared to when the line opened, the task of dealing with weld zone fatigue and durability is emerging as an important issue. In particular, because the total length of the longitudinal weld beads (hereinafter referred to as longitudinal beads) of the major structural members is extensive, it is difficult to focus on only specified sections during inspection. Further, once a crack initiates in a bead, it quickly propagates. For this reason, it is essential in steel bridge maintenance to suppress the occurrence of fatigue cracks in longitudinal beads (Fig. 1).

To achieve this goal, fatigue loading tests and three-dimensional FEM analysis were implemented using a full-scale bridge model or the members of existing steel railway bridge at the Komaki Research Laboratories owned by the Central Japan Railway Company. This led to the development of new steel bridge reinforcement techniques for use in preventive maintenance, such as the reinforcement of floor system joints, the replacement and reinforcement of bearing supports, and the insertion of additional sleepers. These techniques are to be adopted in a large-scale project to retrofit the steel railway bridges of the Tokaido Shinkansen and are introduced below.

Then, following the implementation of these countermeasures to suppress degradation, the company plans to confirm the service conditions of the affected bridge structures and then determine the intervals at which the members of these structures should be replaced as a general retrofitting measure.

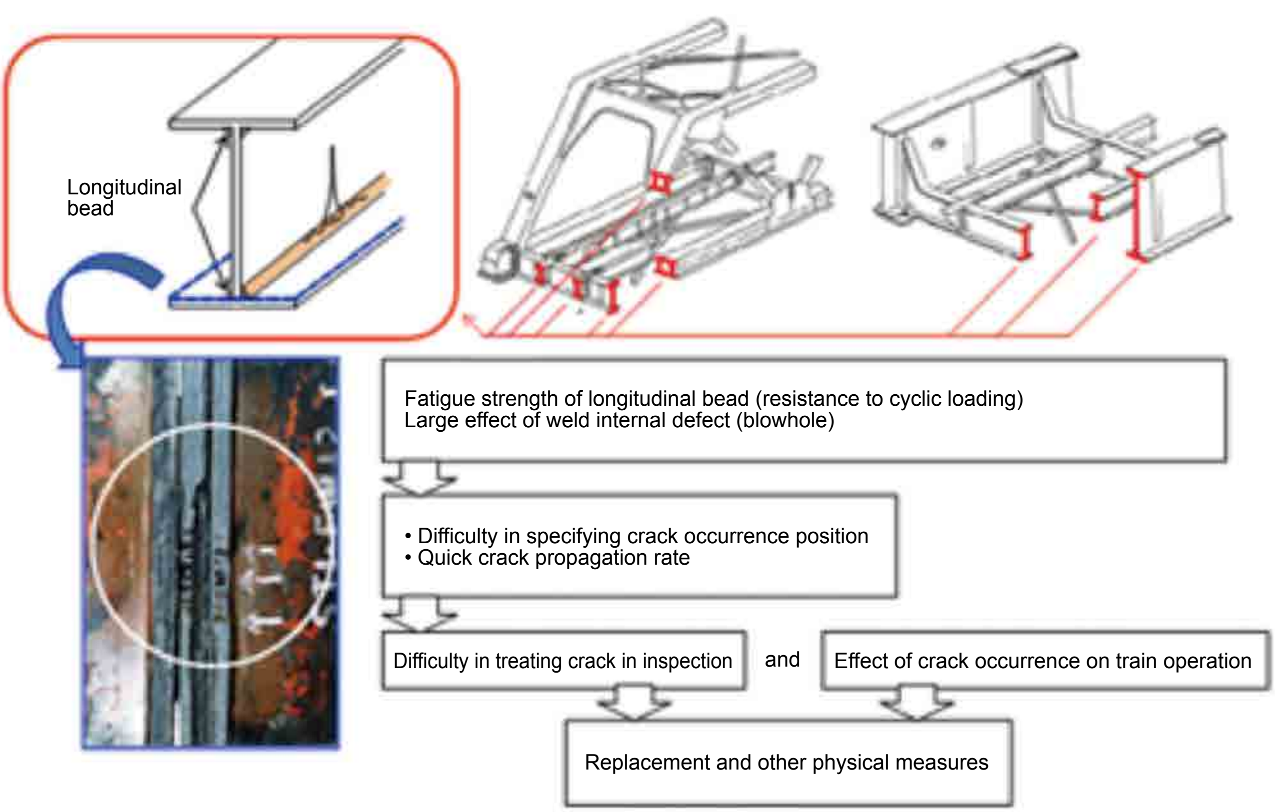
Maintenance and Reinforcement of Steel Railway Bridges

• Assessment of Longitudinal Beads

It is known that the fatigue strength of a longitudinal bead is affected by internal defects in the weld zone. According to the results of surveys of actual bridges using ultrasonic testing and as a result of the dismantling of actual bridges removed from the Tokaido Shinkansen, it is accepted that the existence of a certain level of blowholes is an undeniable fact.

At the Komaki Research Laboratories, fatigue tests were conducted in the following two cases: one test using a steel bridge that was in service for more than 35 years following the startup of the Shinkansen and removed after installation of the new Shinagawa Station in the fall of 2003[1]; and another test using a large girder specimen that was in-

Fig. 1 Longitudinal Weld Bead



tentionally infused with blowholes (Photo 1). The test results for both the full-scale bridge specimen and the girder specimen satisfy the "Class D" designation prescribed in the fatigue design guidelines of the Japanese Society of Steel Construction and other organizations (Fig. 2). And the developed stress measured in all the steel bridges on the Tokaido Shinkansen falls below the Class D fatigue limit, even when taking into account the variation coefficient of the train wheel loads commonly forecasted in train operations. However, in cases when a bridge structure undergoes changes due to other types of damage and, as a result, the developed stress increases and in cases when shocking fluctuations in wheel load occur depending on the maintenance conditions of the rails, sleepers and other structures, it is possible that the developed stress will surpass the fatigue limit of the longitudinal beads[2)].

