Bridge engineering learned from failures
-fatigue and fracture control-

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ABSTRACT: Fatigue and facture cases in bridge structures were reviewed from the view point of failures and retrofitting works. These cases were classified as follows, welding defects were included during fabrication, inappropriate structural detail of low fatigue strength had been adopted, stresses unforeseen in design occurred at joints of members, structures behaved in a manner not expected such as vibration due to wind.

1 INTRODUCTION

The history of bridge engineering is the history of failures. Bridge engineers have learned from failures and developed their engineering, whenever it was happened. The failure cases are the reservoir of information which brings the progress of bridge engineering. In the old days, many stone arch bridges collapsed due to floods and how to fly longer span was an essential issue in bridge constructions. Many suspension bridges fell down because of aero-dynamic instabilities and the preventive measures of safety against winds had been the major engineering issues in the design of long span bridges. A number of steel bridges in U.S.A., and Japan have been suffering fatigue cracking which sometimes resulted in brittle fracture since sixties of the last century. The occurrences of these accidents have been still increasing more and more. How to manage fatigue related problem has been important and urgent assignment to bridge engineers.

The causes of fatigue cracking accidents are widely varied, relating with design, construction and maintenance. The causes of fatigue of steel bridges may be classified as follows:

- Welding defects were included at the time of fabrication.
- An inappropriate structural detail of low fatigue strength had been adopted.
- Stresses and deformations unforeseen in design occurred at joints of members.
- The structure behaved in a manner not expected such as vibration due to wind.

According this classification, detailed review and summary of fatigue cracking cases of bridge structures are provided from the view point of failures.

2 EXISTENCE OF WELDING DEFECTS

2.1 *Butt welds in the flange plate of plate girder bridge*

There are cases of girders failing due to fatigue cracks occurring from welding defects unintentionally left in butt welded joints in bottom flanges.

Figure 1(a) is a case in U.S.A.. this bridge is two spans continuous girder(45.7 + 45.7m), constructed in 1949. This cracking accident was found in 1981. Fatigue cracks initiated from the roots of plate butt welds and penetrated through the bottom flange and resulted brittle fracture in web plate. The groove is X with full penetration, but, the wide zone of incomplete penetration was remained in the center part of joints. The area of incomplete penetration is about one half of section.

(a) In the USA (1949) (b) In Tokyo, Japan (1973)

Figure 1. Cracked Girder: Fatigue crack initiated from insufficient penetration

Figure 1(b) is the case in a simply supported composite girder bridge of 30.3m span in Tokyo, Japan. This bridge was constructed in 1973 and fatigue cracks were found in 1988. Fatigue cracks cut the whole section of bottom flange and arrested in fillet welds between the flange plate and the web plate due to large plastic deformation of weld metal zone (Figure 2(b). The observation of fracture surface revealed multiple fatigue cracks initiated along the root of incomplete penetration welds in the plate butt welds of bottom flange (Figure2(c)). The zone of incomplete penetration occupied about 60% of width and 60% of thickness of butt weld. The groove of butt welds in the design plan is K shape with full penetration, of course.

It is close to impossible to predict the occurrences of fatigue damage caused by such kinds of defects in groove welds with full penetration, because these are failures of substantial quality control of factory. However, after a single case has occurred, it will be possible to take measures such as intensive inspection of joints made in the same factory around the same time using the same welding details. In the case of Figure 1(b), as a result of NDI to butt welds in the same bridge, weld defects of incomplete penetration were discovered in 45 joints from 388 joints.

2.2 *Rigid frame bridge bents*

Figure 2 shows fatigue cracks in the bridge rigid bent in the route 3 of Tokyo Metropolitan Expressway. Fatigue cracks appeared on the surface of welded joint between the flanges of beam and column. The observation of the wall surface of stop holes, wide incomplete zones were remained in the welded joint and fatigue cracks initiated from the roots of partially penetrated welds. The shape of groove in the design drawing is K with full penetration.

