RETROFITTING FATIGUE DAMAGED BRIDGES

John W. Fisher, Alan W. Pense, Robert E. Slockbower, and Hans Hausammann, Lehigh University

This paper examines continuing laboratory and field studies on ways to retrofit fatigue damaged members. Results of a pilot field study on two bridge structures with known fatigue cracks at the ends of cover plates are reviewed. Fatigue damaged members were retrofitted by peening and gas tungsten arc remelting the weld toe. The initial retrofit is summarized and the results of subsequent inspection after 1½ years is reviewed. Also discussed is the retrofitting of several more bridges by peening weld toes on a more extensive scale. In recent years many highway and bridge structures have experienced fatigue damage from out-of-plane displacements. This has resulted in web cracking at the ends of transverse stiffeners and floor beam connection plates which were not welded to tension flanges. Cracking as a result of out-of-plane movement is reviewed and several examples of cracking in a number of bridges is discussed. Nearly all of these fatigue damaged members have been repaired and retrofitted by drilling holes in the web plate at the ends of the horizontal cracks. A series of laboratory studies have been carried out to evaluate the fatigue behavior of stiffeners due to out-of-plane displacement. After fatigue cracking from out-of-plane movement these test beams are retrofitted by drilling holes in the web plate. Subsequently the fatigue damaged girder has been cycled to confirm the adequacy of the retrofitting procedures. These results will be summarized and related to bridges with comparable conditions.

Laboratory Studies on Cover-Plated Beams

Fatigue studies on beams with welded cover plates and long attachments have demonstrated that large reductions in fatigue strength occur when fatigue crack growth occurs at the micro-sized discontinuities that exist at the weld periphery.

In addition, fatigue cracking has been observed in the field at cover-plated beam bridges that carried an unusually high volume of heavy truck traffic causing large numbers of stress cycles(1).

The formation of these cracks showed the desirability of examining methods for improving (upgrading) the fatigue strength of welded joints without changing the design details. In addition, methods are needed for arresting the progress of fatigue damage that occurs at the weld toes of severe notch-producing details where the probability of failure is greatest.

An experimental program was carried out on sixty steel cover-plated beams in either the as-welded or precracked condition, to determine the fatigue strength of these details when treated by techniques intended to extend their fatigue life. Three of the most successful methods reported in the literature for as-welded details were utilized(2,3,4,5). They included: (1) grinding the weld toe to remove the slag intrusions and reduce the stress concentration, (2) air hammer peening the weld toe to introduce compression residual stresses, and (3) remelting the weld toe using the Gas Tungsten Arc process.

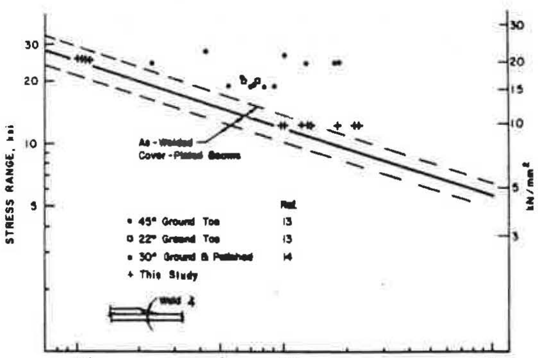


Figure 1. Effect of grinding weld toe on fatigue strength

Grinding the weld toe with a burr to provide a smooth transition and minimize the size of the initial discontinuities was the least reliable method. Some improvement was noted at the lower stress range levels as illustrated in Figure 1, but none at all at the highest level of stress range. Similar results were obtained in earlier studies on as-welded details which indicated that erratic results could be expected.

Peening the weld toe was observed to be most effective when the minimum stress was low. This was true for as-welded and precracked details. This appeared to be directly related to the effectiveness of the compressive residual stresses introduced by the peening process. When peening was carried out on unloaded beams, the application of a high minimum stress and/or high stress range decreased the effectiveness of the residual compressive stresses that were introduced. Several tests were carried out on beams which were peened under a simulated dead load condition. Under these conditions about the same improvement was noted at both high [68.9 MPa (10 ksi)] and low [13.8 MPa (2 ksi)] minimum stress levels and at higher stress range levels as well.

