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Vibration Fatigue of Steel Bridges of the Bullet Train System

Fatigue due aux vibrations dans des ponts en acier du réseau de trains à grande vitesse

Schwingungsbedingte Ermüdung von Stahlbrücken des Japanischen Schnellzugnetzes

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SUMMARY

Fatigue cracks caused by out-of-plane vibration were observed in steel bridges of the Tokaido Shinkansen. Crack growth behaviour, measured cyclic stresses, evaluation of fatigue damage, and retrofitting methods are described.

RÉSUMÉ

Des fissures de fatigue causées par des phénomènes de vibration dans les éléments ont été observées sur des ponts en acier du Tokaido Shinkansen. Cet article présente le comportement de la propagation de ces fissures, les mesures des cycles de contraintes, l'évaluation du dommage en fatigue, ainsi que les méthodes de réparation possibles.

ZUSAMMENFASSUNG

In Stahlbrücken der Tokaido Shinkansen-Expresslinien wurden Ermüdungsrisse festgestellt, die vermutlich durch Schwingungen einzelner Elemente aus ihrer Ebene verursacht wurden. In diesem Bericht werden das Ausbreitungsverhalten solcher Risse, Messungen zyklischer Belastungen, die Ermittlung des Ausmasses von Ermüdungsschäden sowie mögliche Instandstellungsverfahren erläutert.

1. INTRODUCTION

One quarter century has passed since the Tokaido Shinkansen was opened for service. Tokaido Shinkansen is one of the bullet train system in Japan operating at the maximum speed of 220km per hour 230 trains daily (Fig.1). Various types of fatigue damages have happened in steel bridges of Tokaido Shinkansen, some of them being comparatively rare with the those in bridges of conventional railway system. Fatigue crackings due to the out-of-plane vibration-induced stresses are typical ones which appeared in the plate girder bridges, box girder bridges and stringers of truss bridges. Vibration-induced stresses under the passage of bullet trains at these bridge details are not of very high amplitude, but of high frequency. No fatal accident due to this type of crack has yet been reported, but retrofitting works are essential to meet the recent demand for an increasing train speed of 270km per hour and a greater transportation capacity. This report describes a behaviors of fatigue cracking, stress measurements, retrofitting works applied and results of remaining life evaluation.

