Structural Eng. / Earthquake Eng. Vol. 6. No. 1, 31s-39s. April 1989
Japan Society of Civil Engineers (Proc. of JSCE No. 404/I-11)

REPAIR OF FATIGUE DAMAGE IN CROSS BRACING CONNECTIONS IN STEEL GIRDER BRIDGES

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Practical repairing methods for fatigue damage in cross bracing connections in steel girder bridges are studied experimentally. Fatigue cracks initiated in the welded joint between upper flange of longitudinal girder and transverse stiffener to which cross bracing is connected. Various repairing methods are examined and the increasing weld size with TIG dressing is proposed as one of the most suitable repairing method.
Keywords : fatigue, repair, steel girder bride, TIG dressing

1. INTRODUCTION

In recent years, fatigue cracks have been found at welds between vertical stiffener and the top flange of main girders, where cross bracing or transverse girders are attached in highway plate girder bridges as shown in Fig. 1[1]. The occurrence of fatigue cracks at such locations is most typical of fatigue damage to highway bridges in Japan. Such fatigue damage is featured by occurrence of many cracks around the same time at the same kind of detais of bridge, in spite of some amount of differences in details and qualities of fabrication.

The causes of occurrence of this fatigue damage are considered to be secondary stress induced due to relative vertical deflections between adjacent main girders, and to deflections of concrete slabs[2]. In effect, forces of out of plane direction of girder web apply on vertical stiffeners with cross bracing, and it may be said transmission of these forces occurs through fillet welds between stiffeners and top flanges or webs.

Two kinds of retrofitting methods are conceivable for the prevention and repairing of such fatigue damage;

(1) Lowering stresses occurring at the crack location,

(2) Increasing the fatigue strength of joint.

For the former "lowering stresses", it is necessary to alter the structural systems and details, in such ways as connecting the upper strut of cross bracing directly to the upper flange of main girders, increasing

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plate thickness and size of vertical stiffeners[3]. These methods may be very desirable measure in new girder design since they will eliminate causes of damage. However, for application to existing structures, it is often difficult to realize expected structural strength because the work condition in the field is not so good. For example, welding between the vertical stiffener and the top flange would be over-head welding in a confined and narrow space, which could well be the cause of new fatigue damage.

The present study is on the method of repairing fatigue damage occurring at cross bracing attachment from the standpoint of "increasing the fatigue strength of joint". This is based on the follwing conditions observed in many girders in Tomei-Expressway.

(1) Inadequacies of throat depths and extremely unequal leg lengths in weld (fillet welds between top flange and vertical stiffener).

(2) Most of the cracks discovered are of very small sizes occurring at the edge surfaces of vertical stiffeners in spite of having been subjected to exceedingly severe traffic loads for more than 15 years.

(3) These fatigue cracks would not directly lead to catastrophic fracture of the bridge even if they were left unfixed.

(4) At all bridges considered in this study, stringers have been added to the original structures with the purpose of reinforcing concrete slab. And this has been confirmed to be useful in alleviating stress at the locations of fatigue damage by stress measurement of actual bridges and structural analyses[4)5)].

(5) Since the locations to be repaired are very numerous, it is desirable to develop a method to be executed readily, surely, and economically as much as possible.

2. SPECIMENS

In cross bracing connections of plate girder bridges, the locations where the most damage occurs are fillet welds between vertical stiffeners or cross-bracing connection plates and top flanges. When this

AB type+D type Crack

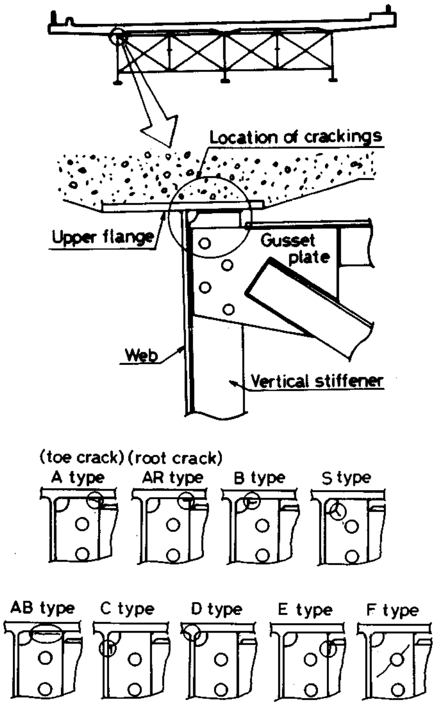

Fig. 1 Types and locations of crackings.