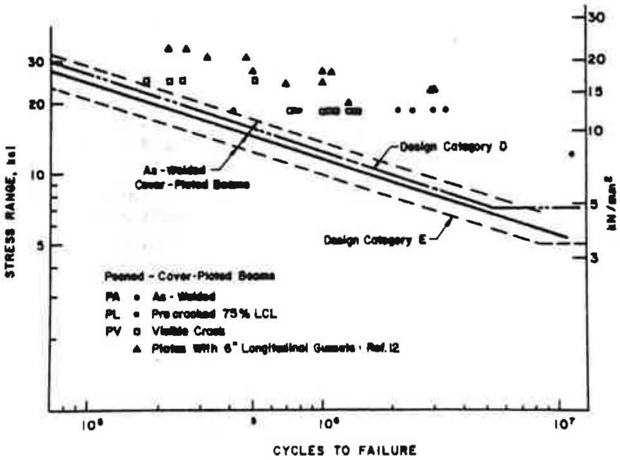


Figure 2. Effect of Peening on Fatigue Strength

The results of all beams with peened details that were tested under a low minimum stress level [13.8 MPa (2 ksi)] or that were peened under their minimum load, are summarized in Figure 2. Those details that were peened in the absence of dead load are not plotted in Figure 2. When the 68.9 MPa (10 ksi) minimum stress was applied to these beams it eliminated most of the beneficial effects of the peening treatment.

The test points designated as PA were as-welded beams treated prior to any fatigue testing. The test points designated as PL were for beams that were first precycled to 75% of the lower confidence limit of as-welded-untreated beams. After precycling these beams were then treated. They had fatigue cracks of various sizes. Two series of precracked beams (PL and PV) and one series of as-welded beams (PA) were tested to determine the effectiveness of peening. Cracks as large as 19 mm (0.75 in.) length and between 1.3 and 3.8 mm (0.05 and 0.15 in.) deep were observed prior to peening. After peening the precycled cracks were no longer visible. Inspection of the fracture surfaces indicate that movement occurred between the crack surfaces to a depth of 3.8 mm (0.15 in.). The precracked details usually failed from continued crack growth from the original crack, but at a slower rate. In a few cases failure was observed from the weld root. It is readily apparent that substantial increases in life were achieved for as-welded and precracked beams after peening, when peening was applied in the presence of dead load. The fatigue strength was increased by at least one design category(6)

Also shown in Figure 2 are test results on small plate specimens with 152 mm (6 in.) longitudinal gussets welded to their surface. These tests were reported by Gurney and were made on as-welded specimens(5). The studies on welded attachments reported in NCHRP Report 147 have demonstrated that the attachment length has a significant effect upon fatigue strength. Hence, these 152 mm (6 in.) longitudinal gusset plates were expected to exhibit slightly more life than those provided by cover-plated beams. This was confirmed by the test data. All of the peened plate specimens fell near the upper limit provided by peened cover-plated beams. This suggests that other details can be expected to exhibit a similar increase in fatigue strength when subjected to peening at the weld toe.

Figure 3. Lap-type defect in a peened weld toe

Transverse sections through several peened weld toes, revealed numerous lap-type defects which were the result of extensive surface deformation. An example of this deformation is shown in Figure 3. The depth of these laps was in the order of 0.05 mm (0.002 in.) to 0.25 mm (0.010 in.) which was approximately the same depth as the original slag intrusions. These defects are believed to be typical of the whole weld toe since they were found on all transverse sections.

Gas tungsten arc remelting at the weld toe termination was observed to provide the most reliable and consistent method of improving the fatigue strength in the as-welded or previously precracked condition. In a few instances the initial crack was not removed. Application of the gas tungsten arc remelt process did not succeed in completely fusing the fatigue crack in these specimens and no improvement was observed. These cases were encountered before suitable procedures were developed to obtain a desired depth of penetration.