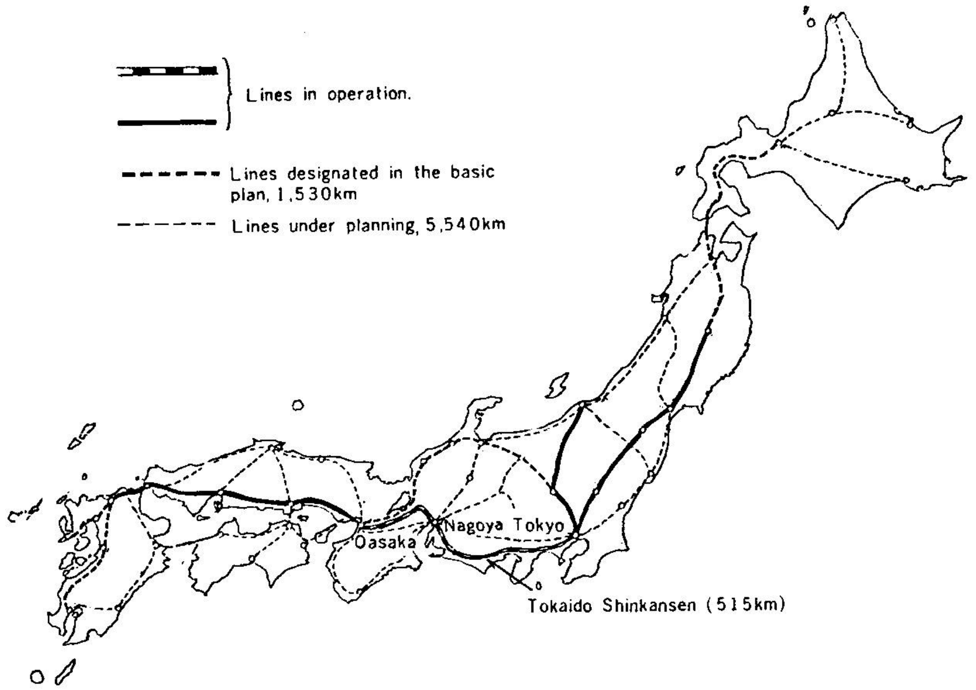


Fig.1 Network of bullet train system

2. OUTLINE OF THE BRIDGE STRUCTURE OF THE TOKAIDO SHINKANSEN

Table 1 shows the quantities of bridges by type of the Tokaido Shinkansen. Steel bridges were designed in accordance with the design specifications established in 1961 and welded structures were adopted for all shop joints. Fig.2 shows the design live load and anticipated use conditions of the Tokaido Shinkansen.

Type	Structure	Number of spans
Steel	Deck plate Girder (I section)	194
	Box Girder	139
	Through plate Girder	155
	Composite Beam	258
	Through Truss	135
Concrete	Reinforce Concrete	3307
	Presstrest Concrete	389
Total		4577

Table 1 Quantities by Bridge Type

Anticipated use conditions

Maximum speed	200 km/h
Number of trains per day	80
Number of cars per train	8 or 12
Design fatigue life	70 years

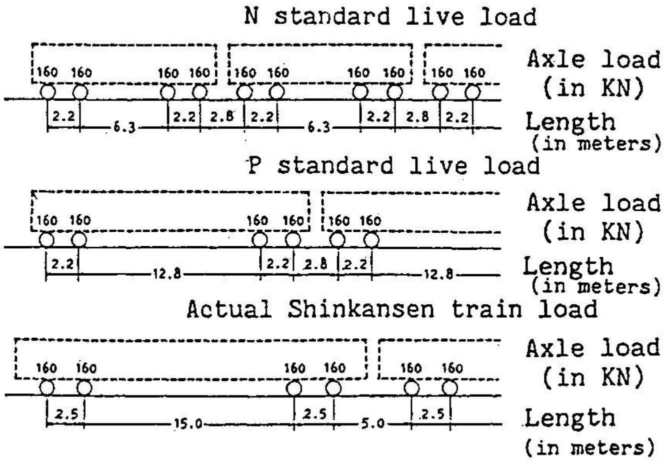
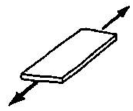
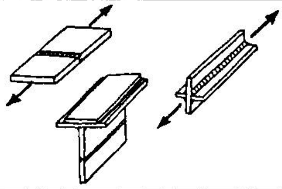
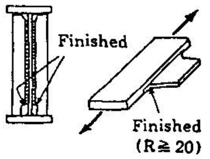
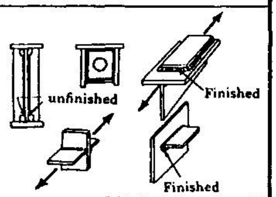
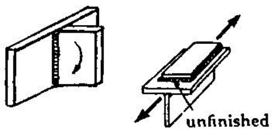


Fig.2 Design live load and anticipated use conditions

Table 2 shows allowable fatigue stresses for various types of joints in the 1961 specifications. These allowable fatigue stresses were based on the fatigue strength at 2 x 10^6 cycles of each structural joints. The designer used the load of 180 kN axle weight which are 20 kN larger than the axial load of the design load as shown in fig.2, taking into consideration a possible increase in traffic in the future. By the end of December 1989, nearly one million trains had passed along the line. The steel bridges in the Tokaido Shinkansen have suffered very severe loading condition caused by high-speed train operation and highly repetitive frequencies. Nevertheless, a fault in the structure has never led to an accident.

In the process of their service, however, some types of fatigue damage have been observed as shown in fig.3. No fatigue damage due to low fatigue strength has not observed in main members where fatigue was assessed in the design stage. However, fatigue cracks which are the result of secondary stresses and displacement induced stresses are often discovered in such various structural elements as girder webs, stringer webs, floor beam webs, side walks, connections of attached facilities and diaphragms. One of these fatigue crackings which occurs relatively frequently and whose cause has not been made much clear is a fatigue crack which occurs on a web plate at the lower end of a siffener (hereinafter referred to as the "lower end of a stiffener"). This fatigue cracks is in parallel with the lower flange.

