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## Fatigue Assessment of Steel Bridges of the Bullet Train System

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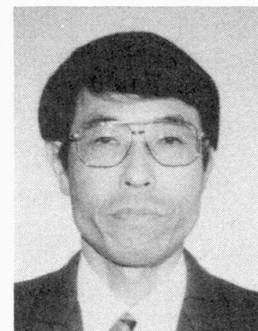
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## Summary

It is over 30 years since the first bullet train system was opened for public service in Japan. The structures of bullet train systems have been supporting the operation without any accident. But they have been showing signs of deterioration. This paper is intended to give an outline of fatigue problems in regard to assessment and life extension works in steel bridges of bullet train systems.

## 1. Introduction

The bullet train systems are the most important trunk lines in Japan. For example, the Tokaido Shinkansen, the first bullet train system completed in 1964, covers a distance of 515 km between Tokyo and Osaka. Nearly 250 trains run daily at the maximum operating speed of 270 km/h. The daily passenger traffic reaches as high as 300 thousands.

The structures of bullet train systems have been supporting the operation without any fatal accident. But they are showing some signs of deterioration. In particular, the fatigue damage in welding joints of steel bridges attract our close attention [1]. However, no fatal accidents have taken place yet because inspections of these bridges have been routinely carried out, and fatigue cracks discovered were properly repaired. Fatigue damage have developed only in the secondary members not incorporating any fatigue design.

Hereafter, there will be a greater demand for increasing the train speed and increasing the transportation capacity, therefore researches for upgrading the structural details efficiently and reliable maintenance are required necessarily.

## 2. Outline of Steel Bridges in Bullet Train Systems

### 2.1 Bridge Structures

Plate girder, truss girder and composite girder bridges have been usually used in the bullet train systems. With regard to the track structure, open floor track with a small dead load is in use for many of steel bridges in the early bullet train system in consideration of economy, because the noise in that type of bridge did not cause any social problem in the construction period. The open floor track is not applied now to the steel bridges in the area where the noise becomes a problem, and the ballasted track or concrete slab track is applied. And the noise insulating devices were attached to most of open floor type steel bridges in the bullet train system in the latter half of 1960's.



## 2.2 Standardization of Design of Structural Details

Standardization of the design has been introduced for the structures of bullet train systems as much as possible, because in the construction of structures it is required to attain the maximum economy. Therefore, similar structural details were adopted for the same type of bridge, and this is to be considered for maintenance. With regard to the welding joints, the welding was really applied to the steel bridges at the same time in construction of structures of the first bullet train system. Some of the details of welding joints have been improved, not being applied at present.

## 2.3 Fatigue Design History

The first codes for the fatigue design of railway welded steel bridges had been established in 1956, being strongly influenced by DIN of the day especially with regard to welding joints. The allowable fatigue stresses had been set up based on fatigue strengths at two million cycles which were considered as fatigue limits.

Fatigue design for the Tokaido Shinkansen system was done based on codes established in 1960, in which the results of fatigue tests which had been carried out in Japan were taken up and the allowable fatigue stresses related to 500 MPa class steel were added. And a consideration was made such as setting design fatigue train load at 18 tons against design train load of 16 tons with the aim of reflecting the difference in influence line length on number of design stress repetitions. On the other hand, for the subsequent bullet train system, the design fatigue train load was set at 19 tons, the increasing train load being considered.

The allowable fatigue stresses were revised in 1970, based on fatigue strengths in non-destructive probability of 95 %. Furthermore, in the design codes established in 1972 for Tohoku and Jyoetsu Shinkansen which are the bullet train systems opened in 1982, the design life of 70 years was set up and stress repetitions during the life were considered in the allowable fatigue stresses.

In 1983, the fatigue design codes were revised with reference to the large scale fatigue tests and other results.

The current fatigue design codes were revised in 1992 reflecting the latest results.

## 2.4 Changes in Circumstances of Service Conditions

The service conditions have been changing from the opening to the present state. Not only the volume of traffic but also the train speed is increasing. These have all changed in the direction of increased severity for structures. In regard with the train speed the maximum speed has been increasingly changing from 210km/h to 270km/h. However, the weight of vehicle is changing lightly in consideration of effect on the structures. The axle load of vehicle with the maximum speed of 220 km/h (car type A) is about 150 kN and that with the maximum speed of 270 km/h (car type B) is about 110 kN.

In these circumstances, some phenomena are observed such as an increasing out-of-plane vibration of web plate. Fig.1 shows the relationship between the train speed and the stress of flange plate in the deck plate girder, and between the speed and out-of-plane acceleration of web plate. Stresses considered in the design stage such as flange plate stress are closely related to vehicle weight rather than train speed, but the out-of-plane vibration is much increasing with train speed.

# 3. Observed Fatigue Damage and Retrofitting Works

## 3.1 Outline of Observed Fatigue Damage

The structures of bullet train systems suffered from very severe loading condition caused by high-speed train operation and highly repetitive frequencies.