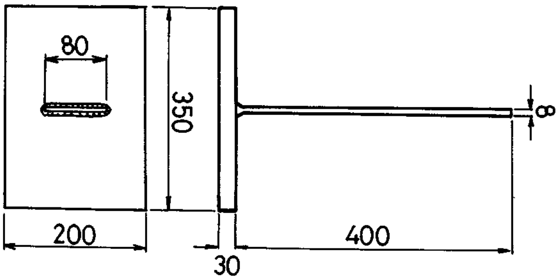

Fig. 2 Dimmensions of test specimens (mm).

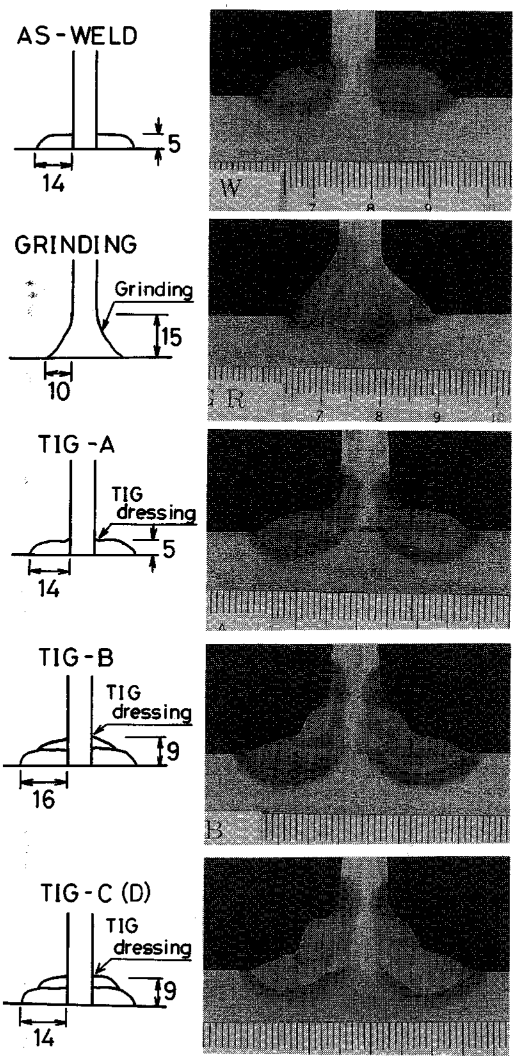

Fig. 3 Weld shapes of test specimens.

portion is extracted, it can be considered as a cruciform fillet weld of load carrying type. The shape and dimensions of specimens which simulate the joint between a vertical stiffener and an upper flange are shown in Fig. 2. These sizes were decided with reference to the actual bridge in which damage occurred. The steel used for specimens was SM 41. The mechanical properties and chemical composition according to mill sheets are given in Table 1. Various kinds of repairs were made at the welds of the specimens. Two specimens were connected together back to back using high-strength bolts, and fatigue tests were performed by applying repeating axial force to an 8-mm plate which represents a vertical stiffener.

Weld details and macro-etch-examinations of the welds of various specimens are shown in Fig. 3. The dimensions of weld detail in Fig. 3 are the averages of measured values.

The AS-WELD specimens are reproductions of fillet welds of the damaged portions. The configurations and dimentions of fillet welds were measured at actual bridges by using dental modelling compound. Based on the these results the configurations and dimensions were set for the specimens. The feature is that throat depths are significantly small and leg lengths were extremely unequal.

The GRINDING specimens are welded by full penetration welding and their weld toes are finished by grinder.

The TIG-A specimens are welded in the same way as AS-WELD specimens, and the toes on the stiffener

Table 1 Mechanical property and chemical composition.

