Design and retrofit for fatigue damage in web gap

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Design and Retrofit for Fatigue Damage in Web Gap

Calcul et réparation des dégâts dus à la fatigue dans les âmes de poutres

Bemessung und Reparatur von Ermüdungsschäden im Steg von Stahlträgern

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SUMMARY
Unanticipated fatigue cracking has resulted because of out-of-plane distortions as a result of the three dimensional behaviour of structures. This paper provides a summary of two separate studies. In one, a recommendation is provided to minimize cracking in the web at diaphragm connection plates in right bridges. The second reviews retrofit procedures for the cracking in longitudinal girder webs at floor beam connection plates.

RESUME
Des fissures imprévues de fatigue ont été constatées à cause des distorsions hors de leur plan des âmes de poutres dues au comportement tridimensionnel des structures. Cet article fournit un résumé de deux études séparées. Dans l'une, une proposition est faite en vue de diminuer le risque de fissuration dans l'âme au droit des raidisseurs pour les ponts. La seconde présente des méthodes de réparation des fissures se produisant à la liaison des âmes de poutres-maîtresses avec les entretoises.

ZUSAMMENFASSUNG
Infolge der räumlichen Tragwirkung stellten sich beim Anschluss der Querträger an die Längsträger unerwartete Ermüdungsrisse ein. Der Beitrag behandelt zusammenfassend zwei verschiedene Untersuchungen. In der ersten Studie wird eine Empfehlung gegeben wie die Ermüdungsrisse beim Anschluss der Querträger an die Stege der Brückenlängsträger auf ein Mindestmass beschränkt werden können. Die zweite Untersuchung bespricht Reparaturmethoden für die Risse in den Stegen der Längsträger.

1. INTRODUCTION

The past ten years have seen much with regard to the development of fatigue design criteria for bridges [1]. Design curves have been established for typical details, and yet fatigue cracking of bridges still occurs. Most frequently, this cracking has resulted because of unanticipated distortions as a result of the three dimensional behavior of the structure. However, cracks have also developed from poor design details and large initial flaws which were not detected nor recognizable as being significant at the time of fabrication. Most bridges are designed for two dimensional behavior with slight provision, if any, listed for consideration of the third dimensional effects on the structure [2].

This paper provides a summary of two separate studies. In one, an analysis and recommendation to minimize cracking in the web at diaphragm connection plates of right bridges is provided. In the second, retrofit procedures are suggested for the cracking that has developed in the longitudinal girder webs at floor beam connection plates.

The diaphragm connection plate is assumed to be welded to the web and compression flange but cut short or tight fit to the tension flange. A comparable condition existed at the floor beam connection plates. A review of a typical floor beam girder bridge is given in Ref. 3, together with the results of an analysis and summary of experimental measurements.

2. DESIGN CRITERIA FOR DIAPHRAGM CONNECTION PLATES

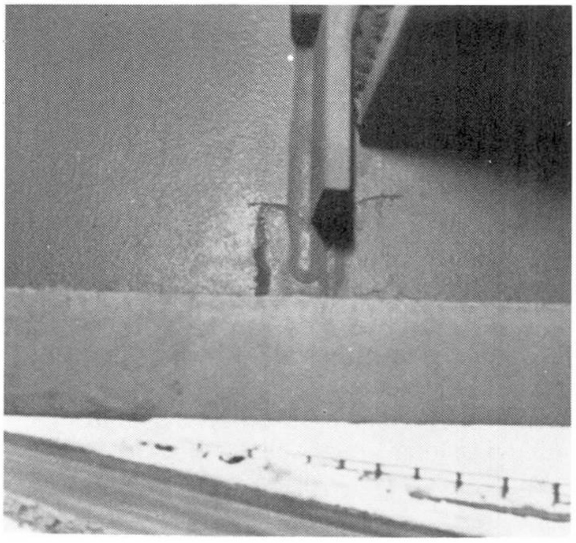


Fig. 1 Cracking at Transverse Diaphragm Web Connection Plate (Skewed Bridge)

Figure 1 shows an example of a detail which developed a fatigue crack. This crack is caused by displacements in the third dimension which occur as a result of interaction between primary (main girders) and secondary members (cross-frames, diaphragms, lateral bracings, etc.). This is the out-of-plane displacement which is not considered in the present design practice [2].