Concerning steel bridges, several types of fatigue damage have been observed as shown in Fig.2[2][3]. These types of damage began to be observed in about 8 years after the opening of service. Some types of fatigue cracks are comparatively rare in the conventional railway system. They are in many cases caused by stress concentrations due to structural details of members, by out-of-plane displacements occurring between perpendicularly crossing members

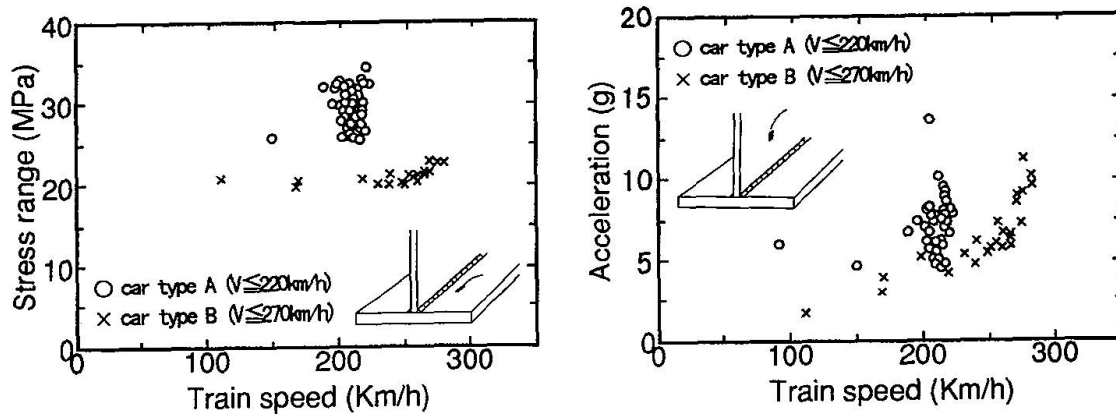


Fig.1 Relationship between train speed and stress range of flange or acceleration of web

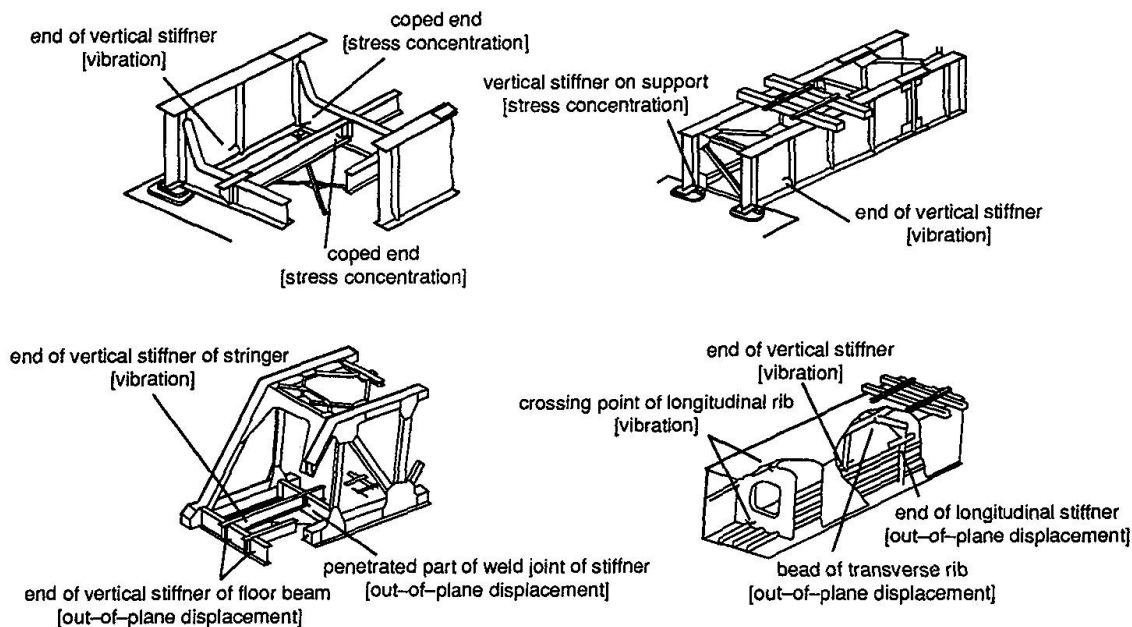


Fig.2 Fatigue damage in steel bridges

such as main girders and cross beams or cross beams and stringers, and by vibration due to distortion under high-speed operation of trains peculiar to bullet train system.