Category	Types of stress	Fatigue allowable stress (kg/cm²)	Types of joints
A	Tension	$\frac{2000}{1-2/3k}$	
$\bar{A}$	Compression	$\frac{2400}{1-k}$	
B	Tension	$\frac{1500}{1-2/3k}$	
$\bar{B}$	Compression	$\frac{1800}{1-k}$	
C	Tension	$\frac{1260}{1-3/4k}$	
$\bar{C}$	Compression	$\frac{1440}{1-k}$	
D	Tension	$\frac{1000}{1-2/3k}$	
$\bar{D}$	Compression	$\frac{1200}{1-k}$	
E	Tension	$\frac{700}{1-3/4k}$	
$\bar{E}$	Compression	$\frac{800}{1-k}$	

Note, $k = \frac{|\sigma|_{min}}{|\sigma|_{max}}$

Table 2 Fatigue allowable stress (1961)

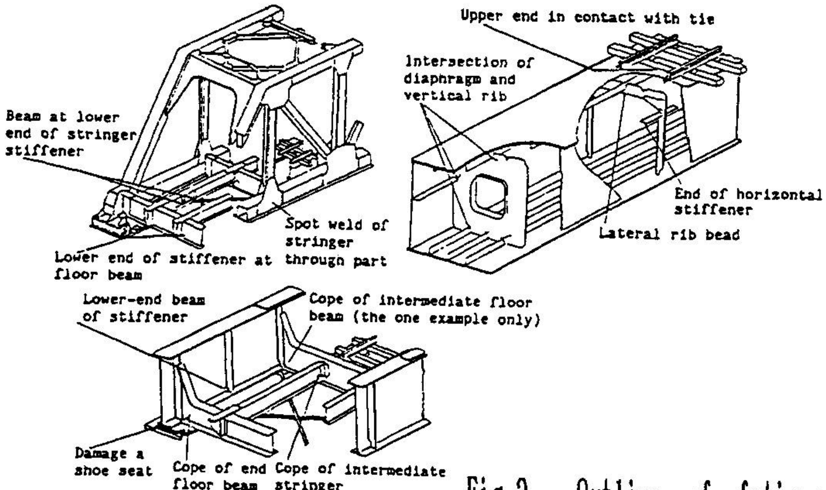


Fig.3 Outline of fatigue damage

3. VIBRATION INDUCED FATIGUE CRACKS IN THE LOWER END OF THE STIFFENER

Fig. 4 shows cracked girder web. This type of fatigue cracks occurred at the lower end of the stiffener on girder webs of box girder bridges and stringer webs of through truss bridges. Characteristically, no crack has occurred at the end of the stiffener to which floor beams or sway bracings are connected. These fatigue cracks began to be observed in 1978. To date, cracks have been observed in about 10,000 details of about 200 box girder bridges and in about 100 details of five through type truss bridges. Countermeasures were taken against all parts of through type truss bridges which were considered to undergo cracking.