The results of all three test series are summarized in Figure 4. The test points plotted as as-welded beams were treated prior to applying cyclic loading. The test points identified as precracked to 75% LCL were all precycled to 75% of the lower confidence limit for untreated details. At that time the "fatigue-damaged" detail was treated with the gas tungsten arc remelt. In some cases no visible crack existed. Those points indicated as visible precracked all had clearly visible cracks prior to treatment. Except for those failures in precracked beams that occurred because of failure to incorporate the complete crack into the gas tungsten arc remelt (see Figure 4), approximately the same increases in life were achieved by all specimens. None of the test series exhibited an influence of minimum stress. Stress range was observed to account for nearly all of the variation in fatigue strength.

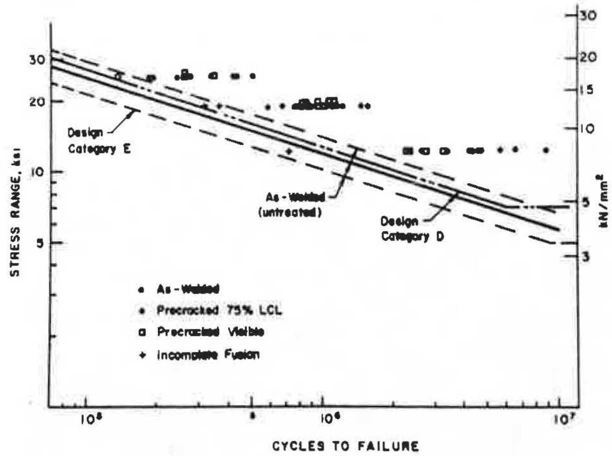


Figure 4. Effect of gas tungsten arc remelt on fatigue strengh

Data available from other sources is primarily on small plate specimens with transverse gussets that provide a non-load carrying joint(3,4). The studies on NCHRP Project 12-7 have indicated that this type of specimen provides fatigue behavior that is similar to stiffener type details(6). No data was available on cover-plated beam details that had been subjected to gas tungsten arc remelting at fillet weld toes.

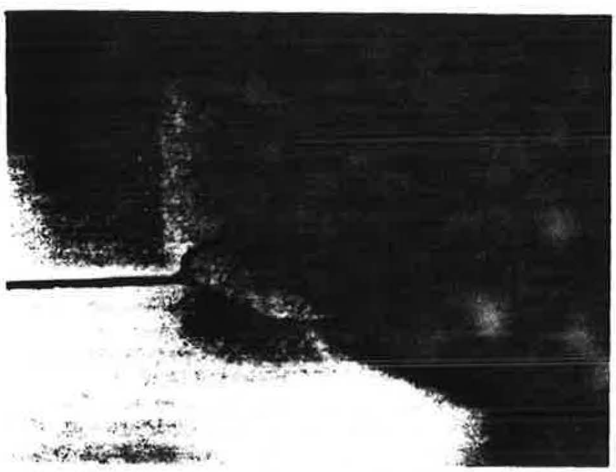


Figure 5. Cross-section of cover plate end weld

An etched cross-section of the transverse end weld is shown in Figure 5. The remelt penetration is visually evident at the weld toe. It was possible to provide up to 5 mm (0.2 in.) penetration in the gas tungsten arc remelt. Figure 5 also demonstrates the reason that an upper bound to fatigue strength was observed for welded cover-plated beams. Improvements in the condition at the weld toe could not affect the growth of cracks from the weld root. Most of the details treated by gas tungsten arc remelt passes had their life governed by failure from the weld root. Treatment at the weld toe forced the failure to the less severe weld root and resulted in greater life. Tests currently underway on full size beams have yielded comparable behavior.

Fatigue Damage in Cover-Plated Beam Bridges

In October-November 1970, during cleaning and repainting of the Yellow Mill Pond Bridge, one of the cover-plated steel beams on the eastbound bridge on span 11 was found to have a large crack(1). The crack had developed at the west end of the primary cover plate on Beam 4. It had grown from the toe of the cover plate transverse fillet weld into the tension flange and up 406 mm (16 in.) into the web.