To define the interaction between the primary and secondary members, and to determine the extent of the contribution of various design parameters to the stresses developed in the web gap region (e.g. web thickness, flange thickness, gap length, etc.), an analytical study was undertaken [4]. The scope and results of the study are too extensive to be completely covered in this paper. Thus, only the highlights of this study will be presented. A "typical" bridge was chosen to be the prototype for the investigation (Figs. 2 and 3). All design dimensions and details of the bridge reflect the current design practice in the U.S.A. This structure was modeled using the finite element method and Program SAP IV [5]. The primary finite element model accounted for all structural details. Subsequently, the structural detail region of interest, the web gap region, was substructured using a highly refined finite element mesh. Various design parameters were changed to assess their effects on the stress concentration near the web gap region. Some of these parameters were: different vehicular configurations, placement of the vehicle, stiffness of the cross-bracings, connection details of the cross-bracings, web and flange thickness of main girders, and web gap length.

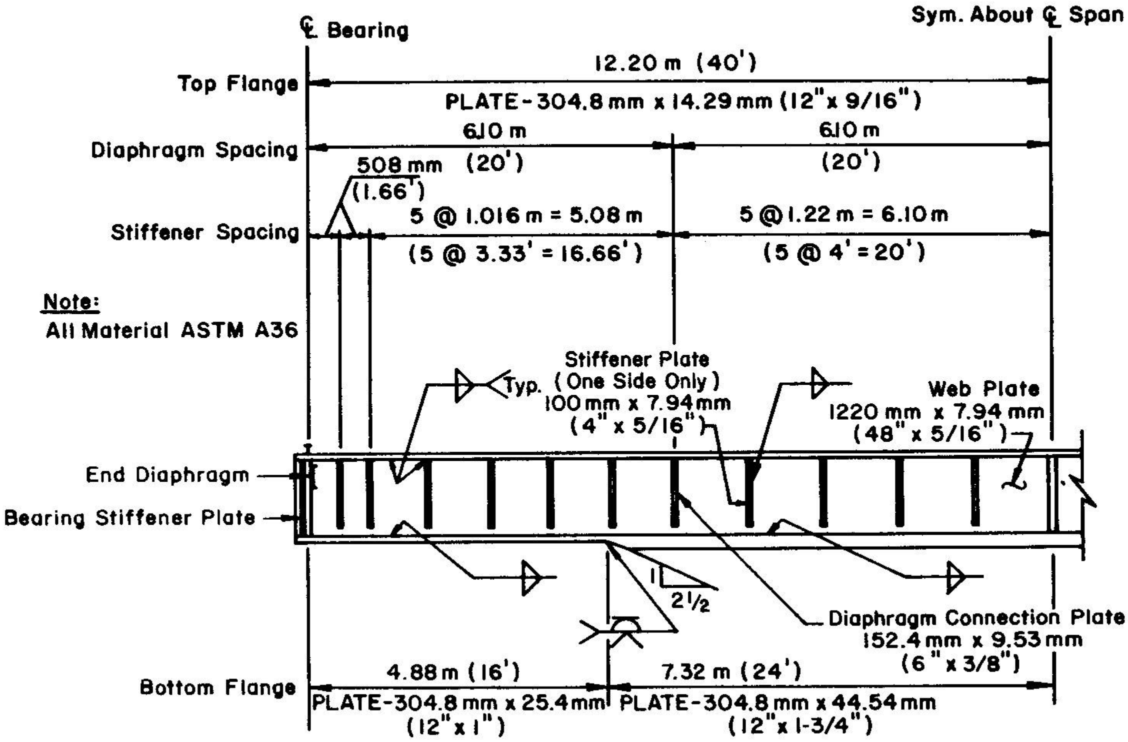


Fig. 2 Elevation of the Bridge Girder

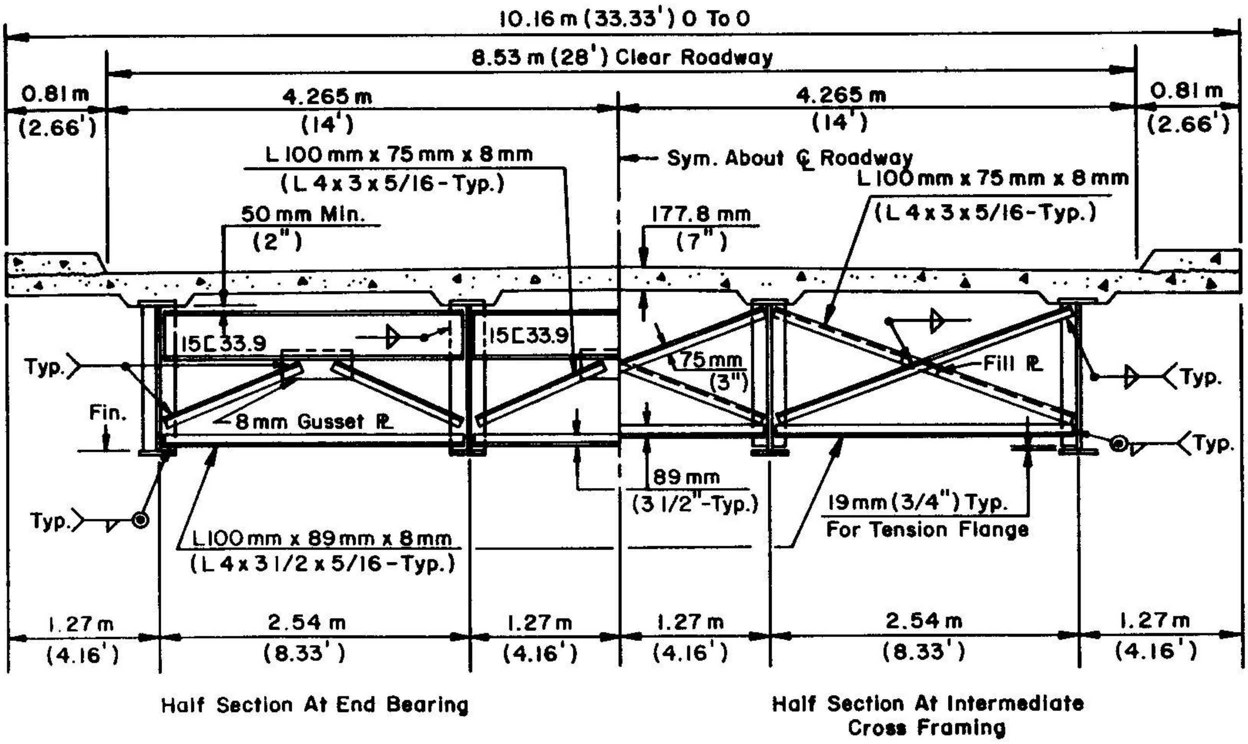


Fig. 3 The Cross-Section of the Bridge

The study indicated that a large stress level exists in the web in the immediate vicinity of the tip of the cut short stiffener. The stress buildup decays rapidly along the length of the girder. The cross-bracing introduces nonnegligible local deformations in the web of the girders. All common detail connections between the cross-bracings and the main girders were found to be "moment resistant connections." It was also noted that practical changes in