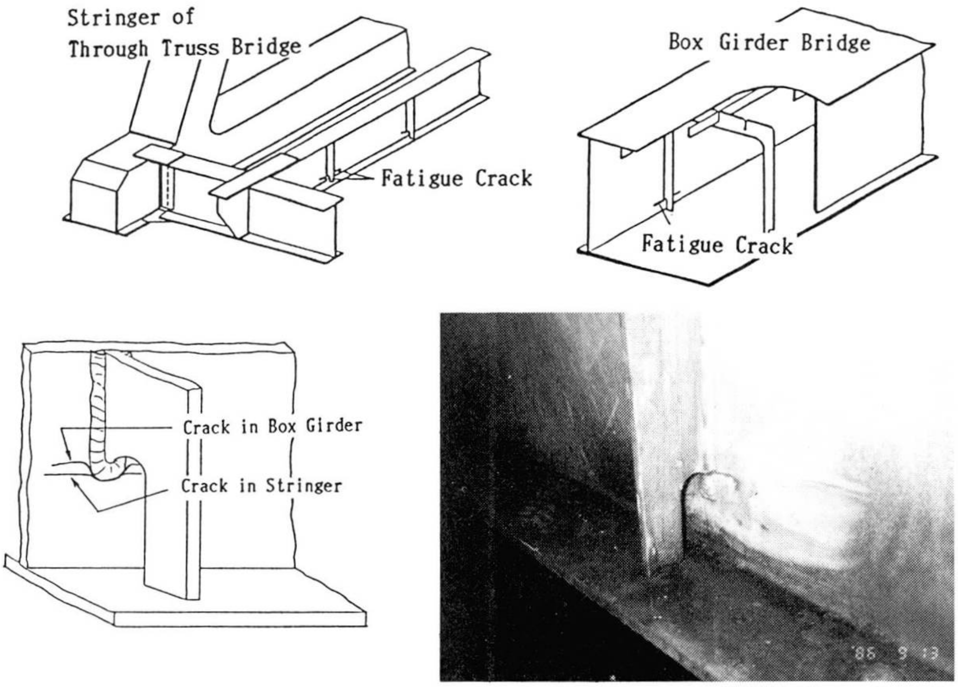


Fig.4 Fatigue crack at stiffener end

4. FAILURE MODE AND ANALYSIS

4.1 Measurement of the Cyclic Stresses

The stringer of a truss bridge was selected for cyclic stress study. This truss bridges has a standard floor system of the Tokaido Shinkansen whose span is 60 m, and the span of stringers is about 10 m. Measurement was made by comparison concerning whether there are sway bracings(Fig.5) and differences in train speeds. Dynamic strain measurement was conducted by placing strain gauges to the back and surface of the web plate at the lower end of the welded portion. The strain gauges were located 5 to 10 mm away from the lower end of weld toes (Fig. 6) stress records were processed as follows.

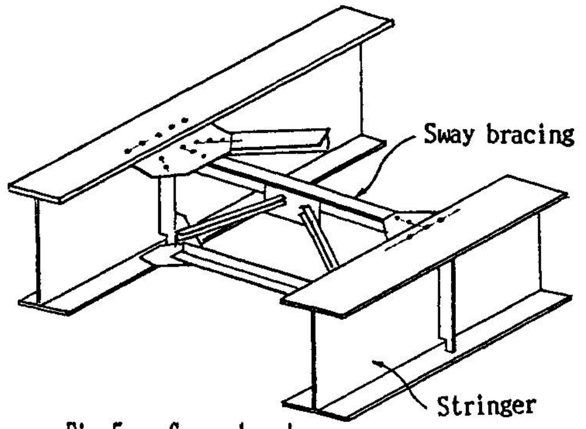


Fig.5 Sway bracing

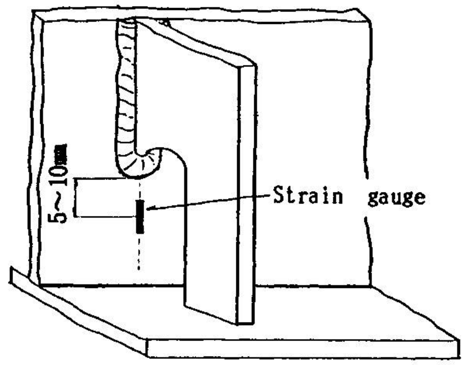


Fig.6 Position of strain gauge

(1) Stress-time analog records were transformed into digital data and the range pair method was applied to obtain stress range histgram.

(2) The linear damage law (modified Miner's law) was applied to evaluate the degree of fatigue damage. The JNR category B design curve was used in the analysis because measured stresses include some influence of stress concentration due to weld beads.

4.2 Results of Measurements and Analysis

Fig.7 shows a typical stress records on the stringer web and the results of frequency analysis. S-1 and S-2 are stress records at same point of stiffener end detail without sway bracing when a train passed away at a speed of 203 km/h and 178 km/h respectively. S-3 is a stress by train passing at a speed of 203 km/h at a position to which sway bracing is connected. Train speeds were obtained, using a strain gauge fitted to the track. Major results of this test can be summarized as follows;
As obviously shown by the comparison between S-1 and S-3, at the detail where sway bracing exists, stress amplitude is smaller than where sway bracing does not exists, and the high-frequency component is also smaller. This is considered to be due to horizontal vibrations of the lower flange.
Stresses in S-1 is the largest measured value in this structure and its stress waveform exhibits a resonating behavior. However, S-2, the stress waveform of a low-speed train (v=178 km/h) does not produce as large stress as S-1 (v=203 km/h). Therefore, horizontal vibration of the lower flange was induced only by the passage of high speed trains. In order to examine this resonating behavior, a stress was separated into the in-plane component and the out-of-plane component as shown in Fig.8. Its results indicate that the stress at the lower end of the stiffener without sway bracing detail is mainly caused by out-of-plane distorsion of the web plate. From this, it seems possible to say that the source of a predominating stress which occurs at the lower end of a stiffener is horizontal vibrations of the lower flange.
Fig.9 shows the stress histgram and the degree of fatigue damage of each stress component. The results suggest that fatigue cracks occurred in several years under S-1 stress history.
However, S-2 shows not so high degree of damage.