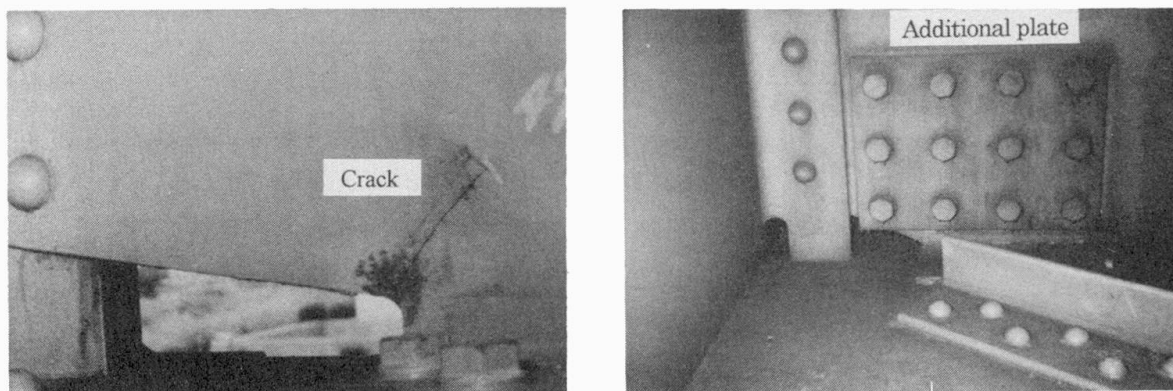
A fatigue crack has not yet been detected at important portion of main members which lead to catastrophic failure where fatigue was assessed in the design stage. Fatigue cracks are, however, often discovered at such as secondary members as side walks, connections of attached facilities and diaphragms or secondary local portions of the main members. These types of damage were already repaired and almost of details which had the possibility of occurrence of the same kind of damage were also retrofitted.

### 3.2 Coped Cross Beam of Through Plate Girder

The damage was observed at the coped end of the web plate of cross beam, as shown in Fig.3. Many cases of damage were observed in end cross beams. The crack usually develops obliquely from the corner of coped end of web plate to the inside of web plate. This type of damage was discovered in about 8 years after the opening of service.

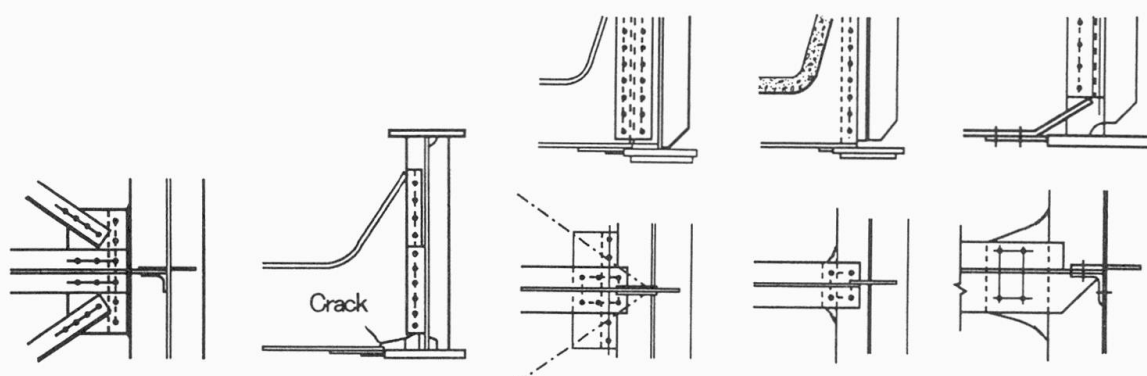
Measurements of actual bridges, structural analysis and fatigue tests were carried out in order to study causes and retrofiting methods. The results are as follows;

(1) The main cause of this fatigue crack was that the end of lower flange of the cross beam was



(a) Crack

(b) Retrofitting method

**Fig.3** Damage and retrofitting of coped cross beam

(a) Original detail

(b) Improved detail

**Fig.4** Improvement of details of cross beam

coped to connect with main girder and this induced stress concentration.

(2) The measured stress becomes higher in the case of normal shoe seat than in the case of damaged shoe seat. Thus, the settlement of supporting point was also one of the causes.

(3) In the retrofitting, additional plates were applied to web plate in order to increase the loading capacity. A sufficient reinforcing effect was obtained.

(4) The details of the currently designed bridge was improved as shown in **Fig.4**

### 3.3 Coped Stringer of Open floor Type Bridge

A crack occurred at the stringer web plate of through plate girder or through truss, as shown in **Fig.5**. This type of crack was discovered in about 10 years after the opening of service.