(a) Test for floor system

(b) Test for single main girder

Photo 1 Fatigue testing condition

Fig. 2 Fatigue Test Results for Longitudinal Bead

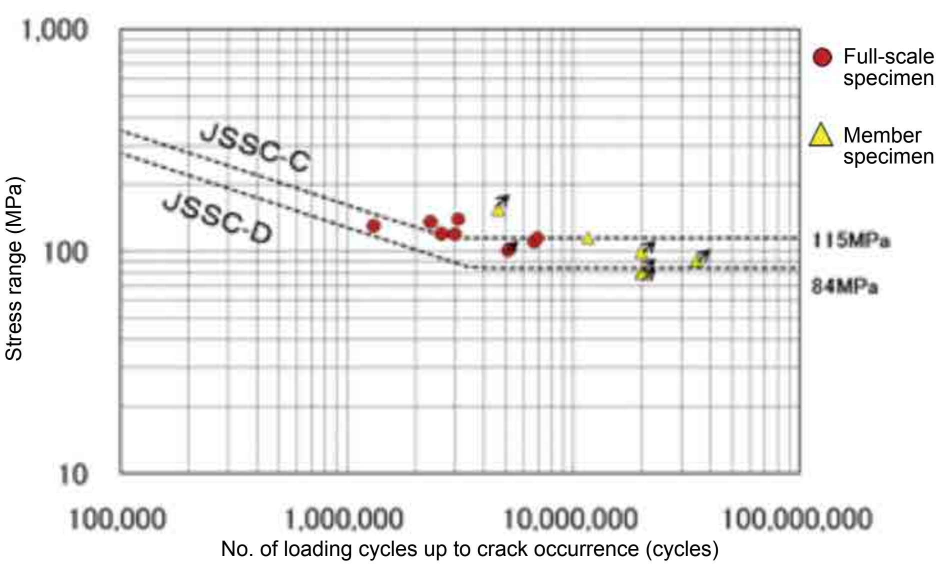
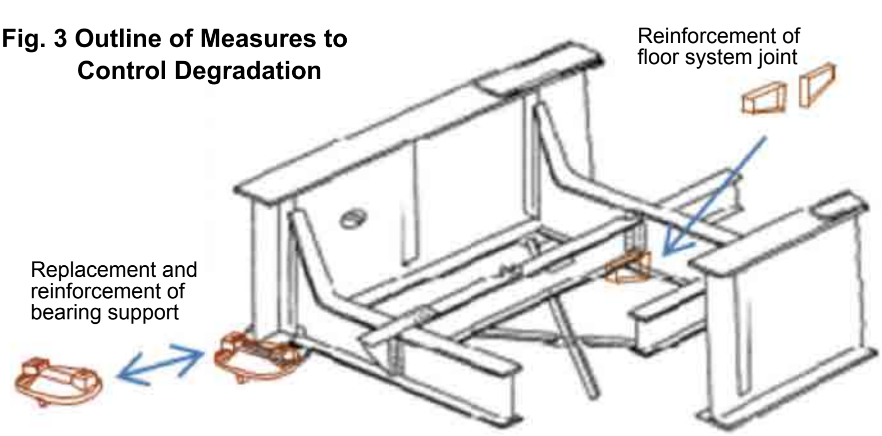


Fig. 3 Outline of Measures to Control Degradation



Two approaches have been developed to suppress the increase of stress in longitudinal beads by preventing the occurrence of the deficiencies that cause changes in bridge structure. One is to reinforce the joints of the floor system (stringers and cross girders) and the other is to replace and reinforce the bearing supports (Fig. 3). These approaches are introduced below:

Reinforcement of Floor System Joints

It has been confirmed in surveys of actual bridge conditions thus far conducted that fatigue cracking has occurred in the floor system joints of part of the through truss and the through plate girders. The through truss, for example, has cracks that initiates in the boxing weld zone at the stringer pedestal and in the boxing weld zone within the slit at the cross section of the pedestal and the vertical stiffener or cross beam. It has been confirmed by means of three-dimensional FEM analysis that such cracks increase the longitudinal bead stress and tend to induce cracks that initiates in the longitudinal beads. In response, we proposed a reinforcement method to suppress the occurrence of cracks at the floor system joints[3)].

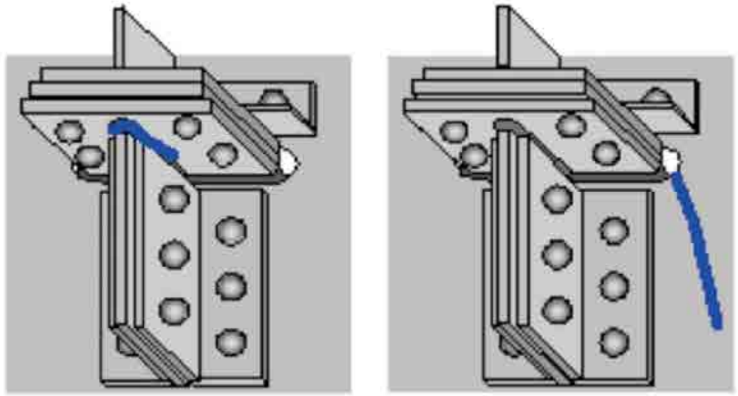
• Reinforcement of Floor System Joints of the Through Truss

It was previously confirmed that several kinds of cracks develop because of problems involving the details of the floor system joints.

First, stresses were measured and the conditions that produced fatigue cracks were analyzed, and then the structural safety issues related to treating fatigue were examined. As a result, it became clear that fatigue cracks could possibly occur in the floor system joints and that the cracks would initiate from two sections: the weld zone at the cross sections of the stringer pedestals and the upper end of the vertical stiffeners, and the boxing weld zone within the slit at the cross sections of the stringer pedestals and the cross beams (Fig. 4).

Next, we proposed a reinforcement method (Photo 2) to control the deformation behavior that caused the above-mentioned cracks. Specifically, static loading and fatigue tests were conducted using a full-scale test specimen of the floor system joints to which reinforcing members were attached. The effectiveness of the attachments was

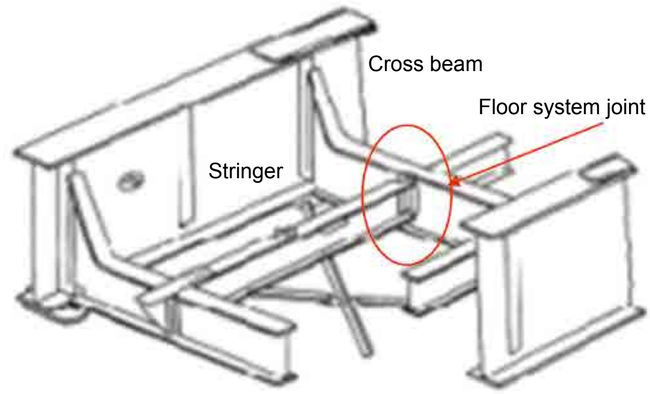
Fig. 4 Fatigue Crack in Floor System Joint (Through Truss)



(a) Crack occurring from stringer pedestal

(b) Crack occurring from boxing weld zone within slit of cross section

Fig.5 Open Deck-type Through Plate Girder

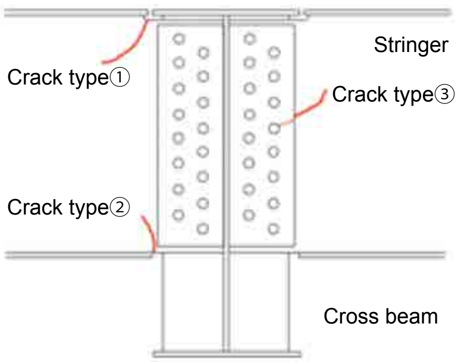


(a) Before reinforcement

(b) After reinforcement

Photo 2 Floor system joint before and after provision of reinforcing measure

Fig. 6 Fatigue Crack in Floor System Joint (Through Plate Girder)



then confirmed as a means to improve fatigue strength, and finally the confirmed method of effective reinforcement was applied to an actual bridge to measure the stress before and after reinforcement. As a result, it was confirmed that, by attaching reinforcing members, stress is reduced to a level below the fatigue limit in the weld zone at the cross section of the stringer pedestals and the upper end of the vertical stiffeners and in the neighborhood of the boxing weld zone within the slit at the cross section of the stringer pedestals and the cross beams.