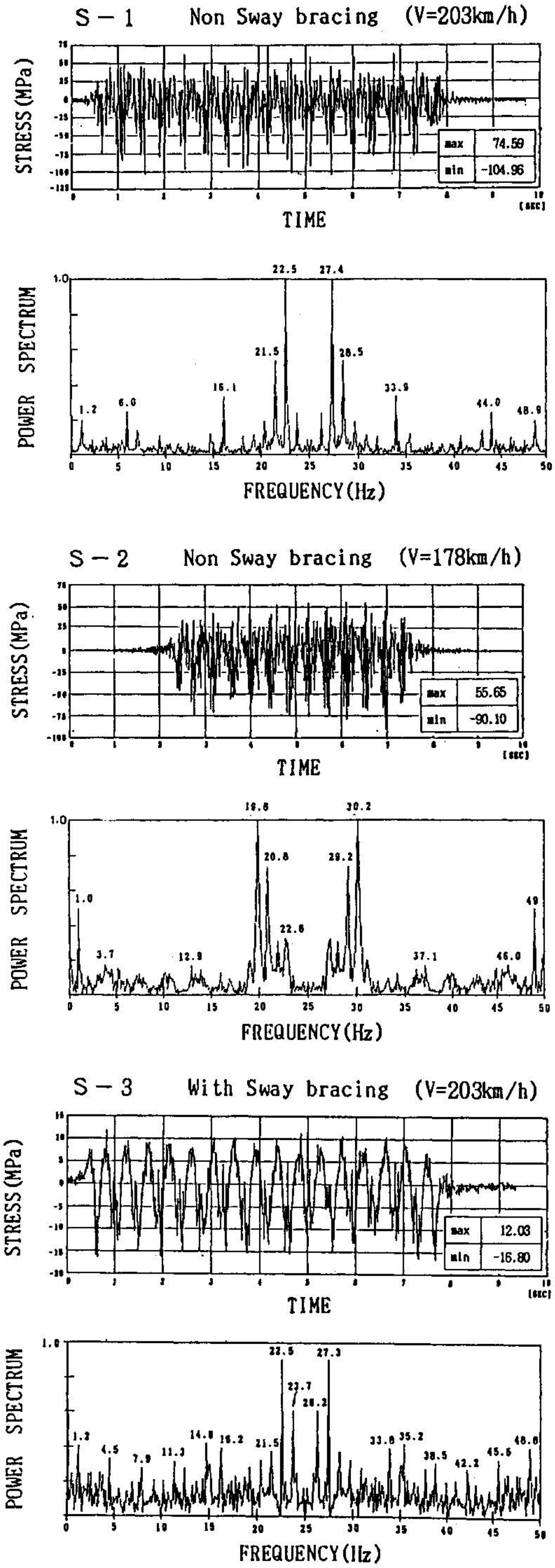


Fig. 7 Representative stress wave in Bridges mesured

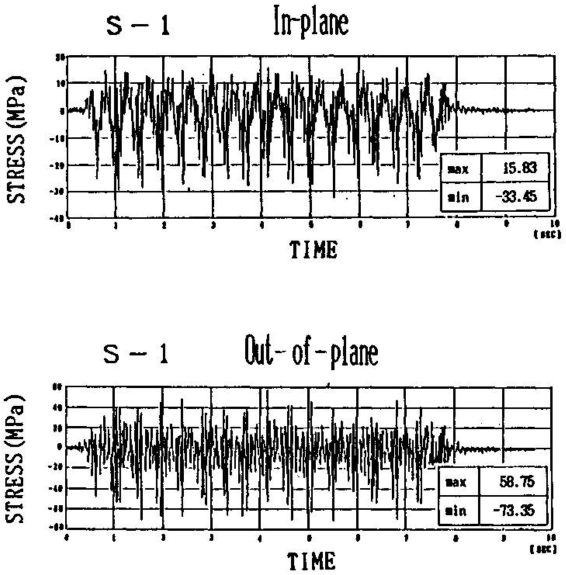


Fig.8 Stress component in In-plane and Out-of-plane of web

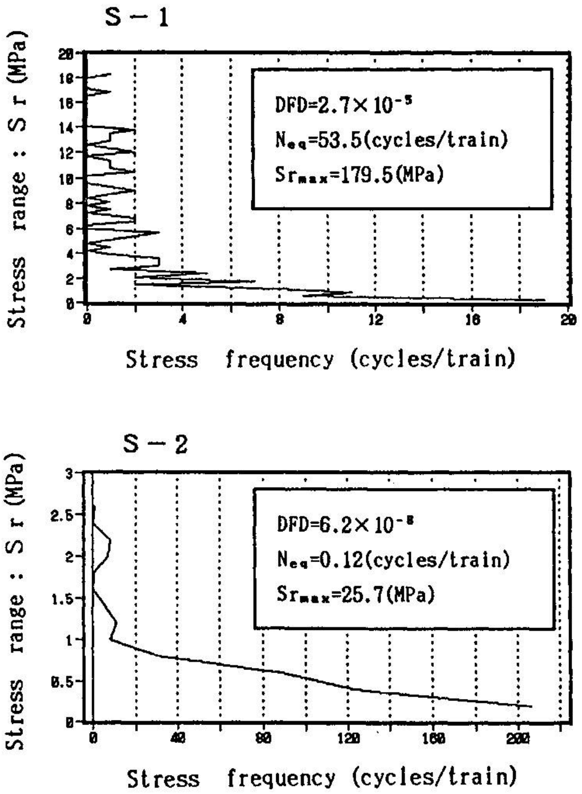
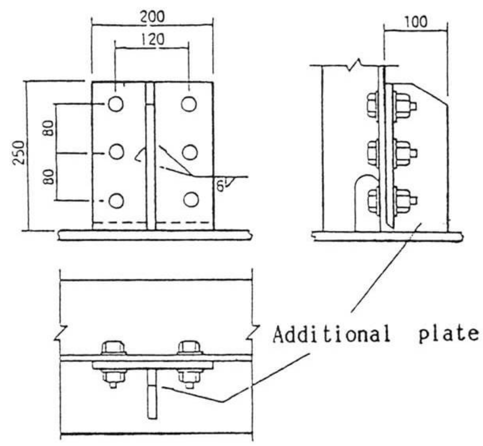


Fig.9 Stress frequency and Degree of Fatigue damage (DFD)



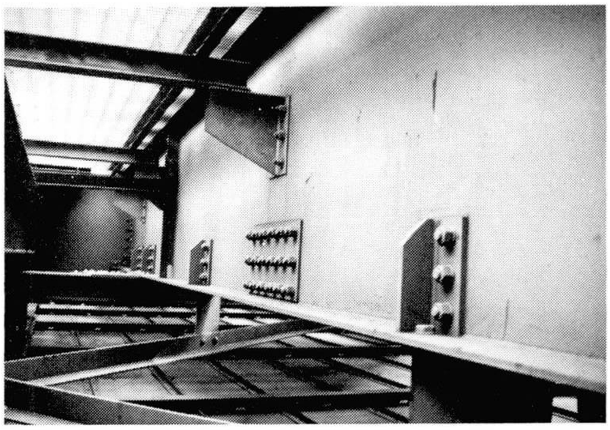


Fig.10 Retrofitting by additional plate

5. RETROFITTING METHODS

As a countermeasure against these fatigue cracks, it is most effective to stop local horizontal vibrations of lower flange which cause fatigue cracks. In some truss bridge stringers, fatigue cracks were removed by applying of arc air gouging and rewelded and then an additional plates were installed by high-strength bolts (Fig. 10). In some cases a sway bracing was also added. Another possible measure is to increase the fatigue strength of the detail. As it is costly to add plates to a slight fatigue crack, it was necessary to consider an economic method for raising fatigue strength. It is advisable to improve the fatigue strength of the toe of a fillet weld and prevent occurrence of additional crack in a part where occurrence of a fatigue crack is not confirmed as a preventive measure. Application of TIG (Tungsten Inert Gas) arc remelting has considered as an effective and economic method for repairing a fatigue crack at the toe of a fillet weld and improving fatigue strength.