the dimensions of the flange thickness do not appreciably change the magnitude of the stress level. It was noted that increased web thicknesses of the girders significantly reduce the secondary stresses, if the gap length is greater than about 51 mm. However, for small gaps (less than 51 mm) the increased thickness of the web adversely affected the web stress.

Observations have disclosed that fitted transverse plate details resulted in the highest web stresses, whereas welded stiffeners produced the lowest stresses (292.3 MPa and 1.0 MPa, respectively). These stresses agreed with observed field data and accentuated the presence of stress concentrations in the detail [1,6]. The distortion of the web gap region at the fitted stiffener focused into a small gap (1.6 mm), resulting in large web stress. This stress buildup was due to the geometry of the region; these stresses do not include effects of residual stress.

The range of stress which occurred in the web gap region was determined to be caused by the relative displacement and rotation between the end of the cut short stiffener and tension flange. A comparison of the theoretical stress ranges to experimental data was then conducted. This was facilitated by using the crack growth relationship:

$$N = \int_{a_i}^{a_f} C \frac{da}{\Delta K^3} = C' \; S_r^{-3}$$

with an initial crack size of 0.762 mm. A final through thickness crack size of 28.6 mm was also selected, but this value had very little effect on the fatigue life estimates. This final crack size was selected, because it represented the length of a typical crack for the experimental data (Fig. 4) [7]. The deflections used in plotting the theoretical values were the relative displacement between the end of the cut short stiffener and the tension flange. This comparison between experimental and theoretical data is shown in Fig. 4. Figure 4 shows that