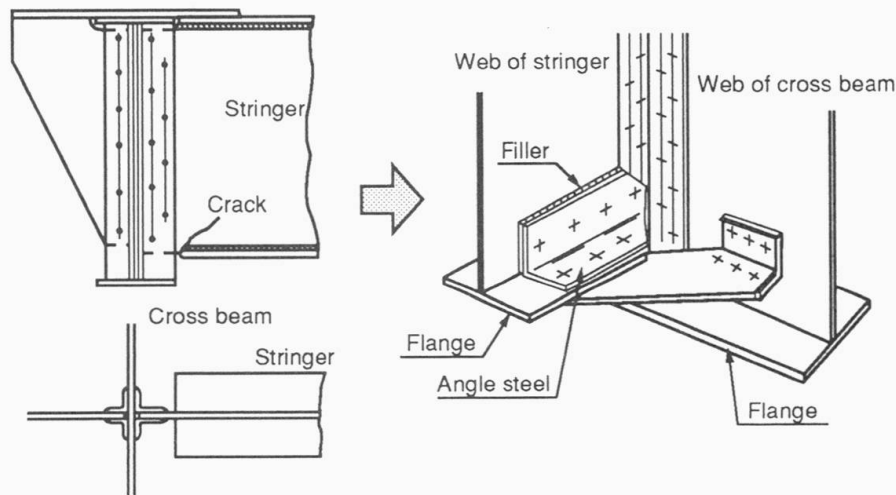
The stringer is a member which is directly subjected to the train load, and to the great lateral force and impact by high speed train. It is necessary to avoid a local stress concentration by improving the lateral rigidity or keeping the stress flow as continuous as possible. However, in some bridges the lower flanges of stringer were not connected to the cross beam. This caused the stress concentration at the cope combined with it and led to cracking.

With regard to the repair, the lower flange was connected with the cross beam web plate by extending it to the stringer end.

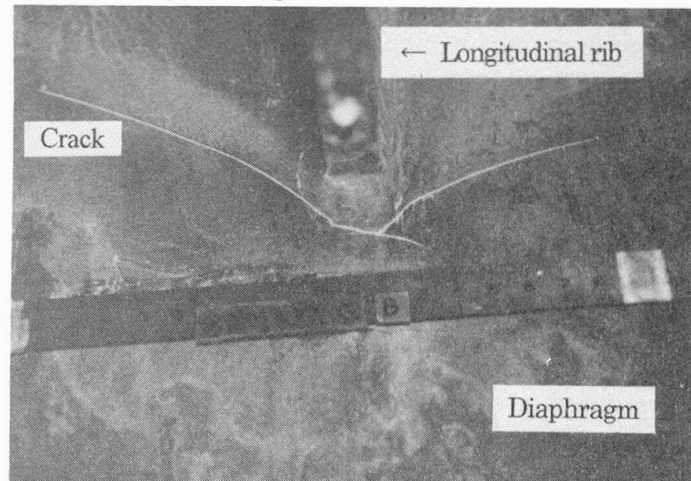
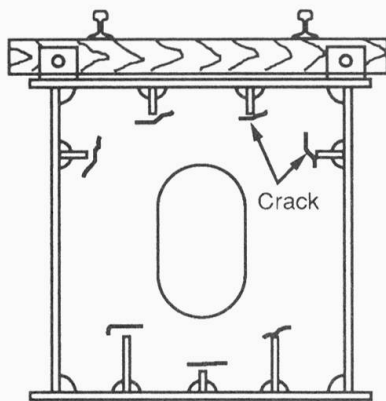
### 3.4 Intermediate Diaphragm in Box Section Deck Plate Girder

In the box section deck plate girder, intermediate diaphragms are provided at intervals of 5 to 6 m in order to improve the torsional rigidity. Diaphragms are connected by welding with flange plates or longitudinal ribs. Fatigue cracks were observed at this detail, on the surface of diaphragm at the toe of fillet weld, as shown in **Fig.6**.





**Fig.5** Damage of coped stringer



**Fig.6** Damage of intermediate diaphragm

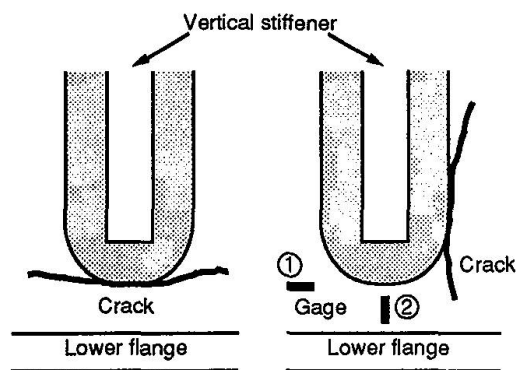
This type of crack is caused mainly by the stress concentration at the toe of weld and the high structural constraint. Diaphragms with such a structural detail are affected by an out-of-plane vibration with the passage of a high speed train and subjected to considerable fatigue at the restrained weld.

This damage was repaired by drilling a stop hole at the crack tip, or rewelding after gouging and the tungsten inert gas arc remelting (TIG-melting) applied at the toe of fillet welds.

### 3.5 End of Vertical Stiffener in Web Plate

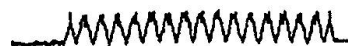
Many fatigue cracks were observed at the lower ends of the vertical stiffeners attached to the web plates of stringers of truss girders or box-section deck girders. This type of fatigue damage was discovered in about 10 years after the opening of service, being hardly observed in the bridges in the conventional railway systems. The crack originates primarily at the toe or the root of fillet weld around the lower end of the vertical stiffener and develops horizontally into the base metal of the web plate. Such a crack may sometimes propagate along the weld toe, and progress in the direction horizontal to the base metal with some length. On the other hand, in the fatigue tests of beam specimen under in-plane bending, cracks usually progress in the perpendicular direction along the toe of weld as cruciform joints. These are shown in Fig.7.