5.1 Application Test of TIG Arc Remelting

Repair of the Lower end of a stiffener by TIG arc remelting is required to be conducted, while ensuring that the form of the toe of a fillet weld will be smooth and that penetration will be deep enough to fuse slight fatigue cracks which occur at the toe and is visually undiscoverable. The appearance of the bead under various application conditions and the amount of fusion in the boxing-welded portion were investigated. As a result, it was found that under the conditions as shown in Table 3 fusion was about 2mm in the case of stand-up position and the shape of the bead was in good condition. It was also found that confirmation of the position of a crack is necessary in carrying out the work because there is the possibility of a gap occurring between the position of the maximum fusion by the TIG dressing and the position of a crack.

Electrode diameter	3.2mm
Current	240A
Voltage	13~14V (arc length 0~1mm)
Speed	45~60s/100mm
Aiming position	0 to 1mm from the toe of weld
Torch angle	90°

Table 3 Condition of TIG arc remelting

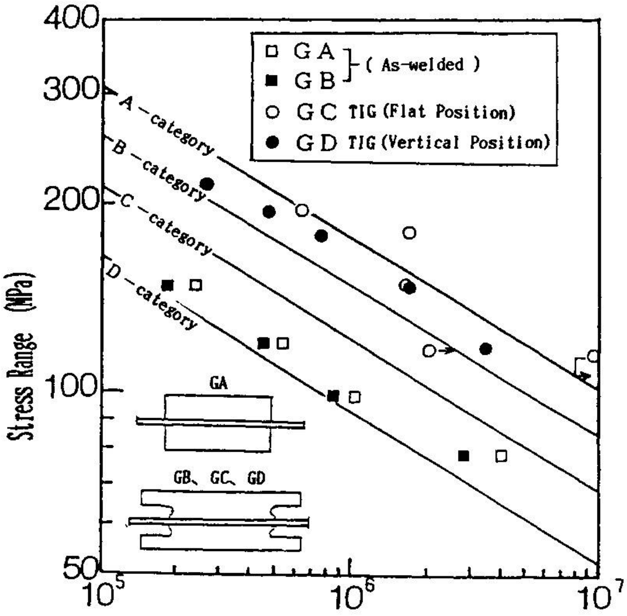


Fig.11 Results of fatigue test

5.2 Improvement of Fatigue Strength by TIG Dressing

In order to confirm the effect of the TIG dressing on this stiffener end detail, fatigue testing was conducted on specimens simulating this detail and applying conditions in actual bridges. Two type of specimens(GC and GD) were TIG dressed under the conditions as given in Table 3. Specimens GC were treated by TIG arc remelting in a face-down position and specimens GD were TIG dressed in a stand-up position, considering actual application conditions.

The result of fatigue testing is shown in Fig. 11. It was found that the fatigue strength of as-welded specimens slightly exceeded the category D of the design curve, while the fatigue strength of TIG dressed specimens exceeded the category B curve. The TIG dressing improved the shape of the toe of weld which affects occurrence of a crack and reduces stress concentration, thus improving fatigue strength.

6. OTHER CONSIDERATION IN FIELD APPLICATIONS OF TIG DRESSING

In executing the work, it was decided that the working procedure would be established as given in Fig.12. All details were observed by qualified inspectors by using eddy current test and magnetic particle test. Before applying the TIG dressing, the shapes of beads were examined. As a result, no special problem was found in the appearance of the beads. However, due to excessive grinding of beads at the time of manufacturing, throat depths of fillet welds were found to be unsatisfactory in some details. In such a weld, in addition to the countermeasures as given in 5, a measure to prevent occurrence of a fatigue crack from the root is necessary. One or two passes of fillet welds were added to such details.