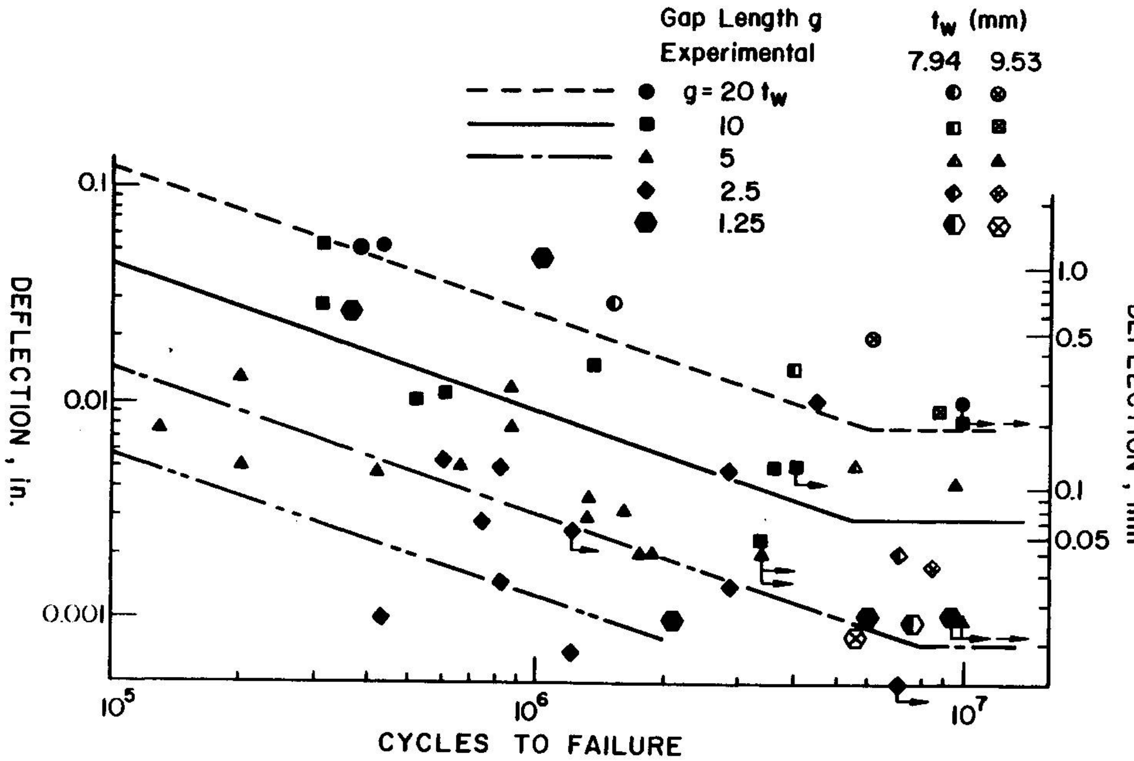


Fig. 4 Experimental and Theoretical Results of Out-of-Plane Displacement

the theoretical data followed the same trends as the experimental data. The theoretical fatigue life estimates generally overestimated the experimental fatigue lives of the details.

Parametric studies were conducted to determine the variation of the maximum web gap stress with varying gap length, web thickness and type of connection of cross-bracing to the transverse connection plate. The results of several studies can be seen in Fig. 4. They correspond to 7.94 mm web with moment connection and a 9.53 mm web with moment connection. The parametric studies permit the following observations and recommendations. The out-of-plane web stresses can be minimized by providing a gap of six to ten times the thickness of the web. They can be effectively eliminated if the transverse connection plate is welded to the tension flange. Welding the transverse connection plate to the tension flange should satisfy the fatigue restrictions of [2], Category "C", with only minor adjustments to the original girder design, because the end of the cut short stiffener is very close to the tension flange and is also classified as a Category "C" detail. If a gap of six to ten times the web thickness is selected, then web buckling under tension field action should be checked.

3. RETROFITTING FLOOR BEAM - GIRDER CONNECTIONS

In many existing structures, high out-of-plane bending and shear stresses in the short, flexible web gaps described in Section 2 for diaphragm connection plates and discussed in Refs. 1 and 3 for floor beam-girder connection plates, are induced by the out-of-plane displacement of the web. This produces fatigue cracking in the web at the weld toe regions of the web-flange welds, the ends of the connection plate-web welds, and/or cracking in the stiffener web weld which originates from the weld root. The problem of out-of-plane displacement-induced fatigue cracking is minimized or eliminated in existing structures by either minimizing or accommodating the displacement. Three techniques have been used to retrofit out-of-plane displacement induced fatigue damaged highway bridges and will be examined in detail in this section. The retrofitting techniques all deal with modification of the flexibility of the short web gap.