Steel	Mechanical properties			Chemical composition (%)	
	Y. P. (Mpa)	T. S. (MPa)	El. (%)	C Si Mn x100	Cu S x1000
SS41	360	440	31	9 18 89	22 8

Y.P. : yield point, T.S. : tensile strength, El. : elongation

Photo 1 Bridge model used for making test specimens.

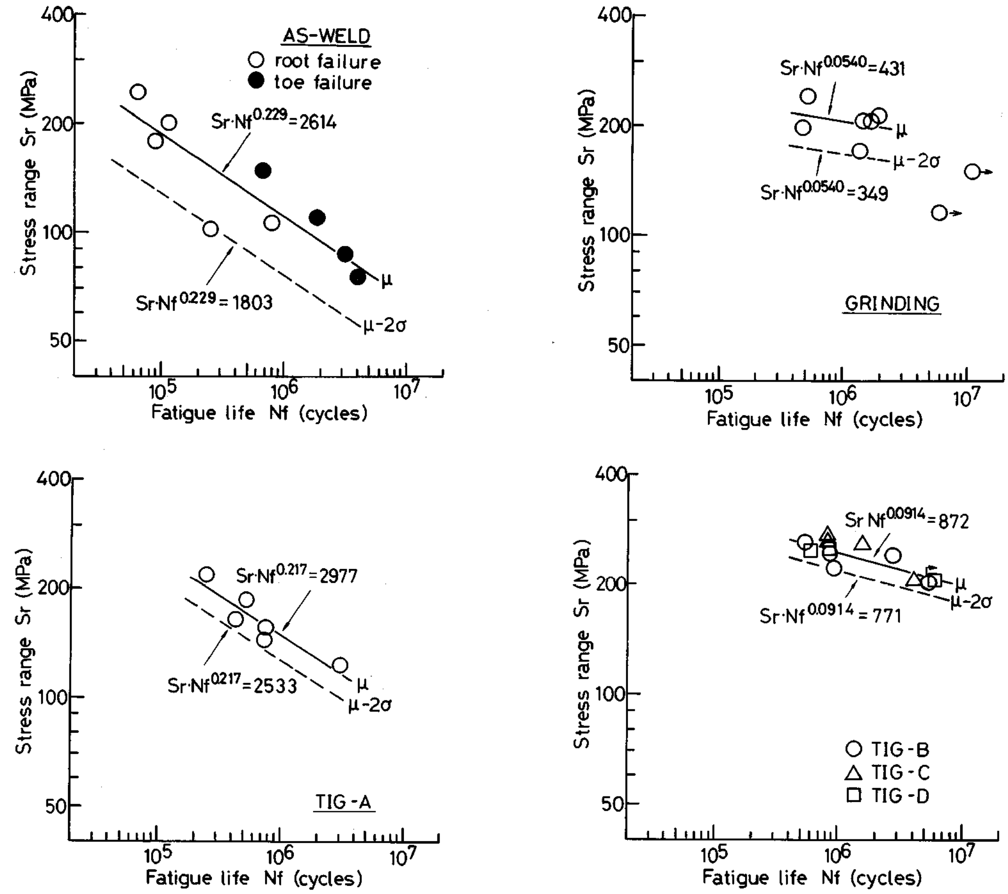

Fig. 4 Results of fatigue tests.

side are finished smooth by TIG dressing.

The TIG-B specimens have additional pass of fillet welding on top after the welding in the same way as AS-WELD specimens, and the toes are finished smooth by TIG dressing. The purpose of the additional pass of welding is to impose strength against root cracking by increasing the throat depth of fillet welds. In

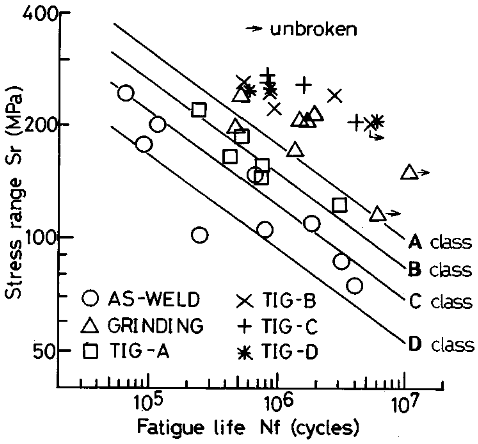

Fig. 5 Test results and standard design curves.

the actual bridges, consierable number of root cracks due to inadequate throat depth have been observed. It is also expected that existing small cracks are remelted when the additional pass is applied.