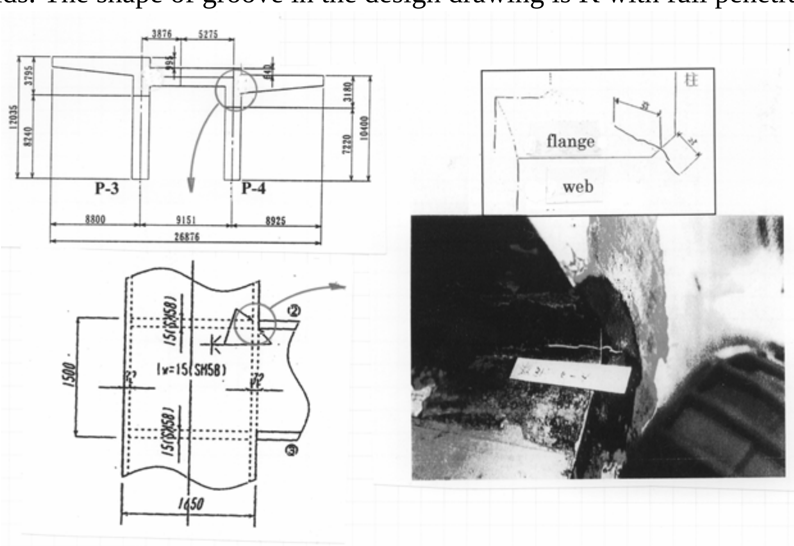


Figure 2. Fatigue Cracks in Rigid Frame Bridge Bent (Metropolitan Expressway, Rt.3, Ikejiri)

Because of complicated crossing of welding beads of plate elements of beam and column, assembling procedures of plate elements, welding details and quality control of these welds become very difficult issues. Particularly, at the corner of beam and flange connections, welded joints from three perpendicular directions come together and there is no weld access hole, welding works were almost impossible and big tunnel-like un-welded zone were remained inherently. These kinds of structural connection details lead the occurrence of inherent weld defects and become the causes of fatigue accidents, eventually. The existence of these inherent defect and stress concentration due to the structural geometry are the reason of this fatigue accident. The designers have to pay special attention to fabrication-ability to prevent the occurrence of defects. The old design methods have be revised even they have been used long days. They are applied beyond the scope of original inquires, sometimes.

3 ADOPTIONS OF JOINT DETAILS WITH LOW FATIGUE STRENGTH

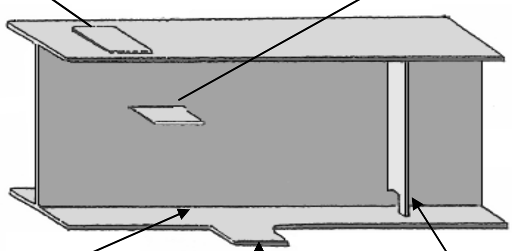
This would include cases of bridges not designed against fatigue in the first place being subjected to conditions of use where fatigue would be a problem. Most of the highway bridges in Japan are classified into this group. Joint details and attachments with low fatigue strength were included in high stress region and which leaded unexpected fatigue cracking accidents in many bridges.

As often occurs with old structures, the allowable stresses used in fatigue design on subsequent review being found to have been inappropriate. Figure 3 shows a comparison of the allowable fatigue design stresses of typical joints in steel railway bridges in 1960 and at present in Japan. Since 1970s, fatigue tests on full-scale and large models have been carried out in U.S. and Japan. The results of these tests revealed the strong size effects of fatigue strengths, the results of large size models were very low compared with existing results of joint specimens. According to these facts, allowable fatigue stresses were reconsidered and made the present values.