Upon the propagation of a fatigue crack in the short web gap, at the junction of the web and flange, and/or at the end of the transverse (vertical) connection plates, the flexibility of the gap relative to out-of-plane displacement has been increased, as the crack provides a horizontal slot. Therefore, when the displacement-induced stress has been reduced sufficiently due to the lengths of the cracks, drilling holes at the crack tips will often constitute a successful retrofit at diaphragm and floor beam connection plates. Figure 5 shows a schematic of the cracks, retrofit holes and resulting displacement.

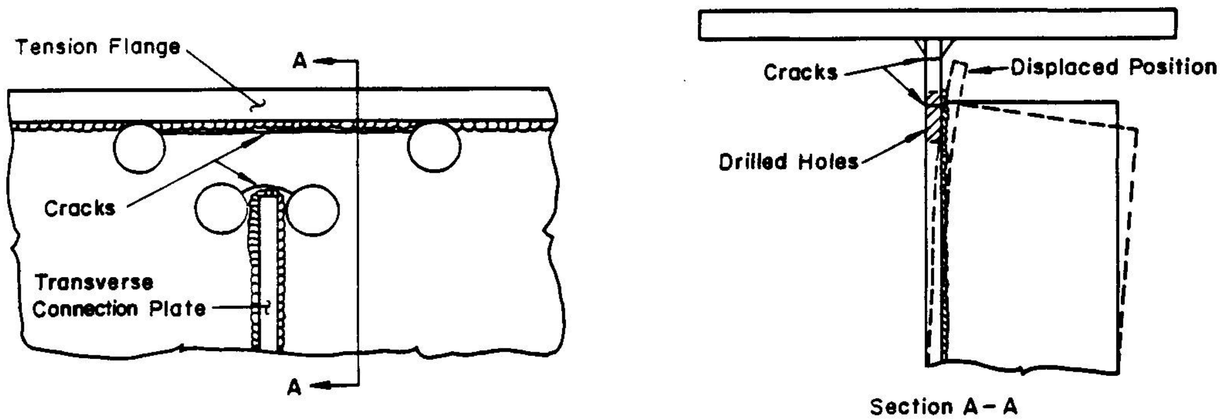
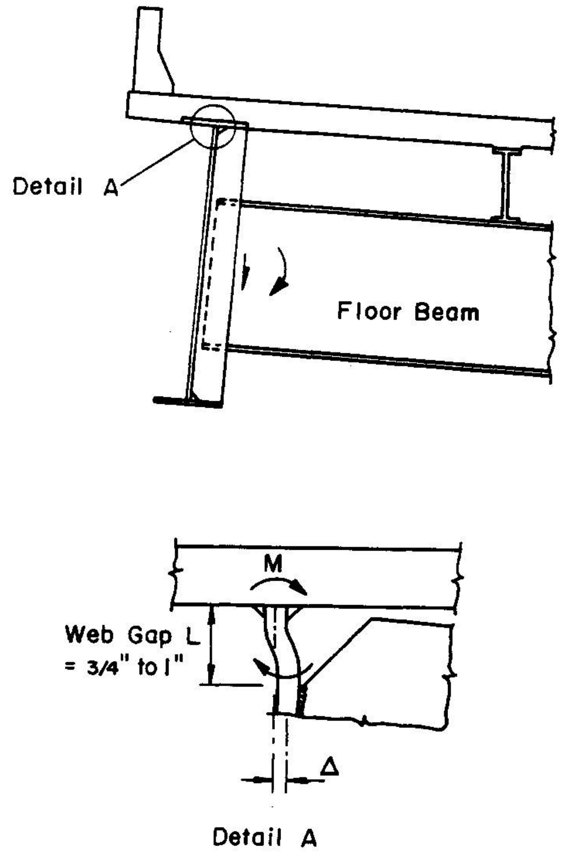


Fig. 5 Schematic of Displacement-Induced Cracks

Several bridge structures have been retrofitted by this procedure for at least eight years and have not shown any evidence of further crack propagation [3,8]. Most of these structures are located on high volume arteries and are subjected to one to two million passages of trucks each year.