Fig.8 shows a comparison between the in-plane stress histories along the bridge axle of lower end of stiffener and out-of-plane bending stress histories of that. This shows that the former waveform is almost the same as anticipated in the design, while the component of vibration is contained in the latter waveform. Thus, It seems that this type of crack in the actual bridge is caused mainly by the increasing of out-of-plane vibration on web plate due to deflection of



(a) Actual bridge (b) Fatigue test  
Fig.7 Progress of crack

① Plane stress waveform along bridge axle



② Out-of-plan bending stress waveform perpendicular to bridge axle



Note : For the position of strain gage at the time of stress measurement, see Fig.7.①②

Fig.8 Stress histories of end of vertical stiffener

sleepers and those due to distortion induced by a high speed operation of trains, in addition to the plane stress in the design [4].

This damage is repaired by applying additional plates to the web plate in order to prevent the deformation after gouging and rewelding, and furthermore, applying the TIG-melting at the toe of fillet welds. This type of fatigue damage is, however, increasing and there are many more points likely to suffer such damage.

### 3.6 End corner of girder reducing section

Regarding the through plate girder and the deck plate girder, there is a type of reducing the height of girder near the bearing. As shown in Fig.9, a crack has been observed along the weld at the corner since several years ago[5]. It was revealed that the stress components perpendicular to the weld bead is superior. The small radius of the corner, groove weld with full penetration not being used, and out-of-plane vibration due to the high speed train operation are conducive.

This damage was repaired by applying additional plates to the web plate in order to decrease the stress after gouging and rewelding the fatigue crack.

### 3.7 Sole Plate of Box Section Deck Plate Girder

A sole plate is usually attached to the lower flange plate with high strength bolts in railway bridges, but in the box section plate girders of long span, fillet welding is used together with bolt joint.

A crack has been observed since several years ago, as shown Fig.10. This crack is observed on both the transverse welds and the longitudinal welds. The crack initiates from the root of weld, propagates along the bead of weld, and enters the lower flange plate [6]. Stress measurements of actual bridges, structural analysis and fatigue tests were carried out in order to study causes and retrofitting methods. The results are as follows;

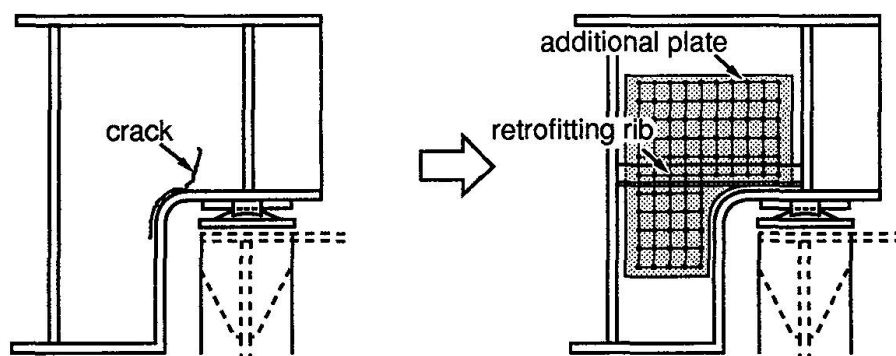


Fig.9 Damage of end corner of girder reducing section

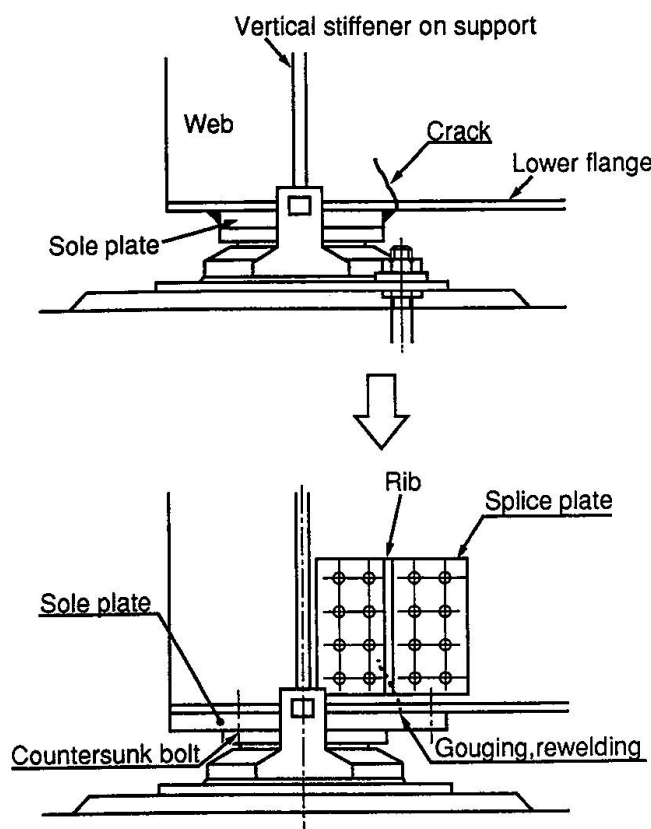


Fig.10 Damage of sole plate

(1) A poor condition of movable bearing increases the stress around the sole plate.