• Reinforcement of the Floor System Joints of Through Plate Girders

It is reported that fatigue cracks have occurred in the joints of the stringers and the cross beams of open deck-type through plate girders (Fig. 5).

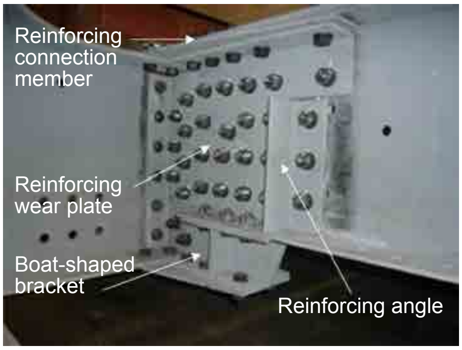
It is known that three types of fatigue cracks have occurred in these joints: ① cracks in the notch section of stringer upper flanges, ② cracks at the notch section of stringer lower flanges, and ③ cracks at the rivet holes of stringer webs (Fig. 6). In cases when these cracks propagate, the structural continuity of the stringers that sandwich the cross beams will be lost, and as a result, it is considered that the stress in the longitudinal beams will increase as the stress increases in the through truss mentioned above.

To solve this problem, a static loading test was conducted to examine methods to reinforce the joints of stringers and cross beams. Then the reinforcing structure that was found to be most effective in reducing stress was selected. Specifically, it was confirmed that by attaching boat-shaped brackets and other reinforcing members to the floor system joints (Photo 3), the stress occurring at the three positions shown in Fig. 6 can be reduced.

Countermeasures for Girder Bearing Supports

It is known from stress measurements at actual bridges that when the functionality of the bearing supports or shoes declines, it is likely to cause a considerable increase in longitudinal bead stress. In particular, in steel bridges in which the sole plate is attached to the girders by welding, stress occurs in the sole plate weld zones and the resulting stress-induced fatigue damage penetrates into the flanges and webs of the main girders, which is cited as a serious deficiency in steel bridge maintenance.

(a) Before reinforcement



(b) After reinforcement

Photo 3 Reinforcement measure employing boat-shaped bracket (through plate girder)

Fig. 7 shows the position where stress was measured on the front surface of the sole plate weld zone before and after shoe replacement, and Fig. 8 shows the measured results. Large stress occurred at the front surface of the sole plate weld zone, and torsion was generated in the lower flange. The stress measured at the shoe before replacement was about 10 times higher than after replacement. Therefore, it is necessary to maintain the soundness of the shoes[4].

Improvement of the Service Life and Durability of Steel Railway Bridges

Fatigue cracks that originate in a longitudinal bead quickly propagate after initiation and are difficult to find during inspection. As an effective measure of dealing with such cracks, it was conventionally considered necessary to quickly replace the affected member. However, as a result of our research, we believe that the service life of steel railway bridges can be prolonged and durability can be improved by the floor system joint reinforcement that suppresses the degradation of those floor system joints and, also, by carrying out measures to maintain the girder bearing supports.

In addition, because exceptionally large wheel loads occur in slab-less railway steel bridges, it is very important to prevent its occurrence. The conventional approach to solving this problem was to thoroughly change the railway structure. But now it is possible to control wheel load fluctuation by installing a plate bearing support structure on the railway in which additional sleepers are inserted among the existing sleepers. A detailed explanation of this new system will be discussed separately.

The service life and durability of the steel bridges on the Tokaido Shinkansen will be improved in the future by applying the diverse reinforcing measures thus far developed. Meanwhile, as regards the necessity to replace railway bridge members, Central Japan Railway Company will continuously monitor the replacement effect and examine the replacement period if the need arises. ■

Fig. 7 Stress Measurement Position at Sole Plate Weld Zone

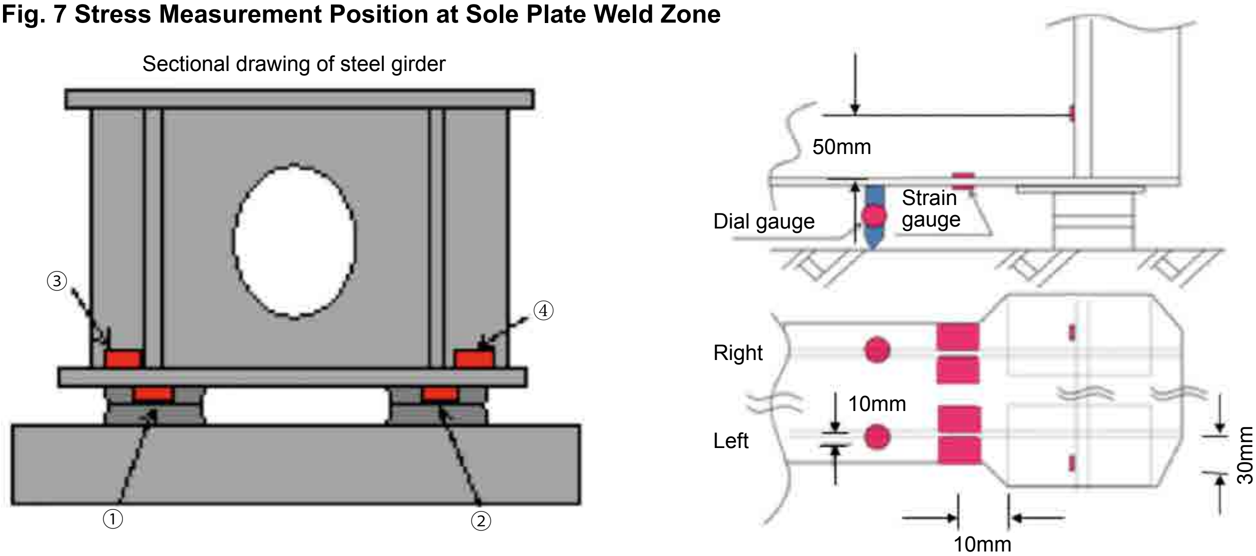
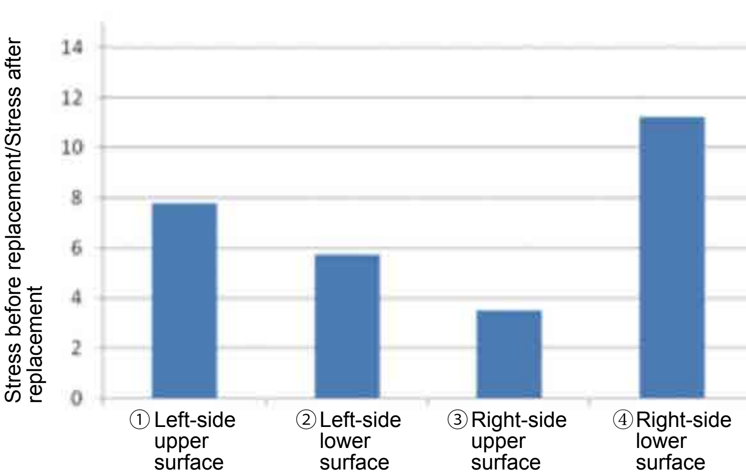