A visual inspection (10X magnification) showed that Beams 3 and 5 in span 11 of the eastbound roadway which were adjacent to the casualty girder had cracks along the cover plate end. These cracks were subsequently verified by ultrasonic testing and a depth of penetration equal to 16 mm (0.625 in.) was measured. They were about half the flange thickness in depth and were found to have a semielliptical shape. An indication of possible fatigue cracking was also observed at five other details on span 10 and two on span 11. No ultrasonic confirmation could be obtained at the other possible crack locations.

In December 1970, after the detailed inspection, a section of the fractured girder was removed and all three damaged girders were subsequently repaired with bolted web and flange splices. The section of fractured girder was taken to Lehigh University for the purpose of investigating the fracture surface and determining the material characterization.

In November 1973, the east end of Beams 2 and 3 in the eastbound roadway of span 10 were inspected again by J. W. Fisher for fatigue damage. This was the first inspection at Beam 2. An indication of possible cracking was observed at Beam 3 in 1970. Cracks were detected visually in both girders at the toe of the primary cover plate transverse weld. A magnetic crack definer(7) indicated that the crack in Beam 2 was approximately 9.5 mm (0.375 in.) deep at one point. The magnetic crack definer could not verify the presence of a crack in Beam 3.

In June 1976, forty cover plate details in the east and westbound span 10 bridges were inspected for fatigue cracking using visual, magnetic particle, dye penetrant, and ultrasonic procedures prior to retrofitting these girders during Phase I of NCHRP Project 12-15(2). Twenty-two of these details were found to be cracked by visual inspection. The smallest visual crack indication was 6.4 mm (0.25 in.) long. Fifteen of these cracks had propagated deep enough to be detected by ultrasonic inspection.

To inspect for cracks it was first necessary to blast clean and remove paint, dirt and oxide which had accumulated in the weld toe region. The visual (10X magnification), magnetic particle and dye penetrant inspection provided data regarding the length of the surface cracks. The magnetic particle inspection was discontinued after examining several cover plates due to difficulty in working with the probe in the overhand position.

The ultrasonic inspection provided data regarding both the length and depth of cracks. Cracks at the weld toe smaller than approximately 2.5 mm (0.1 in.) deep could not be reliably detected by the ultrasonic probe. The deepest crack depth indications of 13 mm (0.5 in.) were found at the west end of the eastbound span 10 bridge in Beams 3 and 7. Comparisons of estimated crack depths from ultrasonic inspection and actual measured crack depths after a fracture surface was exposed indicate that deviations of 1.6 mm (± 0.06 in.) are possible.

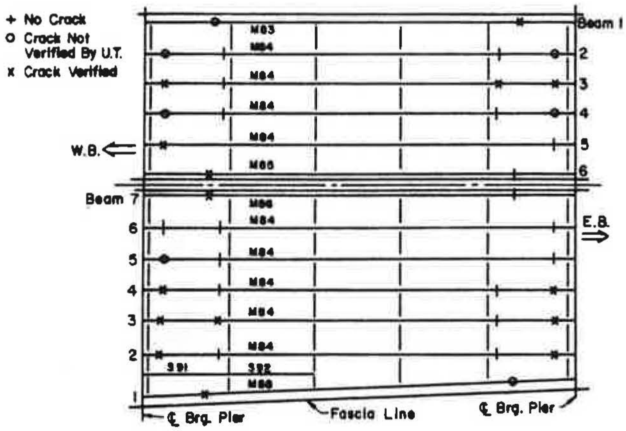


Figure 6. Plan of inspected details in span 10 Yellow Mill Pond Bridge

Figure 6 shows the approximate location of the details which were inspected in span 10 and summarizes the findings. Nine details in span 11 were also visually inspected. Indications of cracking were found at seven details. Very large cracks were observed at the east end of Beam 5 of the eastbound bridge and Beam 4 of the westbound bridge - span 11.