(2) A gap between the sole plate and the flange plate increases the stress.

(3) In the retrofitting, a function of movable bearing must be improved, and sufficient reinforcing effect is obtained by exchanging the sole plate for a new one with bigger size after gouging and rewelding the crack.

## 4. Damage Prevention Works

### 4.1 Outline of Prevention Works

Inspections of railway structures consist of periodic regular inspection which is arrived out every two years and individual special investigation which is done when any damage is detected during a periodic inspection. The inspection is made by qualified engineers of railway company which manages and operates the line. These engineers have thorough knowledge about fatigue and corrosion which occur in bridge structures, and possess the capability of carrying out stress measurements,

nondestructive test, etc. However, with regard to new type of damage, causes and retrofitting methods are usually studied by the advisory committee consisting of specialists including university professors. Damage prevention works are considered based on these inspections and advice. Fig.11 shows an outline of prevention works. The aims of prevention works are mainly as follows;

- (1) To find the cheking points of details.
- (2) To presume the possibility of fatal fatigue damage and new type of damage.
- (3) To recommend a method of repairing such damage.

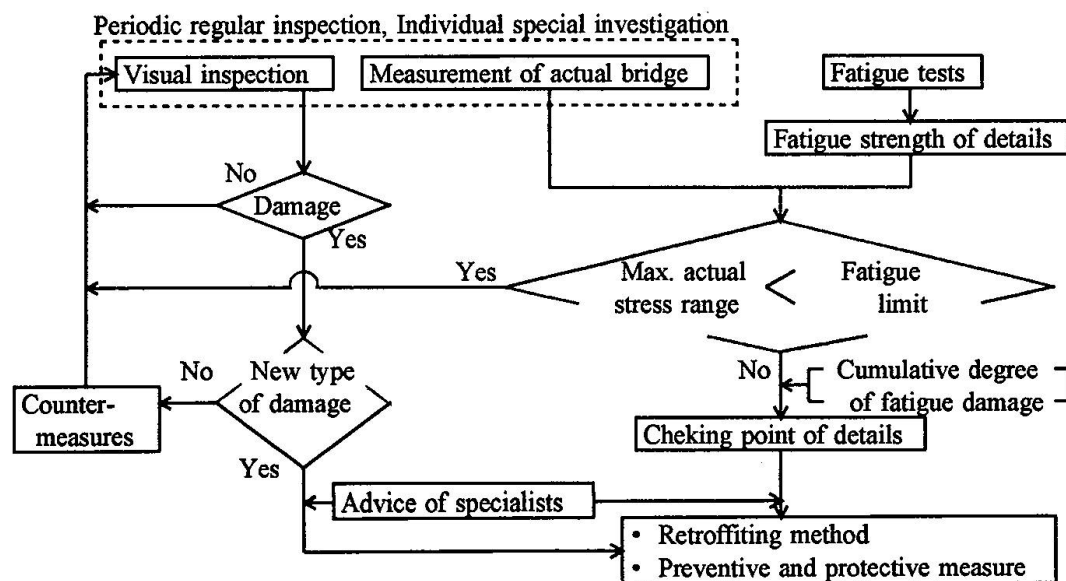


Fig.11 Outline of prevention works





Because most of structural details in steel bridges were standardized, it is likely that similar damage occur at certain points when a certain type of fatigue damage occurs. A more rational maintenance program is an important subject to cope with this serious problem.

When what had been assumed at the time of designing and the subsequent condition of use are considered, certain parts in bridge structures are approaching the ends of their design service lives in calculation. Therefore a number of preventive and protective measures against fatigue damage have been considered. One of these measures is the TIG-melting applied at the toe of fillet welds of lower ends of vertical stiffeners and at the toe of welds on the surfaces of diaphragms. This measure was done about 8 years ago, being applied to all of the box section deck plate girders with ballastless track.

#### 4.2 Damage Checking Points in Prevention Works

One of main prevention works is to determine the fatigue strengths of structural details used in the actual steel bridges. However, fatigue strengths should be determined under the higher repetitions of thousands of loading on railway bridges. Thus, many higher repetitive fatigue tests have been carried out. I-shaped test pieces with various joints such as Fig.12 were used in these fatigue tests. Results of these fatigue tests are reflected in the current design codes.

What have greatly changed in allowable fatigue strengths between the former design codes and the current design codes are longitudinal welds between flanges and webs, ends of gusset plates welded to webs, ends of gusset plates welded to flanges, ends of vertical stiffeners, and fillet welds between sleeper pads and flanges, shown in Fig.13. They are the major checking points in the prevention works.