Fig. 8 Stress Ratio Before and After Shoe Replacement



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Maintenance of Metropolitan Expressways

by Masasumi Okada, Kanagawa Operation Bureau, Metropolitan Expressway Co., Ltd., Takao Mizoguchi, Kanagawa Construction Bureau, Metropolitan Expressway Co., Ltd., and Tomohiko Aikawa, Engineering Department, Metropolitan Expressway Co., Ltd.

The Metropolitan Expressway is a network of expressways constructed to alleviate chronic traffic congestion in Tokyo, Japan's capital, and in its surrounding areas. In 1964 when the Tokyo Olympic Games were held, about 33 km had already been put into service. In the nearly fifty years since then, expressway construction paralleled national economic development and in 2010 had reached a total operational length of 301.3 km. In that year, the Metropolitan Expressway registered the highest level of traffic volume in Japan (115,000 vehicles per 12 hours, or 11,000 vehicles per hour).

Maintenance of the 301.5 km-long Metropolitan Expressway is summarized in the following.

Expressway Features

From 1958, the Metropolitan Expressway Public Corporation—which in 2005 was reestablished due to administrative reform as Metropolitan Expressway Co., Ltd., a private company—has managed the construction, operation and maintenance of the Metropolitan Expressway.

The 301-km expressway network is divided into two types of roadway: four-lane (two lanes in each direction) and six-lane (three lanes in each direction).

The Metropolitan Expressway was constructed by maximum use of areas above existing rivers, canals, highways and other public sites. As a result, in terms of the structures that comprise the expressway, flat roads account for only 5% of the total extension, while viaducts, bridges and tunnels account for 95%. Meanwhile, viaducts and bridges account for 80% of the total length, with a total number of 11,800 spans (7,770 composed of steel girders/RC slabs, 1,340 of steel girders/steel decks, and 2,690 of PC and RC girders), and a total of 8,680 piers (2,885 steel piers and 5,795 RC piers).

Currently, 1.8 million people per day use the Metropolitan Expressway, as do one million vehicles, of which full-sized vehicles number 100,000. However, the traffic volume differs according to the section of the expressway network, ranging from 80,000 vehicles/direction/day on six-lane sections to 1,500 vehicles on four-lane sections.

In order to safely inspect, repair and reinforce the expressways, it is necessary to secure workspace by temporarily closing lanes. Further, in order to ease maintenance-induced traffic congestion as much as possible, work conducted in heavily traveled sections is scheduled to avoid daytime hours and is carried out at night when traffic volume is relatively light.

The cost of maintaining and operating the Metropolitan Expressway was ¥61 billion in 2012. Of this amount, the cost of inspection and repair/reinforcement was ¥42 billion, and that of operating the toll roads was ¥19 billion.

Inspection and Repair/Reinforcement Systems

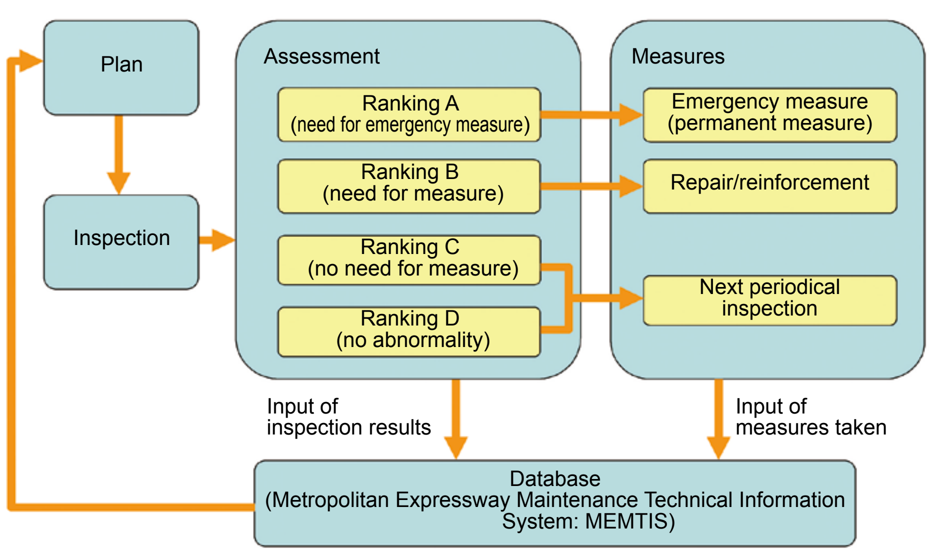
In order to continuously carry out inspection and repair/maintenance work on its expressway network, Metropolitan Expressway Co., Ltd. follows the systematic process shown in Fig. 1: inspection plan→inspec tion→assessment→execution (repair and reinforcement)→inspection plan.

Inspections are divided into three types depending on aim and frequency: routine inspections (vehicular patrol and walking inspections), periodic inspections (close-range and instrumental inspection), and emergency inspections (in response to earthquakes etc.).

• Routine Inspections

These are conducted to observe structural condition and to check for falling objects on roads. The frequency of these inspections varies—from patrol inspections basically every two hours to walking inspections of ex-

Fig. 1 Inspection and Repair System



Ranking A	Damage requiring emergency measure (fear to third-party damage)
Ranking B	Damage requiring planned maintenance
Ranking C	No need for measure because of slight damage (damage is recorded)
Ranking D	No damage (inspection is recorded)

pansion joints and other devices every five years.

• Periodic Inspections

In order to acquire a detailed understanding of the state of structural condition, close-range inspections of structures are, as a rule, conducted every five years. These are the most fundamental and important inspections.