In November 1976, a brief inspection was made by J. W. Fisher at span 13. Four large cracks were detected without removing the paint. These cracks were first observed with field glasses from the ground. It is believed that these cracks must be approximately 150 to 250 mm (6 to 10 in.) long and about 13 mm (0.5 in.) deep for the crack to break the paint film at the weld toe. This condition is probably also related to the ambient temperature. Decreasing temperatures cause a more brittle paint coat and increase the likelihood of the paint to crack.

In September 1977, a brief inspection was made by J. W. Fisher and A. W. Pense and three additional beams in the eastbound structure had cracks at the ends of secondary cover plates. These cracks were observed to be about 6 mm (0.25 in.) long and occurred at several points along the transverse weld toes.

Retrofitting Fatigue Damaged Bridge Members in Span 10 - Yellow Mill Pond

Peening and gas tungsten arc remelting procedures were used to retrofit the cover-plated beams in span 10 of the Yellow Mill Pond Bridge which were found to have fatigue damage.

Grinding of the weld toe to reduce the size of the initial discontinuities and severity of the stress concentration had shown little or no improvement of fatigue strength of fatigue damaged members. Hence no attempt was made to employ this procedure at Yellow Mill Pond.

Peening of the weld toe introduces compressive residual stresses. The weld toe was mechanically air-hammer peened until it was plastically deformed.

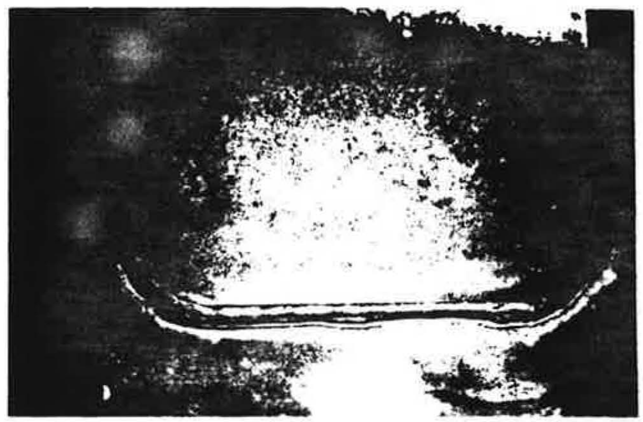


Figure 7. Peened weld toe on Yellow Mill Pond Road

Peening was performed with a small pneumatic air hammer operated at 0.17 N/mm^2 (25 psi) air pressure. The end of the peening tool was radiused with a 19 mm (3/4 in.) radius about one axis and a 3 mm (1/8 in.) radius about the other axis. All sharp edges were ground smooth. Several minutes were required to peen the weld toe. Peening was continued until the weld toe became smooth. A peened weld toe at Yellow Mill Pond is shown in Figure 7. The depth of indentation due to peening was approximately 0.8 mm (0.03 in.).

The Gas Tungsten Arc Process (GTA) removes the nonmetallic instrusions at the weld toe and reduces the magnitude of the stress concentration by smoothing the weld termination. The tungsten electrode was manually moved along the toe of the fillet weld. This melted a small amount of the fillet weld and base metal. Provided that the cracks are not too deep, the metal around the cracks can be sufficiently melted so that after solidification, the cracks will have been removed.

The welding equipment used was a 200 amp DC power source with drooping V-I characteristics. A high frequency source was used to start the arc. The electrode was 4.0 mm (0.156 in.) in diameter with a 4.8 mm (0.188 in.) stick out. The composition of the electrode was 2 percent thoriated tungsten. A Linde HW-18 water-cooled torch was used. The entire welding unit was mounted on a Bernard portable carriage which also contained the water supply and a recirculating pump to cool the torch. The portable carriage was mounted on the rear of a truck with the portable gasoline power supply. A 15 m (50 ft.) line from the welding unit to the torch permitted the welded access to the girder.

A series of preliminary tests were conducted in Reference 6 to find the effect of welding variables on GTA remelt penetration. The results of this study indicate that maximum penetration is obtained by the use of helium shielding gas and a cathode vertex angle between 30 and 60 degrees.