In other cases, the cracking itself may not lower the cyclic stress enough, and cracking may eventually reinitiate from the drilled holes. Thus, other more involved retrofitting techniques have been used in a number of bridge structures where such conditions were thought to exist.

A more reliable method to increase the flexibility of the web gap, and thus better accommodate the out-of-plane displacement, as well as prevent the reinitiation of cracks from the drilled holes, involves the removal of a portion of the connection plate (stiffener) adjacent to the web gap, as illustrated in Fig. 6. Through the removal of this stiffening element, the web gap can be lengthened to accommodate the displacement with a significant reduction in out-of-plane bending stress [1]. This decreases the stress range throughout the web gap between the web-flange fillet weld toe and the end of the connection plate, so that cyclic stresses are below the fatigue limit. This insures that cracks do not reinitiate from the holes drilled at the crack tips.



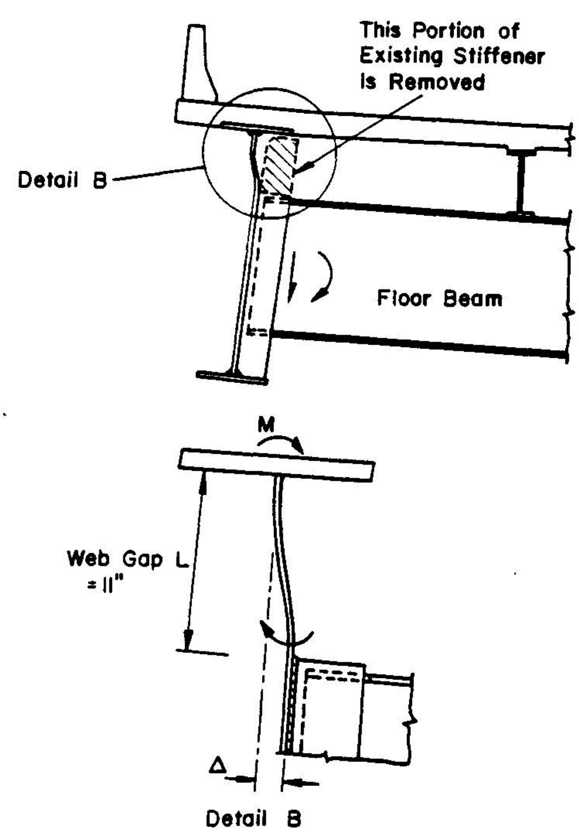


Fig. 6a Distortion of Original Web Gap Fig. 6b Distortion of Retrofitted Web Gap

In order to remove a portion of the connection plate, 50 mm diameter holes are drilled or cut by hole saw in the connection plate tangent to the girder web, as illustrated in Fig. 7. The hole is placed at the depth the stiffener is to be cut back. The connection plate is then removed by cutting vertically along the fillet weld connecting it to the web and horizontally into the hole, as also shown in Fig. 7. The fillet weld and remaining connection plate are ground away from the web, and the horizontal cut edge of the connection plate is also ground smooth. Liquid penetrant inspection identifies any cracks in the web gap or along the web-flange connection.