According to this fact, comprehensive fatigue damage prevention works had been performed for all of steel bridges, total number of girders is 1511 and total length is 27km, in the Tokaido Shinkansen (the bullet train system). In order to assess fatigue damage, stress measurements under service conditions were performed in many bridges covering all types of bridges. These

Joint Type	Sleeper pad	
Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	$\overline{D}$	120
Current code '92	F	65

Joint Type	Out-plane gusset (t=20mm)	
Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	D	100
Current code '92	F	65



Joint Type	Longitudinal welding	
Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	B	150
Current code '92	C	125

Joint Type	Cruciform weld	
Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	D	100
Current code '92	E	80

Joint Type	In-plane gusset (t=20mm)	
Code	Class	Fatigue strength at 2 million cycles (MPa)
Old code '60	C	126
Current code '92	F	65

Figure 3 Change of allowable fatigue stresses from ’60 code to ’92 code

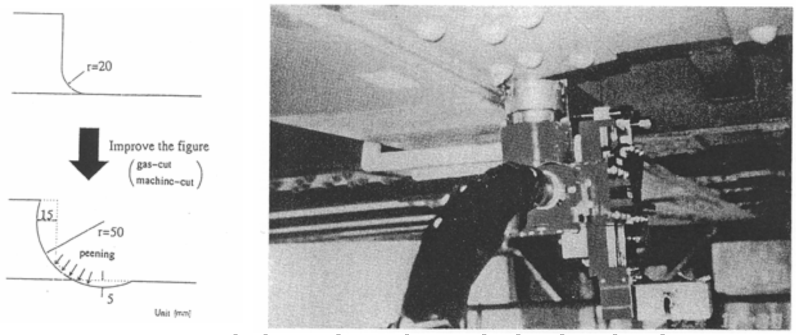


Figure 4. Cutting works by applying the newly developed tool

measured results indicated actual stresses in girders were fairly lower than calculated stresses, about 50-65% of calculated ones. The possibility of fatigue damage in a large number of members is high on the bases of the design stresses, but there are a few cases when the actual stresses are analyzed, eventually. From these studies, it was indicated that only the ends of in-plane gusset plate attached to flange plates were the details with low safety margin against fatigue.

As a preventive measure, improving the fatigue strength of gusset end detail was studied. Figure 4 shows one of the methods being proposed to gusset plate on flange edge, in which a specially developed circular cutting device as shown in Figure 4 is used to shape the radius of the gusset end large and the fatigue strength is increased by reducing local stress concentration at this location which becomes the initiation point of fatigue cracking.

4 OCCURRENCE OF UNEXPECTED STRESSES AND DEFORMATIONS

Various simplifications and assumptions have been established in order to calculate stresses in the structural members in bridge design. Particularly, with regard to connections between perpendicularly crossing members such as main girders and cross beams, and cross beams and stringers, simple supports or pin connections are often specified. Consequently, stresses and deformations obtained from design calculations differ considerably from these in actual structures, especially in the vicinities of connections. These calculations are on the conservative side when determining cross-sectional dimensions, but are often the causes of fatigue because of secondary stresses due to restraint moments. Other unexpected stresses and deformations cases are at concentrated loading regions such as supports of girder. Fatigue damage around the deck plates of orthotropic steel bridge decks is also caused by local deformation due to concentrated wheel loads.

4.1 *Stresses due to Restraint Moment at the Connections of Crossing Members*

(1) Main girder-floor beam connection in through type plate girder
Figure 5 shows a through-type plate girder railway bridge in which fatigue cracking occurred when the flanges of floor beams were provided with cut-outs to facilitate joining at connections attaching floor beams to main girders (Miki. 1995, Miki et al. 1989). In the design, floor beams are provided simple support by main girders and thus only shear force, reactions of cross beams, are transmitted at the joints, and there is no problem if cut-outs are made in flanges. However, since floor beams are fixed to main girders in the actual structure, fixed-end restraints moments are produced. As a result, fairly large direct stress components will occur in the flanges. Because of the large decrease in cross section and change in configuration due to cut-outs of the flanges, large stress concentrations occur at these locations.

(2) Main truss chord to floor beam Connection
Figure 6 shows fatigue cracking which occurred at the connection between floor beams and panel points of trusses in a deck-type truss girder highway bridge. This damage is due to the occurrence of restraint moment at the end of floor beam. Because, the relatively thin connection plate was connected only with the web plate by single splicing bolted joint, extremely high

stresses were originated at the top end of welded joint between the connection plate and the web of top chord.