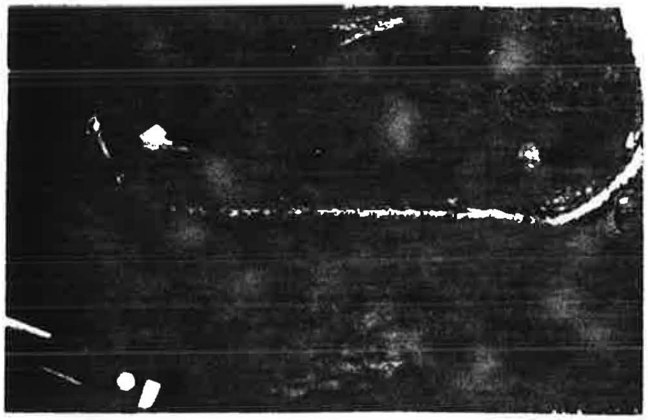


Figure 8. Transverse fillet weld after gas tungsten arc remelt

All retrofit welds on span 10 were performed in the overhead position. The areas to be welded were sandblasted to remove the mill scale that promotes undercutting. A helium and argon mixture shielding gas and a cathode vertex angle between 30 and 60 degrees were used as the mixture provided about the same penetration as helium alone. Travel speed was approximately 1.3 mm/sec. (3 in/min.). The retrofit weld was started on the longitudinal weld toe and continued along the transverse weld toe. The weld finally terminated at the opposite longitudinal weld toe. Intermediate terminations were made at approximately 100 mm (4 in.) intervals because of the duty cycle of the portable welding unit. Each of these terminations were carried up to the weld face to prevent cratering at the weld toe. Figure 8 shows a transverse fillet weld after the gas tungsten arc retrofit.

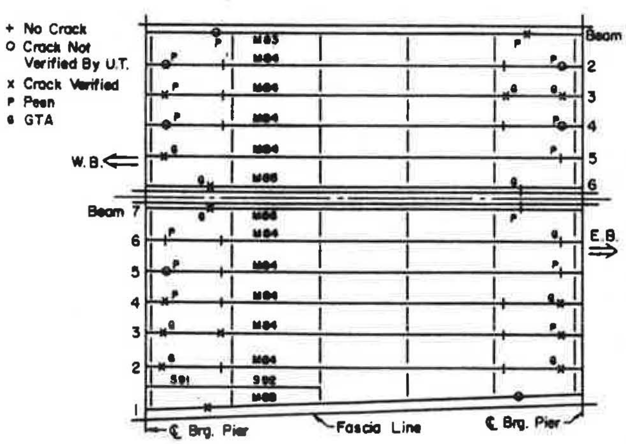


Figure 9. Repair methods on span 10, Yellow Mill Pond Bridge

Twenty-five of the cover plate details in span 10 were repaired after being inspected. Fourteen were peened and eleven were gas tungsten arc remelted. Figure 9 summarizes the type of repair which was made at each cover plate weld toe.

Seven of the remelted details which had cracks detectable by ultrasonic examination were reinspected after the repairs were completed. The east primary details on Beams 2 and 4 (eastbound bridge) both produced a spot indication at a depth of 3.2 mm (0.125 in.). The ultrasonic examination of the west primary details on Beams 3 and 7 (eastbound bridge) which had cracks about 13 mm (0.5 in.) deep, indicated a large embedded crack. The remelt at these details did not change the crack depth. The depth of remelt penetration was approximately 6.4 mm (0.25 in.) (see Figure 10). These cracks were purposely treated without gouging and rewelding by conventional means in order to evaluate the effectiveness of the treated details. The length of time required for the crack to penetrate back through the weldment could be compared with theoretical estimates of life extension. No crack indications were found at the west primary detail of Beam 2 (eastbound bridge) or at the east primary and secondary details of Beam 3 (westbound bridge).