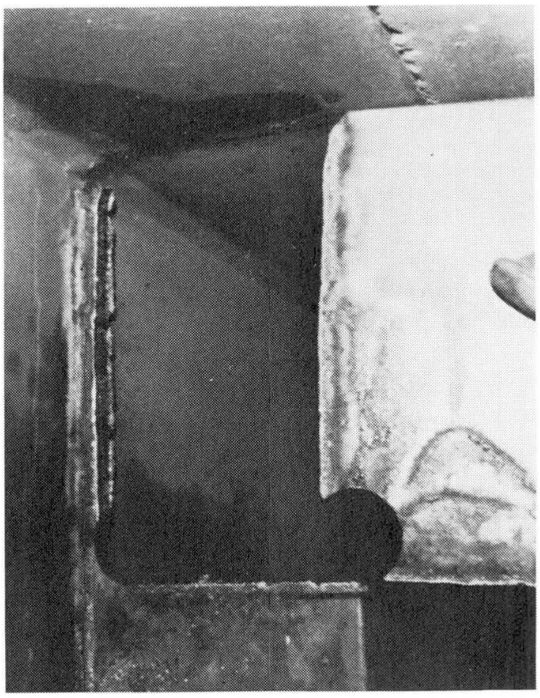


Fig. 7 Removal of a Portion of Connection Plate

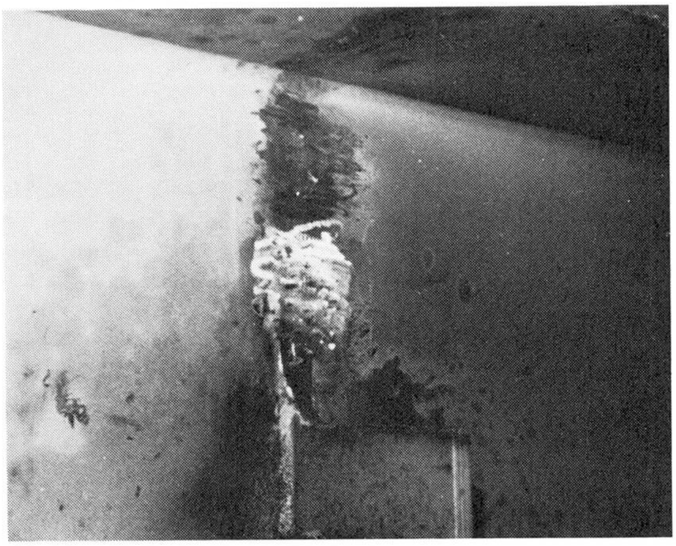


Fig. 8 Ground Surface of Web Gap

Figure 8 shows the surface of the web gap. Holes are then drilled in the web at the crack tip to prevent cracks from further propagation. Figure 9 shows a typical condition in the web gap with drilled holes isolating the cracked areas. High strength bolts are then installed in the holes as shown in Fig. 10 and tightened in order to further enhance the fatigue resistance of the drilled crack tips.

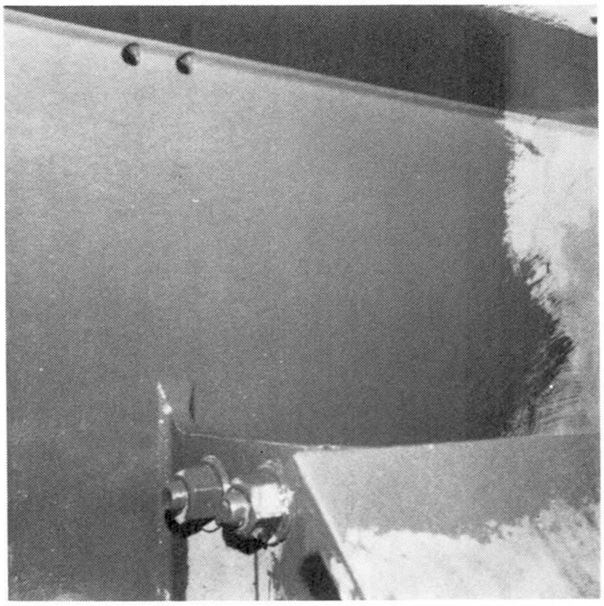


Fig. 9 Web with Drilled Holes Isolating the Cracks

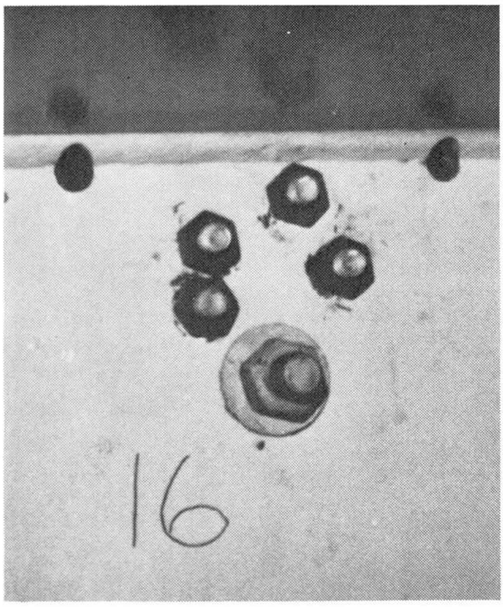


Fig. 10 Bolts Installed in Holes to Enhance Fatigue Resistance

The original short web gap has generally been lengthened to about 30.5 cm in order to accommodate the out-of-plane displacement in the web gap.

The isolated cracks can be checked to insure that fatigue cracks do not reinitiate by evaluating the parameters [9]:

$$\frac{\Delta K}{\sqrt{\rho}} < 10.5 \sqrt{\sigma_y}$$

where ΔK is the stress intensity range in the web between the edges of the retrofit holes in MPa $\sqrt{mm}$, ρ is the hole radius in mm and σ_y is the yield point of the web steel in MPa. The web bending stress due to traffic is generally low in the web, so that most of the retrofit holes are seldom greater than 24 mm.