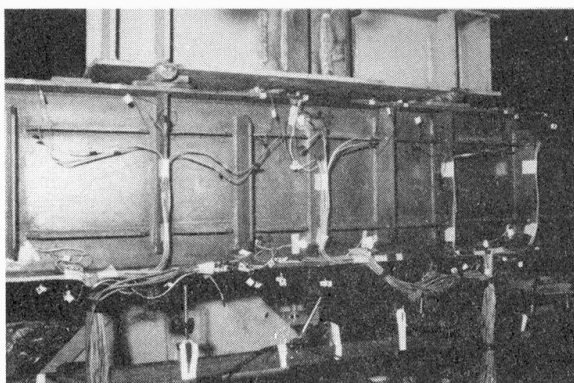


Fig.12 Fatigue test specimen

#### 4.3 Measurement of Actual Bridges

Another important preventive work is to grasp the degree of fatigue damage based on stress measurements in actual bridges. It is thought to consider maintenance based on stresses actually occurring in bridge members, since actual stresses are normally low compared with those in design calculations, measured stresses of the same type bridges show wide differences, and stresses not to be considered in the design such as out-of-plane stress of web plate are measured in some details. The reason for actual stresses being low compared with calculated stresses in case of railway bridges, lies in the load distribution due to rails and secondary members.

Stress measurements consist of grasping the maximum nominal stress range occurring in the detail (Fig.14) for comparison with the fatigue limit and the cumulative degree of fatigue damage.

The rain flow counting method is used to obtain stress range histograms from stress records, and the equivalent stress range and the cumulative degree of fatigue damage are calculated by applying the Miners law.

Many of bridges should be measured with priority, and in the measurements the following points should be considered in deciding the priority of bridges. This is also similar to bridges in the conventional railway system.

- (1) Open floor type bridges are apt to be subjected to effect of fatigue, on account of large stress range.
- (2) Short span bridges are apt to be subjected to effect of fatigue, on account of repetitive loading.

#### 4.4 Estimation of Steel Bridges

Fig.15 shows the results of stress measurements in actual bridges [7]. Measured stress range is very small in comparison with calculated one, as described in 4.3. This shows clearly that high possibility of fatigue damage in a fairly large number of members is found on the basis of design calculated stress, but there is hardly any cause as far as actual stress is concerned [8].