Residual Fatigue Life after Retrofitting

Since the field repair of the Yellow Mill Pond Bridge members was only recently completed, the effectiveness of this repair must be judged on the basis of available laboratory studies on similar members. Fortunately both experimental data and analytical techniques exist to make this assessment. Peening was most effective in the laboratory when the initial cracks were very small. For this reason, peening was selected for retrofitting all beams where ultrasonic inspection was unable to confirm a visual indication of cracking or where neither inspection technique detected cracking. All cracks greater than 3.2 mm (0.125 in.) deep were gas tungsten arc remelted.

No cracks were indicated by ultrasonic inspection at ten of the cover plate ends which were peened in span 10. Four cover plates which were peened had a maximum depth indication of approximately 3.2 mm (0.125 in.). Therefore, the effectiveness of peening at Yellow Mill Pond should be comparable to the results plotted in Figure 2.

The laboratory studies on fatigue damaged details that were retrofitted by peening indicated a greater tendency for improvement at the lowest level of stress range tested [82.7 MPa (12 ksi)]. The details yielded fatigue lives up to 10^7 cycles. Since the stress range experienced at Yellow Mill Pond seldom exceeds 41.4 MPa (6 ksi), this procedure should be even more successful in prolonging life. The lower level of applied stress range will make the peened detail more effective because the induced compressive residual stresses at the crack tip are not likely to be overcome. As a result the details should be subjected to cyclic stresses that are well below the effective crack growth threshold. Tests currently underway have verified the expected behavior. Beams subjected to 41.4 MPa (6 ksi) or 55.2 MPa (8 ksi) stress range have been retrofitted after developing small fatigue cracks at the weld toe. These details have been retrofitted by peening and have not experienced any further crack growth after being subjected to 30 million stress cycles.

The increased fatigue strength developed by the retrofitted (gas tungsten arc remelted) precracked beams also suggested that substantial increases in fatigue strength could be expected at the lower stress ranges to which the Yellow Mill Pond Bridge beams were subjected. The crack growth threshold of Category D appears to be about 58.4 MPa (7 ksi), which is substantially above the stress ranges experienced at Yellow Mill Pond (see Figure 4). Hence, retrofitting by the gas tungsten arc remelt procedure should eliminate the possibility of subsequent cracking.

The probability of a root failure occurring is dependent on the relative size of the weld with respect to the thickness of the cover plate. As the ratio between weld throat width and cover plate thickness increases, the probability of root failure decreases. For the W14x30 cover-plated beams the ratio of throat width to cover plate thickness is 0.31. This ratio at the primary and secondary cover plate details of the interior beams (W36x230) at Yellow Mill Pond is 0.28 and 0.32, respectively. Therefore, comparable results should result at Yellow Mill Pond. The scatter in the fatigue lives of remelted details is due primarily to the effectiveness of melting the material surrounding the fatigue cracks. The maximum crack depth closed in the remelting test beams was approximately 3.8 mm (0.15 in.). Ultrasonic inspection of the large fatigue cracks at the west end of Beams 3 and 7 (eastbound roadway) after remelting indicate that the depth of penetration was approximately 6.4 mm (0.25 in.). A sample plate was cleaned and gas tungsten arc remelted at the Yellow Mill Pond. The specimen was then sectioned, polished and etched. The depth of penetration was measured between 3.5 mm (0.14 in.) and 5.8 mm (0.23 in.).

After the remelt retrofit was completed, the details that had provided indications of cracking were ultrasonically inspected. No indications of residual cracks were found at the primary or secondary details of Beam 3 (east end, westbound roadway) and at the primary detail of Beam 2 (west end, eastbound roadway). This indicated that the gas tungsten arc remelt procedure had effectively eliminated the small fatigue cracks that were detected at those details.

The increased fatigue life as a result of peening or remelting should increase the crack growth threshold stress range, $\Delta\sigma_{TH}$. If the peening operation is capable of embedding the crack initiation sites in a compressive residual stress field a significant increase in the threshold stress range will be observed. The gas tungsten arc remelt procedure reduces the stress concentration by smoothing the transition at the weld toe and also minimizes the embedded discontinuities and fatigue cracks. Therefore, $\Delta\sigma_{TH}$ will also be increased.