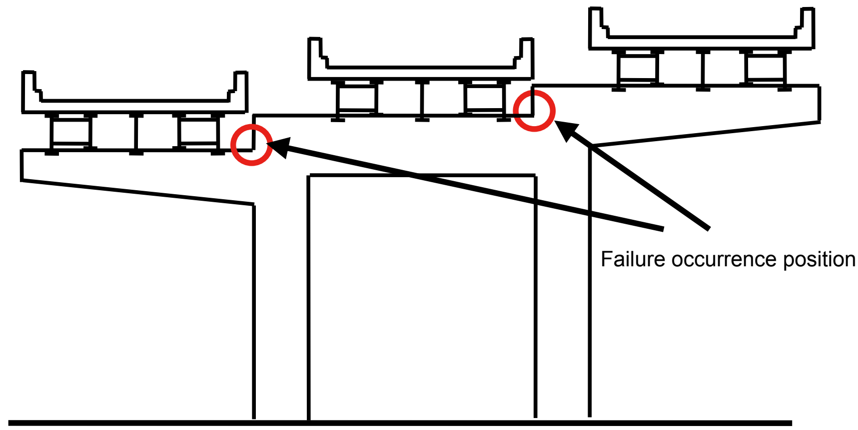
• Emergency Inspections

These inspections are conducted to check for structural damage and to assess damage levels after the occurrence of great earthquakes and torrential rainstorms.

The sections to be repaired are determined by assessing and ranking the inspection results in accordance with our database (METIS: Metropolitan Expressway Maintenance Technical Information System). After the repairs are completed, the repair record is input into the database and saved as data for use in working out subsequent inspection and repair plans.

A routine patrol inspection is shown in Photo 1, a close-range walking inspection in Photo 2, a close-range periodic inspection in Photo 3, and an instrumental inspection in Photo 4.

Fig. 2 Occurrence Position for Fatigue Cracking in the Corner of Steel Bridge Pier



Examples of Repairs and Reinforcements

Repair and reinforcement include the replacement of pavement that exhibits rutting and cracking, and the repainting of steel structures to prevent the corrosion. Recent concerns have focused on the repair and reinforcement of fatigue phenomena in steel structures. Examples of fatigue failure and appropriate countermeasures are introduced below.

Photo 1 Patrol inspection (confirmation of structural condition using a patrol vehicle while running on the expressway)

Photo 2 Walking inspection (walking inspection of structures by inspectors in areas of restricted traffic flow during nighttime)

Photo 3 Close-range inspection (inspection by cherry picker; daytime inspection is available at sites where traffic flow is not adversely affected)

Photo 4 Instrumental inspection (ultrasonic testing of welds of steel structures)

Photo 5 Occurrence of fatigue cracking (observed in weld line where beam and column cross)

• Fatigue Cracking in the Corners of Steel Bridge Piers

In 1997, fatigue cracking was found in the corner of a steel bridge pier. A detailed investigation confirmed that this kind of cracking is the result of propagation of fatigue. Then, the cause of fatigue condition was assumed by implementing a fatigue test on a full-scale test specimen. As a result of further examinations, a method was adopted that joins splice plates with existing bridge piers using high-strength bolts in order to ease the stress concentration in areas where cracks occur.

Fig. 2 and Photo 5 show the position where fatigue cracking occurs, and Photo 6 shows the reinforcement method using steel plate.

• Fatigue Cracking in Steel Decks

Fatigue failure is often found in steel decks. In particular, there are confirmed cases in which fatigue cracks that occur in the welds of steel deck plates and trough ribs propagate in the direction of the deck plate thickness and then penetrate into the deck plate.

As a countermeasure against such fatigue

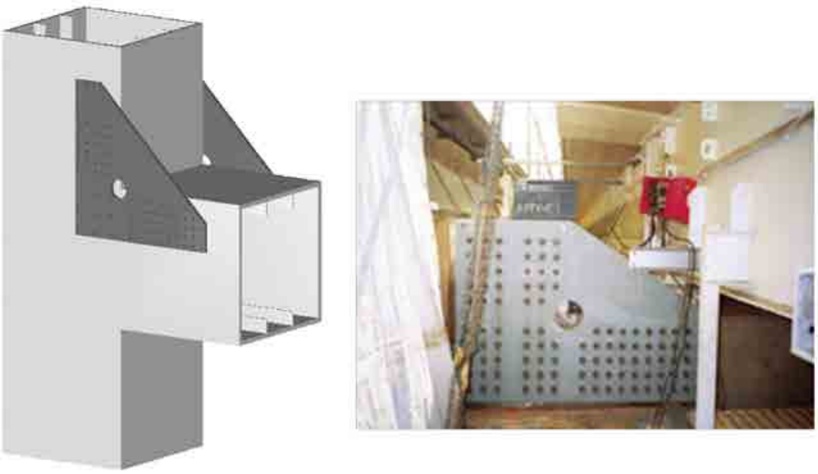


Photo 6 Reinforcement of steel bridge pier by splicing steel plate to pier

cracks, a method has been adopted that uses steel fiber-reinforced concrete (SFRC), having a larger elastic modulus than asphalt pavement, to suppress by means of the rigidity thus obtained the strain that occurs in steel deck welds and trough ribs.

In order to confirm the reinforcing effect provided by the use of SFRC, it was necessary first to decide the material and structure to be applied. Then, the fatigue test was conducted using the adopted paving method that called for 8 cm-thick asphalt pavement to be replaced by 5 cm-thick layer of SFRC over which a 3 cm-thick layer of asphalt was laid.

Photo 7 shows the SFRC placement and the pavement structure used by this method.

• Fatigue Countermeasure for RC Slabs

In order to reinforce RC slabs, two methods have conventionally been adopted to ease the bending moment produced by the live load that occurs in RC slabs: 1) the additional installation of one or two stringers between the main girders and 2) the joining of steel plates to RC slabs in the expectation that, when joined, the steel plate and the existing RC slab will mutually resist the bending moment occurring in the RC slabs. However, because steel products are used for these methods, rigid scaffolding was required and it was necessary to pay special attention to the handling of the members to be attached.

Later, it became clear that the development of RC slab damage is caused by a fatigue phenomenon attributable to vehicle traffic. In response, a method of reinforcement using a new material was established. This method pastes carbon-fiber sheeting to the rear surface of RC slabs to mitigate RC slab fatigue.

Photo 8 shows an RC slab that is reinforced by a carbon-fiber sheet that was pasted to the rear of the slab and then painted, and the reinforcing work underway.

Photo 7 Provision of countermeasure for fatigue in steel deck by means of SFRC paving (blue section: epoxy resin)

Photo 8 Reinforcement of RC slab with carbon-fiber sheet (sheet is pasted in the direction rectangular to slab; coating is provided to prevent deterioration of epoxy resin)

Examination of Large-scale Renewal of Expressway Facilities

Metropolitan Expressway Co., Ltd. has systematically and regularly implemented inspections of its various expressway structures and conducted the necessary repair work demanded by the inspection results. However, the total length of the Metropolitan Expressway network is now 301 km, of which 30% (or 100 km) consist of structures with 40 or more years in service since startup. As a result, damages have occurred in numerous expressway structures that have supported vehicle traffic for many years, and further serious damages have been found in some of these structures.