The TIG-C specimens are for ascertaining the that fatigue strengh in case of perfoming TIG-B type repairs in a condition of fatigue cracks are left in root portions. Based on the test results of the AS-WELD specimens, the load range was set so that failure would occur as 8×10^5 cycles, and repairing was done after loadig 7×10^5 cycles.

The TIG-D specimens have details identical to TIG-B, but TIG dressing was done in very adverse conditions assumable to occur in the actual field practice such as raising electric current and shaking the torch. Fabrication of TIG-A, B, C, and D specimens was performed in the model bridge shown in Photo 1 with the aim of reproducing repair work efficiencies at damaged portions of the actual bridge.

An electro-hydraulic fatigue testing machine of dynamic loading capacity of$\pm$50 tons was used for the fatigue tests. Loading pattern was Sinusoidal wave form with frequency of 5 to 10 Hz.

3. RESULTS OF FATIGUE TESTS

Fatigue test results are shown in Fig. 4 with mean line (μ line), and (μ-2σ) line (σ : standard deviation) which are obtained from regression analyses. All of the test results and the design allowable S-N curves in the Design Standards for Steel Railway Bridges[8] are shown in Fig. 5. Approximately half of the AS-WELD specimens failed due to fatigue cracks occurring from toes, while the remainder failed due to fatigue cracks initiated from roots. This indicates that the throat depths of these specimens were close to the limits of root cracks and toe cracks[6]. Test results of cracks occurring from toes were distributed in the long life range, and the fatigue strengths were slightly higher when compared with specimens failed due to root cracks. Such a trend was similar to the results of past studies[6]. According to the Design Standards for Steel Railway Bridges, joints of this type are classified into Class C under the condition that "adequate size of fillet weld will be possessed", that is, failure would occur at the toe. However, the test results in this case indicate that it would be not conservative to apply allowable stress of Class C to this connection. When throat depth of the weld is small and cracks occur from the root, it may be classified into D or poorer is indicated.

The fatigue strengths of GRINDING specimens matched the design curves of Class A with the exception of one specimen. Fatigue cracks occurred from the slightest surface defects in grinding finish. It is an extremely laborious task to accurately finish by grinder, and there are cases when sharp cuts and scars are left after grinding.

The fatigue test results of TIG-A specimens are plotted around the design curve of Class B with fatigue strengths slightly higher when compared with AS-WELD specimens. Because the configurations of the toe were made smooth by TIG dressing, fatigue cracks were initiated from roots in all specimens. It is thought that the improvement effect of fatigue strength by TIG dressing was not so significant because of the insufficiency of throat depth in the first place. However, it is possible to repair mimute cracks formed at the toe by merely performing TIG dressing because remelting can be done to a depth of about 2 mm from the surface by TIG dressing. The results of experiments here indicate that the strength of a joint made in this manner is considerably higher than AS-WELD specimens.

The fatigue strengths of TIG-B specimens were very high, far above the design life curve of Class A. In one of these specimens fatigue cracking occurred from the toe of the fillet weld on the side of the bottom plate (assuming a flange) with thickness of 30 mm, indicating the great effect of improvement in fatigue strength through the combination of fillet welding of one pass and TIG dressing. TIG-D specimens possesses almost the same fatigue strength of TIG-B specimens in spite of having slightly irregular surface.

The fatigue strengths of TIG-C specimens were also very high and equal to those of TIG-B specimens. Fatigue cracks were all formed from toes subjected to TIG dressing.