A more extensive retrofit may be required if the transverse connection plates cannot be removed, because lateral forces are not displacement limited. This may result at piers where lateral forces are transmitted into the reactions. Under these conditions, it is desirable to prevent the movement in the web gap by attaching the transverse connection plate to the tension flange.

This can be accomplished by either welding or bolting the connection plate to the flange in order to provide a positive attachment and prevent the distortion. The alternative of welding the connection plate to the flange is generally not exercised for two reasons. First, there is concern regarding the quality of welds which can be provided under field conditions. Second, in older bridges, the weldability and fracture toughness of the flange material is unknown. Hence, welding the connection plate to the flange is generally not a practical alternative. The web gap distortion can be eliminated by providing a bolted connection between the connection plate and the flange. If the retrofit is carried out with the concrete slab in place, it is necessary to remove a rectangular portion of the slab in order to gain access to the embedded flange, as illustrated in Fig. 11. Bolt holes are then drilled in the flange and connection plate, and splice angles are inserted between the flange and the connection plate, as shown in Fig. 12. The resulting connection prevents distortion in the web gap.

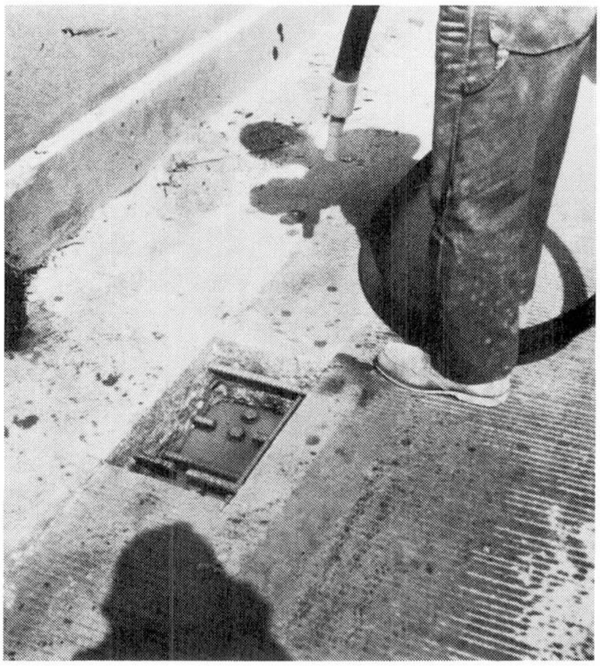


Fig. 11 Portion of Slab Removed to Gain Access to Flange

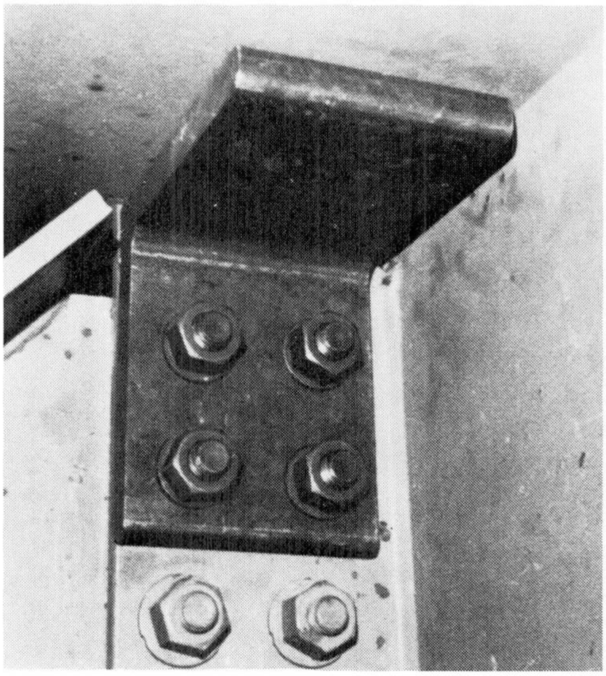


Fig. 12 Splice Angles Inserted Between Flange and Connection Plate

Bolting the connection plate to the flange is much more costly than removing a portion of the connection plate to increase flexibility. Therefore, this option should only be used when increasing web gap flexibility is not adequate to prevent further crack growth.

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