Given this situation, Metropolitan Expressway Co., Ltd. has worked out a large-scale renewal plan for specified sections of the 301-km expressway. This plan calls for bridge reconstruction and the replacement of floor slabs in an 8 km-long section where serious damage has been found, as well as the renewal of structures where necessary; and the repair of entire structures in a 55 km-long section where large-scale repair is required. Meanwhile, trial calculations for the approximate cost of these projects have been made: ¥380 billion for the 8-km section and ¥250 billion for the 55-km section, for a total of ¥630 billion.

Under its basic guiding principle of pursuing safety and driving ease and of supplying high-quality services for its users, Metropolitan Expressway Co., Ltd. is committed to carrying out maintenance work in the future. (Refer to Photo 9) ■

About Metropolitan Expressway Co., Ltd.

Further information about expressway maintenance and other operations of the company is available at:

- Corporate information: http://www.shutoko.co.jp/english/
- Maintenance operations: http://www.shutoko.co.jp/english/technology/mm-maintenance/

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Photo courtesy: Metropolitan Expressway Co., Ltd.

Photo 9 The Tsurumi Tsubasa Bridge (foreground) and the Yokohama Bay Bridge (background) at the Bay Shore Route of Metropolitan Expressway. Safe traffic is assured capitalizing on advanced expressway maintenance technologies.

Replacement of RC Slabs with Steel Decks on Mikawaohashi Bridge

by Tomoo Mito
Eight-Japan Engineering Consultants Inc.

The Mikawaohashi Bridge is a non-composite steel girder bridge having a length of 398 m and a continuous 2+3+2 span composition. It is installed where the Kanazawa-Mikawa-Komatsu Route, a major local highway, crosses the Tedori River in Ishikawa Prefecture. The bridge was in need of retrofitting due to the following concerns: increasing traffic volume, progressive deterioration of the RC slab caused by airborne salt, the need for another sidewalk to be installed on the bridge's upriver side (bridge widening) and measures to treat the B-type live load required by the *Japanese Highway Bridge Design Specifications*.

To improve the above situation which the bridge was suffered, the previously adopted, conventional RC slab for the bridge was replaced with a steel deck (Fig. 1). The replacement project that was conducted on the Mikawaohashi Bridge is outlined below.

Fig. 1 Section of Mikawaohashi Bridge (before and after replacement of slabs)

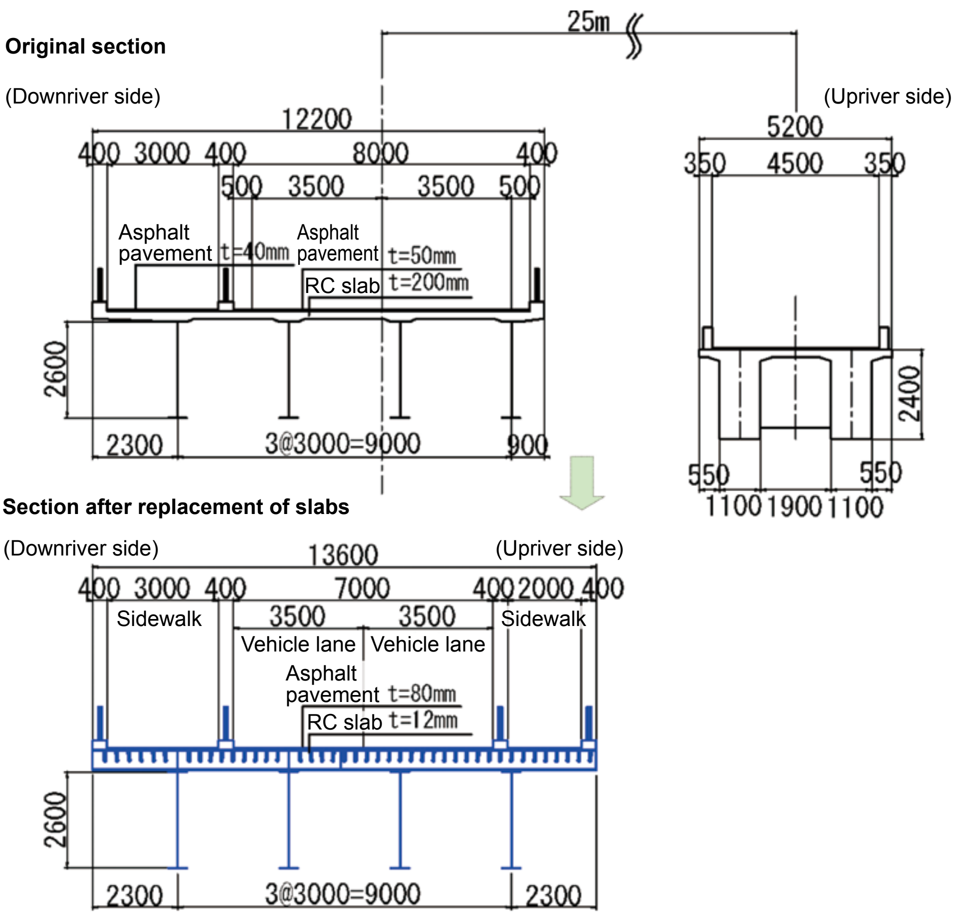
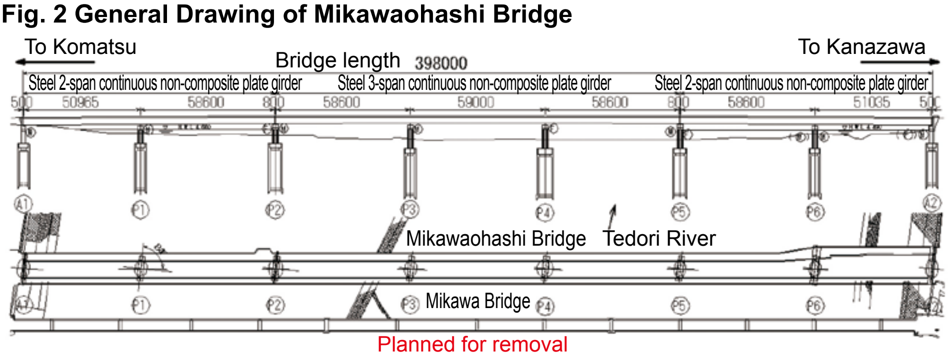


Fig. 2 General Drawing of Mikawaohashi Bridge



Bridge Outline and Need for Replacement

The Mikawaohashi Bridge is installed 200 m from the mouth of the Tedori River in Mikawa-Minami, Hakusan City, Ishikawa Prefecture. The bridge has a length of 398 m (span divisions: 51.0+58.6, 58.6+59.0+58.6, 58.6+51.0 m). The superstructure is composed of two 2-span continuous non-composite steel plate girders and one 3-span continuous non-composite steel plate girder; and the substructure is an RC wall-type pier with an open caisson foundation (Fig. 2).