As the reinforcing measure the top flange of a floor beam and the flange of the upper chord member of a truss should be joined using a connection plate. In order to adjust the gap and difference of level between the flange of top chord, thick plates of 40mm are adopted as connection plates. In order to secure enough space for welding works, concrete at the top surface of the truss was blasted off by water jet.

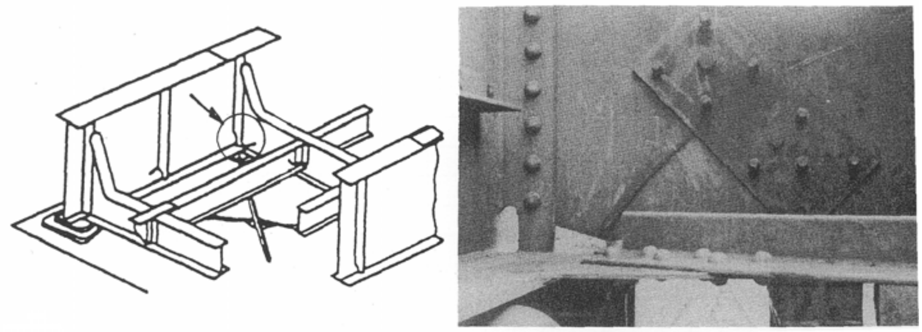


Figure 5. Through-type plate girder railway bridge, Fatigue cracks initiated at the connections between main girders

(3) Cross bracing connection in plate girder bridges

Figure 7 shows fatigue cracking which occurred at welds between a vertical stiffener used attaching cross bracing of a plate girder bridge and an upper flange and web. Such fatigue is due to fairly large forces being produced at the various members of the cross bracing from the difference in deflection between main girders caused by vehicle loads and the forcible deformation of the upper flanges of plate girders by deflection of the concrete deck in a direction perpendicular to the bridge axis.

Various methods are employed for repair of this damage considering degree of damage and ease of execution. For the smallest cracks, in case there is no problem about size with fillet welds, re-welding the crack is done by TIG dressing. In case of small size of fillet weld with the possibility of crack occurrence from the root, after one to three passes of fillet welding on top of the crack, the toe is finished by TIG or grinding. When the crack is large, it is removed by

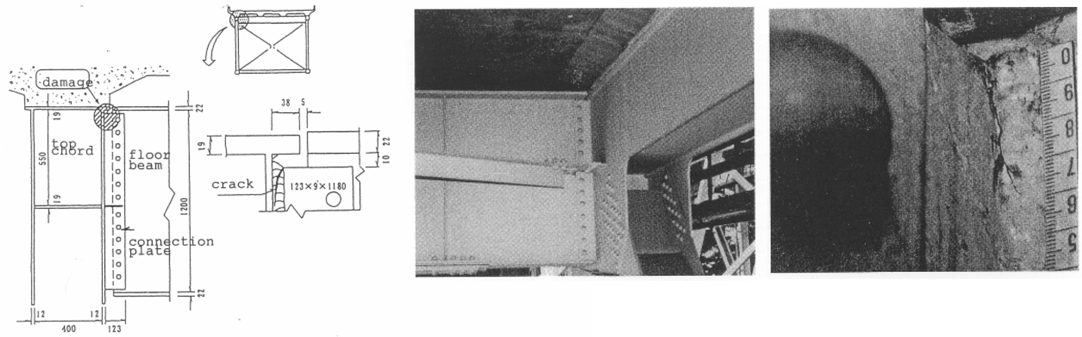
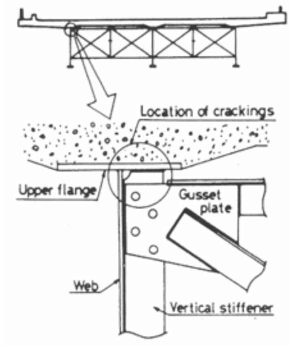


Figure 6. Fatigue crack at the connection between floor beam and truss panel point