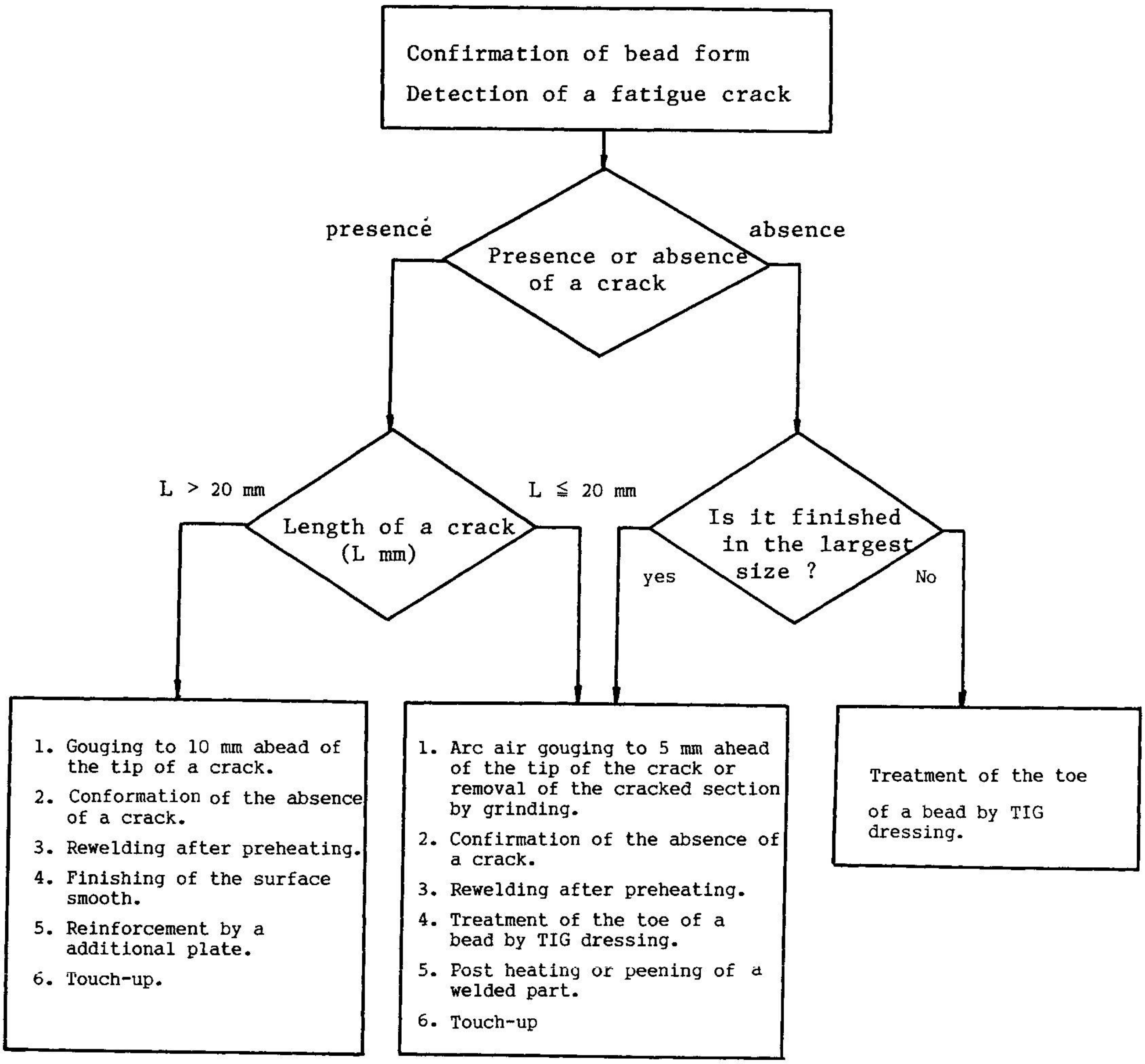


Fig. 12 Retrofitting process

7. CONCLUSION

The retrofitting by the TIG dressing was conducted in 300 box girders. The repaired parts have been inspected specially emphatically but recurrence of a fatigue crack has not been reported.

At present, an evaluation of current soundness is underway for all steel bridges of the Tokaido Shinkansen in preparation for higher speeds and larger transport capacities in the future. As part of this program, several bridges of all types were selected as model bridges and various studies including local stresses, fatigue crack growth behavior in various structural details, and monitoring system of fatigue cracking are being performed.