The Mikawa Bridge formerly located upriver from the Mikawaohashi Bridge was constructed in 1938. It was used for vehicle traffic until 1972 when the Mikawaohashi Bridge was put into service, and was afterwards used for bicycle and pedestrian traffic (Photo 1).

However, removal of the Mikawa Bridge became necessary because of an inadequate condition in terms of the river section. This then generated concerns about the form of the new replacement sidewalk that would be installed on the upriver side of the Mikawaohashi Bridge. In addition, B-type live load reinforcement was implemented for the main girders of the Mikawaohashi Bridge from 1997 to 1998. But as deterioration of the concrete slabs (corrosion of the steel reinforcing bars, peeling-off of the surface concrete) progressed, diverse countermeasures were required, including: widening of the bridge to accommodate the new sidewalk, measures to treat

Photo 1 Mikawaohashi Bridge and Mikawa Bridge (taken from left coast side)

the B-type live load on the slabs, and reduction of bridge weight that was also effective for seismic retrofit.

Tasks and Approaches Aimed at Increasing the Service Life of the Mikawaohashi Bridge

Three major issues faced the Mikawaohashi Bridge:

- **Progressive Deterioration of the RC Slab**

A previous investigation of the RC slab made in 1996 showed that many cracks had developed and that the surface concrete was sloughing, and subsequent periodic bridge inspections showed that the deterioration was progressing. In a 2006 investigation of the chloride ion content, it was confirmed that the presence of chloride ions in the steel reinforcement would soon reach a critical level where corrosion would occur (Photo 2). Accordingly, it was assumed that deterioration of the RC slab would progress quickly due to corrosion of the steel reinforcement caused by damage from salt water.

- **Necessity of Installing Another Sidewalk**

It was determined that the Mikawa Bridge (pedestrian bridge) located just upriver from the Mikawaohashi Bridge had to be removed without delay because of an inadequate condition in terms of the river section as mentioned above. In response, the citizenry of nearby Hakusan City demanded that after removal of the Mikawa Bridge a replacement sidewalk be installed on the upriver side of the Mikawaohashi Bridge so as to secure safety and convenience.

- **Lighter Weight for Improvement of Bridge Seismic Performance**

It was made clear through separate examinations that to widen the bridge while utilizing the RC slab as it was would increase the weight of the superstructure, thereby impairing the bridge's seismic retrofit. Therefore, reducing the weight of the superstructure was an indispensable requirement for improving the seismic performance of the entire bridge structure.

In order to find an approach that would accomplish these three tasks, extensive examinations had to be made with regard to widening the bridge and, further, to replacing the RC slab and the effect that would have on structural performance, work efficiency and the existing bridge piers. As a result of these studies, steel decking was adopted as the method to replace the RC slab because of the following reasons: widening of the bridge and reduction of the dead weight could be made compatible, without any adverse effect on the work to reinforce the B-type live load already underway, and the weight of the superstructure could be reduced sufficiently to achieve improved seismic performance within the required range of design.

Photo 2 Exposure of steel reinforcement of RC slab (photo taken in 2006)

Replacing Slab with Minimum Effect on Traffic

In replacing the RC slab with a steel deck, the following two aspects were taken into consideration when determining the optimum bridge width.

- **Determination of Final Width—Based on the Stress of the Main Girders**

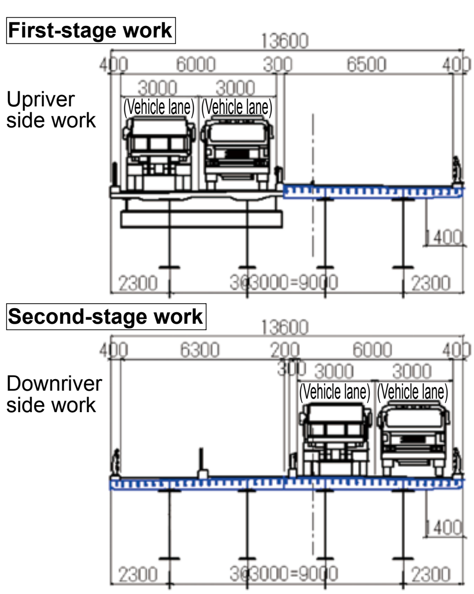
Trial calculations to determine the total width of the bridge upon completion considered four cases with respect to the stress of the main girders: B=12.6 m, 13.6 m, 14.6 m and 15.6 m. As a result, it became clear that the total width must be 13.6 m or less in order to remain within an allowable stress of the main girders that were currently undergoing reinforcement for the B-type live load.

- **Determination of Final Width—Based on Replacement Work**

Because traffic in both morning time and evening time is heavy and, further, because a crossing is located at the end of the bridge, it was feared that traffic congestion would occur while the RC slab was being replaced by a steel deck. Therefore, an important premise was that two lanes (one in each direction) had to remain open during the replacement work. The narrowest width that would allow two lanes to remain open during the work was 0.4+6.0×2+0.3×2+0.2+0.4=13.6 m.

As a result of the above examinations, a width of 13.6 m was adopted as the optimum width of the bridge. (See Fig. 3)

Fig. 3 Determination of Final Width—Based on Replacement Work (In the case of securing two lanes: Total breadth of 13.6 m or more is required)



Design Concepts of Slab Replacement

- **Stress Verification in Main Girders**

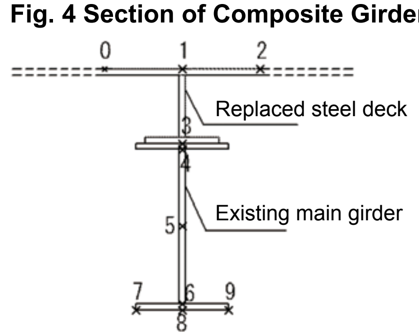
While the Mikawaohashi Bridge is a non-composite girder structure, in its replacement of RC slabs with steel decks, stress verification was implemented by taking into account the composite effect produced by combined use of steel deck and main girders and based on the following concepts:

1) The dead load before installation of the steel deck would be borne only by the sections of the existing main girders.
2) The dead load after installation of the steel deck and the live load would be borne by the composite sections including the steel deck. (Fig. 4)
3) Stress verification would be made at each step in the replacement work and by putting together the stress occurring in the above two.
4) Because of reduced thickness due to corrosion in the sections of the main girders, the section calculations would be made by decreasing the thickness of the webs and lower flanges based on the survey results. Meanwhile, the calculations would be made by taking into account the sections where the existing girders are reinforced using stiffening plates.