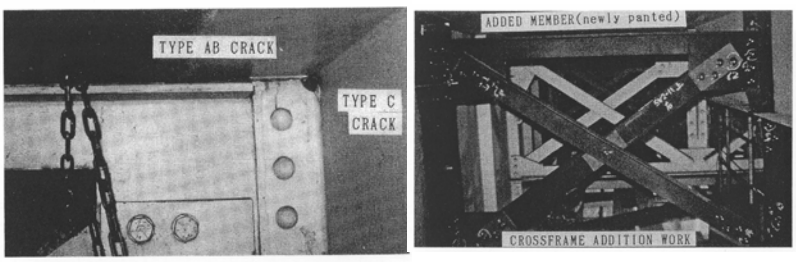


Figure 7. Fatigue damage case at the cross bracing connection details

gouging after which re-welding is performed by full-penetration welding, with toe finishing done by TIG or grinding. The results of follow-up investigations indicated that most of the repaired spots remained sound, but fatigue cracks have occurred again where stresses produced were large and where cracks had been large.

Improving the deformation behavior of the structure as a whole is also done by installing cross bracings and stringers. In this case, the plate thickness of vertical stiffeners for attaching new cross bracings is changed from the about 10 mm used in the past to about 16 mm, while their widths were made as large as possible. Further, to lower stresses at damaged parts, horizontal members of old cross bracings were removed.

4.2 *Stresses due to concentrated loads*

(1) Sole plate end

Support points on a bridge may be said to be points of the greatest load concentrations on the bridge. Beam theory is normally employed in designing a bridge, but when sizes of girders and locations of supporting points are considered, the stresses occurring near support points differ considerably from those obtained by the beam theory. Consequently, the regions around of supporting points are susceptible to fatigue damage.

Figure 8(a) shows fatigue cracking which occurred in a plate girder bridge where a sole plate was attached to the underside of a bottom flange by fillet welds(Tateishi et al. 1994). For sole plates, when rotating function is lost, very high stresses occur at the front surface of the sole plate.

An example of repair is shown in Figure 8(b). The sole plate was changed to a longer one. Joining with the bottom flange was done using high-strength bolts. Cracks which had penetrated into the web, holes were drilled at the tips along with which splicing was done using high-strength bolts. The holes at the tips of cracks were also stopped with high-strength bolts. When cracks of webs are large, welding may also be done.

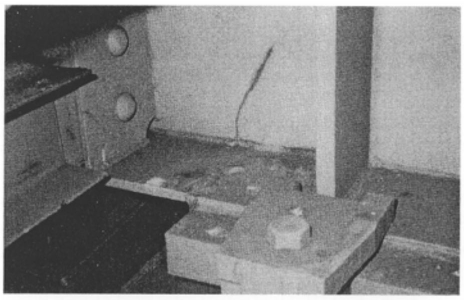


Figure 8 (a). Fatigue Crack at Sole Plate End

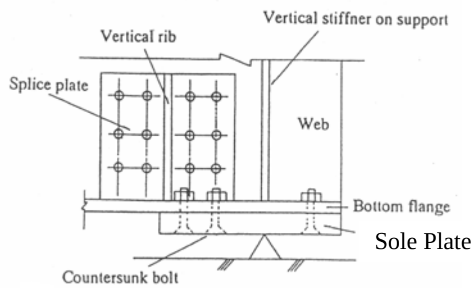


Figure 8 (b). Retrofitting method for sole plate detail

(2) Cut-out web at girder end

Figure 9 shows fatigue cracking which occurred where a cut-out had been made at the end of a girder of a plate girder bridge. Regard to this part, bending stress is extremely small according to the beam theory, but since a cut-out is provided, a high stress component is produced in the normal-line direction at the corner of the curved part. This stress component becomes higher the smaller the curvature ratio of the curve. When the bearings of supports loss the functions of rotating and sliding, the stress raising at the corner of cut out web becomes more severe. Since the flange and the web are joined by fillet welding, when looked at locally, this fillet weld is supposed to transmit load. Fatigue cracks often are initiated from the roots of welds because of this. This means fatigue cracks appeared on the surface after it grew long.