4. EVALUATION OF REPAIRING EFFECT

Fatigue damage in highway bridges is closely related to the number of tracks passing[7]. At the section of the Tomei Expressway where damage occurred, a total of 220, 007, 494 vehicles (eastbound and westboud combined) has passed through during the 16 years from the opening time 1968 to 1983. In view of the fact that the bridges of the eastbound and westbound roads are independent of each other, and from the make-up ratios of vehicle types, it is considered that approximately 30 million large vehicles had passed across in each direction and 2.5 million large vehicles had passed in each direction annually during 1978 to 1983. It is necessary to conduct further detailed investigations on the degrees of stress and the number of cycles of stress occurred at the individual locations of damage caused by the passing of these large vehicles. In this study, however, an approximate evaluation of the fatigue life remaining after repairs was made based on the actual situation of this traffic load and the fatigue strengths of welded joints repaired described in the preceding chapter.

If it is assumed that AS-WELD specimen is failed by the passage of 30 million large vehicles, the corresponding stress range, from the regression line of the S-N line in **Fig. 4**, will be

$$S_r=\frac{2\,614}{(3\times10^7)^{0.229}}\doteqdot50.7\ \text{MPa}$$

This value is in correspondence with the equivalent stress range in case of a variable stress range condition.

(1) The number of passages of large vehicles until fatigue failure in case of finishing toes by TIG, when evaluated by the average S-N line for TIG-A specimens, will be

$$N=\left(\frac{2\,977}{50.7}\right)^{1/0.217}=1.42\times10^8\ \text{cycles}$$

and if it can be assumed that 2.5 million large vehicles pass across annually, the remaining fatigue life will be approximately 57 years.

When considered on the basis of the μ-2 σ line,

$$N=\left(\frac{2\,533}{50.7}\right)^{1/0.217}=6.73\times10^7\ \text{cycles}$$

and this will be approximately 27 years.

(2) The number of large vehicles passing until fatigue failure, in the case of gouging out the damaged portion and finishing by grinder after complete penetrating welding, when using the average S-N line of GRINDING specimens will be

$$N=\left(\frac{431}{50.7}\right)^{1/0.0540}=1.63\times10^{17} \text{ cycles}$$

And, when considered on the basis of the μ-2σ line,

$$N=\left(\frac{349}{50.7}\right)^{1/0.0540}=3.27\times10^{15} \text{ cycles}$$

and the remaining fatigue life will be 1.3×10^{9} years.

(3) In case of repair fillet welding of the damaged portion by one pass and finishing the toe by TIG, the number of large vehicles passing, when the average S-N line for the TIG-B, C, and D specimens is used, will be

$$N=\left(\frac{872}{50.7}\right)^{1/0.0914}=3.3\times10^{1.3} \text{ cycles}$$

And, if the μ-2σ line is used,

$$N=\left(\frac{771}{50.7}\right)^{1/0.0914}=8.6\times10^{12} \text{ cycles}$$

Consequently, the remaining fatigue life will be 3.4×10^{16} years.

In case fatigue damage at the actual bridge was not after 16 years from start of use, but from a fairly early time, the above mentioned remaining fatigue life would be an overestimated one.

With Fig. 5 as a reference, and assuming the joint in the present condition to be Class D according to the Design Standards for Steel Railway Bridges[8], and this is to be improved to Class A by repairs, the remaining fatigue life will be

$$\left(\frac{15.3}{8.05}\right)^{4}\times\frac{3\times10^{7}}{2.5\times10^{6}}=157 \text{ years}$$

The fatigue strength given by the Class A design curve can be secured by one pass of reinforcement welding and TIG dressing. When improvement has been done up to Class B, the remaining fatigue life will be

$$\left(\frac{12.75}{8.05}\right)^{4}\times\frac{3\times10^{7}}{2.5\times10^{6}}=76 \text{ years}$$

Aa shown in Fig. 5 the fatigue strength given by the Class B design line, can be adequately secured by grinding, one pass of reinforcement welding, and TIG dressing.