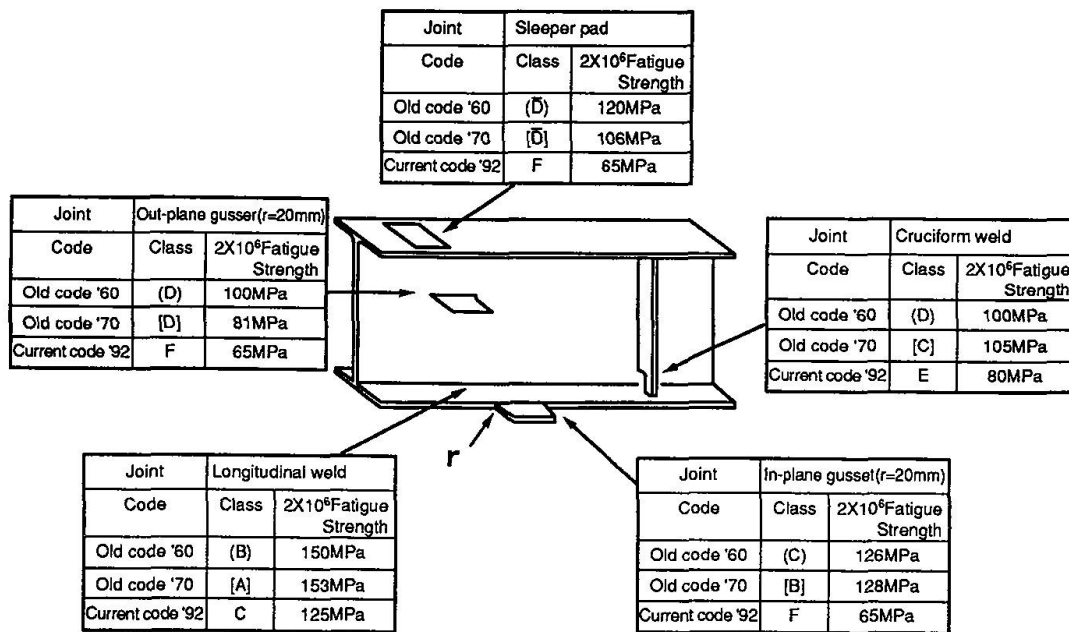


Fig.13 Changes of allowable fatigue stresses in design codes

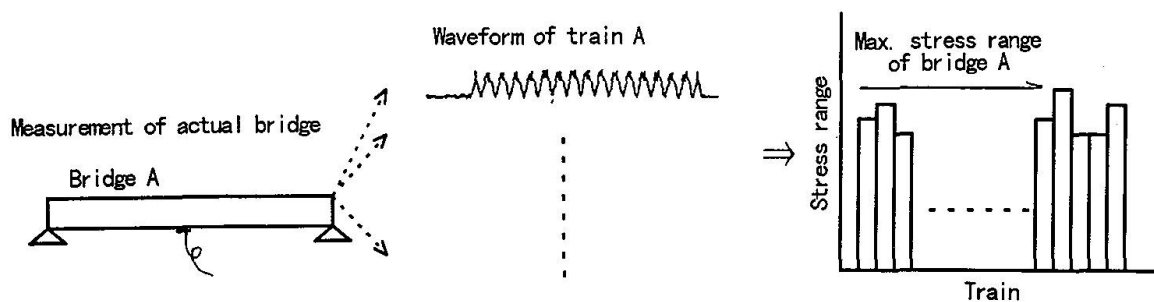


Fig.14 Maximum stress range of a bridge

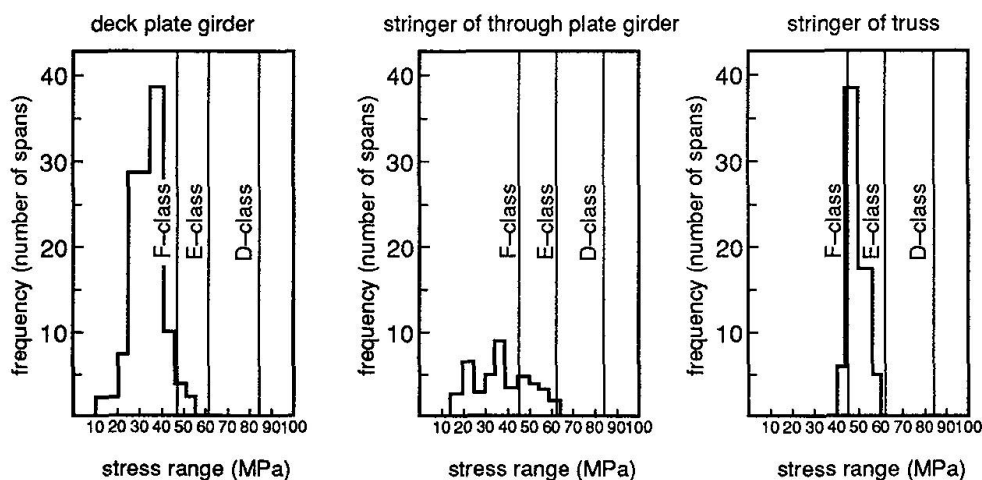
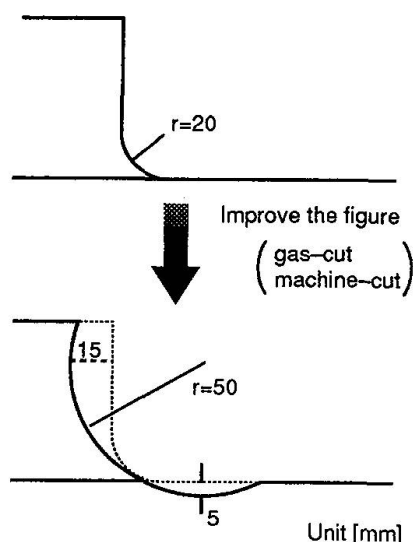


Fig.15 Variation of measured maximum stress range and fatigue assessments

Regarding the welding joints categorized into C class and D class such as longitudinal welds joining flanges and webs, it should be considered that there is ample allowance to be taken, and therefore it is justified to think that fatigue damage will hardly occur after this. However, the degrees of allowance of welding joints categorized into F class such as ends of



**Fig.16** Improving work of flange gusset detail

gussets attached to flanges or fillet welds of sleeper pads attached to flanges are slightly small. This shows the possibility of occurrence of fatigue damage. Hence, these details should be placed under watchful inspections, and remedial measures regarding fatigue strength should be worked out. For example, remedial measures of ends of gussets attached to flanges have been studied. One of conceivable methods is to increase the radius of the fillet at the end of gusset (Fig.15) [9]. Fatigue tests have been carried out and it has been made clear that fatigue strength will be upgraded from F class to C class. Regarding the sleeper pads, this detail and remedial measures are still under study.

## 5. Concluding Remarks

The service environment of a bridge, such as loading, often differs widely from what had been assumed at the time of designing.

To evaluate the degree of soundness of bridges, preventive works against fatigue damage should be carried out mainly by clarification of fatigue strength of used details and by measurements of actual bridges.

Considering some cases of measurements, the fatigue damage will hardly occur in the details categorized into C class and D class such as longitudinal welds joining flanges and webs hereafter. However the details categorized into F class such as ends of gussets attached to flanges or fillet welds of sleeper pads have the possibility of suffering fatigue damage.

Such preventive programs, including appropriate retrofitting works, have been taken from the early stage of fatigue damage in steel bridges. As a consequence, we are maintaining the tracks in good condition without any serious accidents. However, there is a growing demand for increasing the train speed or transportation capacity. This will probably add more and more to the burden on the structures, therefore researches for upgrading the structural details efficiently and reliable maintenance are becoming more and more important.

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