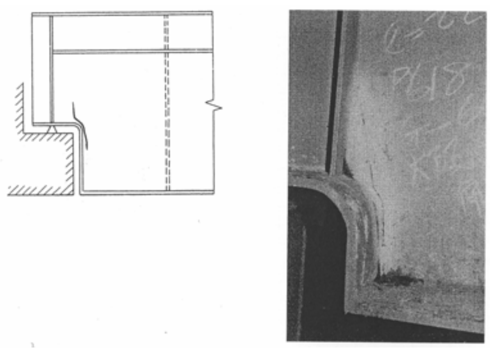


Figure 9. Fatigue Crack at Cut-out Girder Ends

(3) Orthotropic steel bridge decks

The orthotropic steel deck system is light weight compared with concrete deck slabs and suited to long span bridges. Because decks support traffic loads directly and the thinner orthotropic steel bridge decks are flexible, actual stresses due to traffic loads in elements are different from these in design calculation, and fatigue is very severe. This is also concentrated loading oriented problem.

Figure 10 shows the fatigue cracks in one box section girder bridge in the Metropolitan Expressway. Fatigue cracks have developed at various joint details, including welded joints of trough ribs used backing strips, scallops at the intersections of the longitudinal and transverse ribs and welds between trough ribs and deck plate.

The detailed studies through FEM analysis and field measurements indicated the main reason of fatigue damage in the longitudinal ribs and deck plates is the local deformation of deck plates between two webs of trough ribs as punching shear behavior of plate. In order to prevent the occurrence of these fatigue cracks, the increase of the thickness of deck plates is essential and the width of trough rib must be less than the width of truck tires, approximately less than 400mm. As the retrofitting method, the reforming to the composite steel bridge deck by applying steel fiber reinforced concrete has been studied.

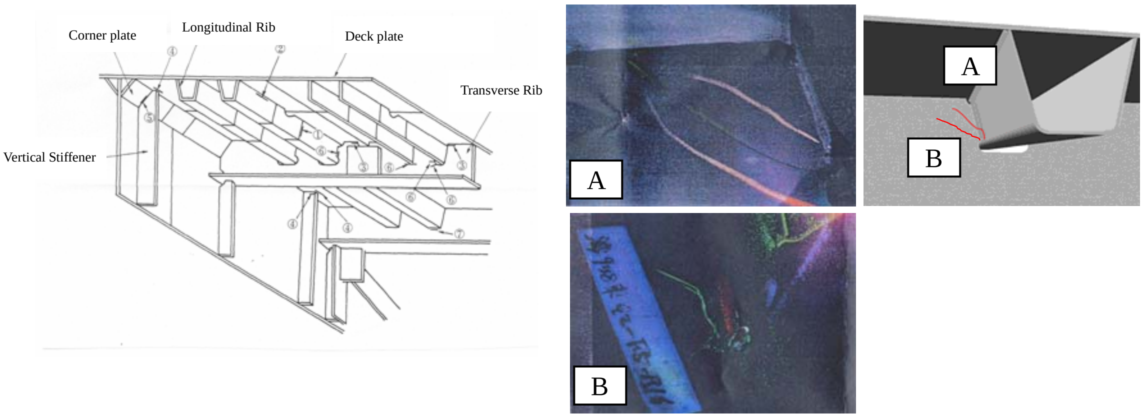


Figure 10. Fatigue cracks in orthotoropic steel decks

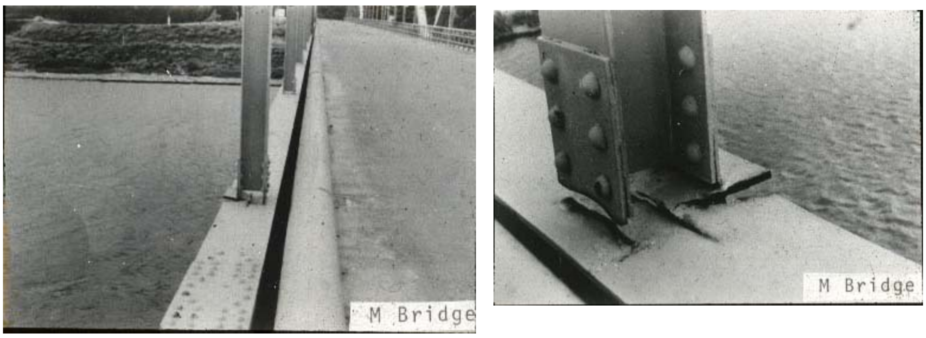


Figure 11. Fatigue damage caused by vibration due to wind.