- **Calculation of Stress at Each Step of the Work Process**

Because the stress verification is made by tak-

Fig. 4 Section of Composite Girder

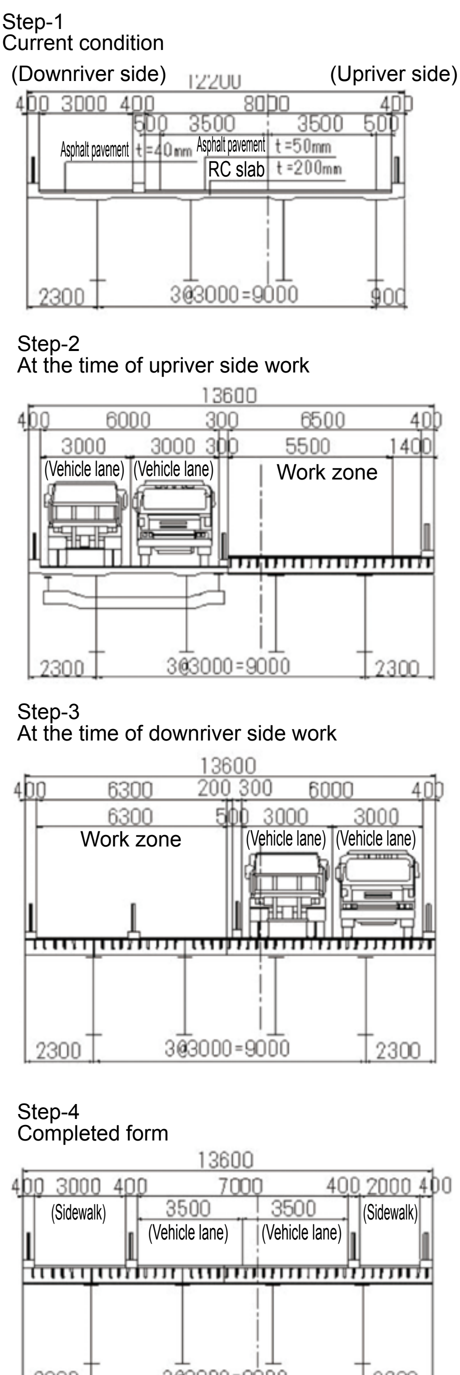


ing into account the composite effect of the steel deck and the existing main girders as mentioned above, and because the replacement work is done in progressive steps that take traffic management into account, the structural system differs at each step in the process.

To cope with such a situation, the steps in the replacement process were worked out as shown in Fig. 5. In order to prevent bridge collapse, the stress conditions were checked at each step to confirm that both the allowable values and the design loads lay within specified levels pertaining to safety during the replacement work, to the stress of the main girders upon completion of the work, and to the bearing reaction force and design load.

The steps of the work process are shown in Fig. 5.

Fig. 5 Replacement Work by Step



Reinforcement of Existing RC Slab during Replacement

At the midway point of replacing the RC slab with the steel deck while simultaneously keeping two lanes of traffic open, the middle slabs at the center of the existing RC slab overhang the support structure. Because the wheel loads work on the downriver side of the overhanging section of these slabs and because the slabs have a thin wall thickness (16 cm) and are designed for sidewalk use, they are reinforced using temporary stringers and brackets. (See Fig. 6 and Photo 3)

Successful Replacement of RC Slab with Steel Deck

The work of replacing the RC slab with a steel deck on the Mikawaohashi Bridge started in 2010 and was completed in 2014.

The service life of the bridge has been prolonged by removing the existing RC slab, which was suffering progressive deterioration and declining structural strength, and by replacing it with a steel deck. And, further, seismic performance has been improved by reducing the overall structural weight. On top of this, because a sidewalk was added to the upriver side of the Mikawaohashi Bridge, the Mikawa Bridge was removed without impeding conventional traffic (Photo 4).

Further, because two lanes of traffic were maintained without the installation of a temporary bridge during the replacement work, not only was the effect on traffic minimal, but both the construction term and the overall cost were reduced.

It will be glad if the current report serves as a useful reference when planning future bridge slab replacements, which are forecasted to increase. ■

Fig. 6 Reinforcement of Downriver Side RC Slab

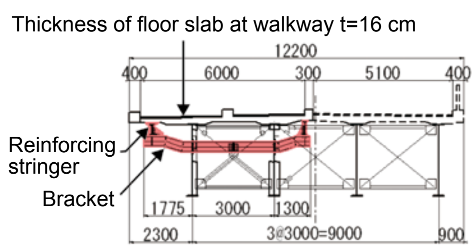


Photo 3 Bracket for use for RC slab reinforcement

Photos 4 and 5 Mikawaohashi Bridge after replacement with steel deck

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Second Steel Construction Conference in Cambodia Being Planned

The Japan Iron and Steel Federation (JISF) is planning a second conference to be held in Cambodia. It is tentatively titled “Recent Technologies for Steel-structure Construction 2014” and will take place in Phnom Penh, Cambodia in December 2014, under expected joint sponsorship by JISF, the Ministry of Public Works and Transport (MPWT) of Cambodia and the Institute of Technology of Cambodia (ITC). In early June of this year, a meeting to promote the conference was held between JISF and its two Cambodian counterparts.

The conference will consist of a number of sessions. In the session for engineers, there will be five lectures given by experts from both nations that will discuss steel-structure technologies in the fields of ports/harbors, bridges and buildings. In another session that will feature participation by key persons from both nations, extensive discussions will focus on the promotion of steel-structure construction in Cambodia.

In December 2012, JISF joined with MPWT and ITC to hold the first conference, titled “Conference on Advanced Technologies for Steel Construction 2012,” in Phnom Penh. The event was a success with the participation of about 200 engineers from the governmental, academic and private sectors. ■

Conference on Advanced Technologies for Steel Construction 2012 held in 2012 in Phnom Penh, Cambodia

Request for Participation in Survey of *Steel Construction Today & Tomorrow*

Steel Construction Today & Tomorrow, a joint periodical of the Japan Iron and Steel Federation (JISF) and the Japanese Society of Steel Construction, is published three times a year. It is the only English periodical that distributes technological information about steel construction in Japan to the worldwide construction community.

We are conducting a survey of the periodical’s readership regarding publication of the three issues planned for fiscal 2014. The survey’s major aim is to gain an accurate understanding of reader needs so as to enhance the usefulness of the publication. The survey forms are available as follows.

• At the JISF Website

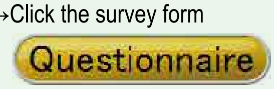
→Enter “jisf” in the search window of your internet browser

→Click on the tab for JISF’s English website

The Japan Iron and Steel Federation

→Click on the tab for Steel Construction Today & Tomorrow

→Click the survey form



• Printed Form for Faxing

A survey form is enclosed in the magazines sent to our regular subscribers. Please answer the questions in the form and fax to +81-3-3667-0245.

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