5. PROPOSED RETROFITTING METHOD

Based on the above results of study described so far, it is proposed that the method as applied to TIG-B specimens be the basis for the repair method for actual bridges. According to the results of investigations of actual bridges, most of the fatigue cracks are Type A, as shown in Fig. 1, and their surface length is less than 10 mm at the edge of box welds. For cracks of this kind, gouging is not especially necessary and should be repaired by fillet welding performed on top and toe finishing by TIG dressing. This repair method can be applied similarly to cracks of B and C types also. The crack sizes to which this repair method can be applied would be of a degree that the surface length will not exceed plate thickness of the vertical stiffener.

In case a crack of Type A or B grows further, that part is to be completely removed by gouging and filled by welding, and moreover, additional welding performed overall. In this case also, the toe is to be finished by TIG dressing. When the crack becomes large, gouging is to be done over the entire length of the weld, and full penetration welding is to be performed. However, there is a possibility of adverse effects of large heat input by full penetration welding to the surrounding members and concrete deck plate. Therefore, extending full penetration welding to unnecessary portion must be avoided. There is considerable possibility of a crack initiated from the root having grown substantially at the root even though the length at the surface may be short. Therefore, gouging is to be performed until the crack can be completely removed. Subsequent treatments would be the same as the case of a crack formed from a toe.

Priority was given to TIG dressing instead of grinding as the method of finishing the toe of the weld, because of the sureness of the effect of finishing against fatigue and the efficiency of operations. As shown in TIG-D specimens, as for TIG dressing, there is hardly any reduction in the effect of improving fatigue strength even in case of fairly poor "appearance" of surface configurations. On the other hand, grinding with the purpose of improving fatigue strength must be done perfectly, up to the state of removing the toe of the weld completely, but it is fairly difficult to perform sure and adequate grinding at a large number of locations in the field where working conditions are unfavorable. As for the time required for operations on the job, it was approximately 3 minutes for TIG dressing in contrast to the approximately 15 minutes for grinding.

Phote 2 is a view of trial execution on an actual bridge. The upper supporting member of the cross bracing and a part of the diagonal member flange have been cut away because they hinder repairing of welded portions.

6. CLOSING REMARKS

Methods of repairing fatigue cracks occurring at edges of vertical stiffeners connected to steel composite girder cross bracing were examined from the viewpoint of increasing fatigue strengths of connections. The principal results obtained were the followings:

(1) The fatigue strength of a load carrying-type cruciform fillet welded specimen reproducing the configuration and dimensions of damaged portions was very low, and did not even meet the requirements for Class D according to the Design Standards for Steel Railway Bridges.

(2) The fatigue strength of a connection on which a single pass of fillet welding was performed and the toe was finished by grinder was very high, and the requirements for allowable stress of Class B in the Design Standards for Steel Railway Bridges were amply satisfied. The fatigue strength of a connection with the toe subjected to TIG dressing was even higher and satisfied the requirements for Class A.

(3) Remaining fatigue life after repairing was evaluated based on loading histories of the Tomei Expressway, which is the object of repairs and the fatigue tests carried out here, it was shown that ample remaining fatigue life can be obtained by the repair method proposed here.

(4) Considering the effect of improving fatigue strength, the sureness, and working efficiency, it can be said that TIG dressing is superior to grinding as a method of finishing toes of welds.

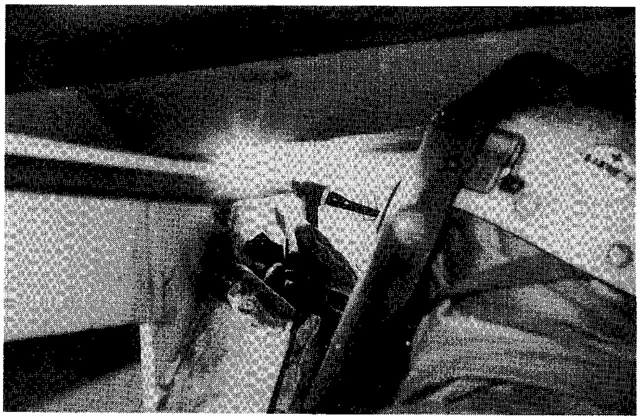

Photo 2 Repairing of actual bridge.

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(Received April 20 1988)