5 VIBRATION

5.1 *Wind*

When winds applied on bridge members of dull sections such as H-shape, box section and circular section, vortexes are originated in the back steam of wind, and these vortexes induce vibrations on bridge members, called Karman vortex shedding. These unexpected vibrations cause fatigue damage in bridge members. Figure 11 shows a case in an arch type bridge, fatigue cracks cut the connection plates between the main girder and the vertical suspension members. Fatigue cracks propagated into the flange of main girder. Similar fatigue damage in vertical members occurred in several arch type bridges in Japan. Various Types of retrofitting works have been tried. In order to suppress the occurrence of vortexes, the surfaces of members changed to rough by winding spiral wires with members which provides easy separation of air stream. The change of natural frequency of members is also one effective countermeasure. Vertical members were connected by applying wires. The details of connections were improved to be high fatigue resistance.

5.2 *High speed train*

When a train travels at high speed, there may be a case of vibration of a kind unforeseen occurring in a bridge. Figure 12 shows examples of fatigue damage due to vibaration in bridge structures of the Shinkansen Line of Japan. Since start of operation in 1964, speeds have been increased and from about the time that 200 km/h was exceeded, vibrations of bottom flanges of stringers in plate girder bridges and truss girder bridges in directions perpendicular to bridge axes began to appear prominently when crossed by trains. Also, vibrations in out-of-plane directions occurred in diaphragms of box-section girder bridges. Fatigue cracks have occurred in vertical stiffeners and ribs restraining such vibrations.

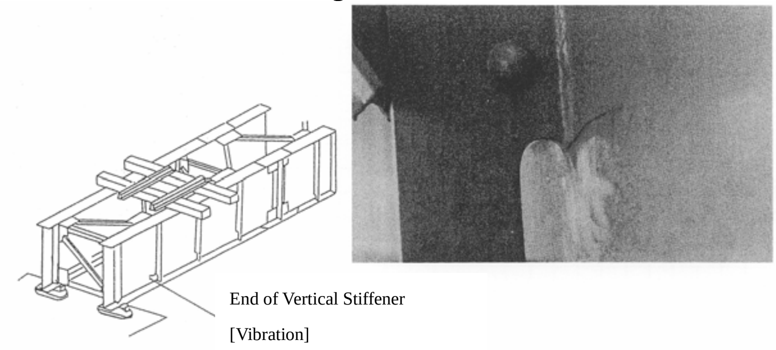


Figure 12. Fatigue damage due to the out of plane vibration of bottom flange of girder in bullet train systems

6 CLOSING REMARKS

As mentioned at the beginning, the failure experiences provide essential and indispensable information for the development of bridge engineering, particularly, bridge maintenance technology. International Institute of Welding(IIW) Commission XIII "Fatigue Behavior of Welded Components and Structures, Chairman: S. Maddox" Working Group 5 "Repair of Fatigue Loaded welded Structures, Chairman: C. Miki" has been collecting fatigue cases and establishing data base. The part of bridge cases is provided in the web site "http://iiw-wg5.cv.titech.ac.jp/. This web site is also linked from the IIW web site.

Bridge maintenance technology is a highly sophisticated discipline combining materials, welding, structural response, structural dynamics, strength evaluation, fracture mechanics, and non-destructive evaluation. These are all of elements of bridge design, fabrication and maintenance. Because of the long life of bridges, some cases over 100 years, we have to know not only the present states of these, but also the whole histories of these. However, we are able to apply new advanced technologies to this difficult task. A number of bridges have been aging and facing some kinds of deterioration in Japan. The establishment of maintenance technology is pressing issue for us, because, bridges play key roles of road and rail networks that support our society and economical activities.

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