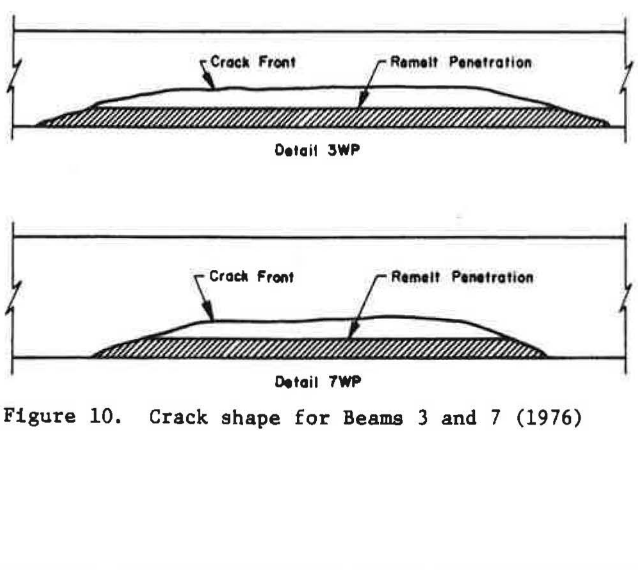
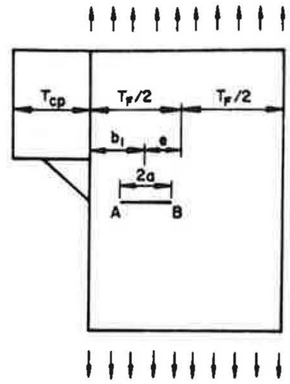


Figure 10. Crack shape for Beams 3 and 7 (1976)



$$K_{I,A} = \sigma\sqrt{\pi a}\ F_A\left[\frac{a}{T_F/2 - e},\ \frac{e}{T_F/2}\right] F_G\left[\frac{b_1 - a}{T_F},\ SCF\right]$$

$$K_{I,B} = \sigma\sqrt{\pi a}\ F_B\left[\frac{a}{T_F/2 - e},\ \frac{e}{T_F/2}\right] F_G\left[\frac{b_1 + a}{T_F},\ SCF\right]$$

F_A, F_B from Ref. 8

$$F_G = \frac{SCF}{\left[1 + \frac{1}{0.1473}\left(\frac{a}{T_f}\right)\right]^{0.4343}}$$

Figure 11. Stress intensity model for embedded cracks

The shape of the large embedded cracks at the west primary detail of Beam 3 (eastbound bridge) and at the west detail of Beam 7 (eastbound bridge) are shown in Figure 10. The stress intensity model for these embedded cracks is shown in Figure 11. This approximate model combines the solution for an eccentric crack(8) with the stress gradient correction factor, F_G, defined in Ref. 9. Utilizing this model and the crack growth rate $da/dN = 3.8\times10^{-9}\ \Delta K^3$ (ΔK in units of $MPa\sqrt{mm}$, da/dN in units of mm/cycle), the number of cycles necessary for the crack to propagate through the retrofit weld toward the weld toe was estimated. It was assumed that when the embedded crack penetrated the exterior flange face it would quickly become an elliptical surface crack with the major semidiameter axis being defined by the crack shape ratio prior to retrofitting.

For this study the retrofit weld penetration was assumed to be 4.8 mm (3/16 in.). The estimated number of cycles necessary to propagate the embedded cracks through the retrofit weld at a stress range of 13.1 MPa (1.9 ksi) for Beams 3 and 7 were 7.0 and 6.7 million cycles, respectively. The elliptical surface cracks for both beams were approximately 13.5 mm (0.53 in.) deep at the beginning of the final stage of crack growth. An additional 1.0 and 2.7 million cycles would be necessary for the cracks to grow through the flange thickness for Beams 3 and 7, respectively.

The stress intensity model for the growth of embedded cracks probably overestimates the fatigue life since it does not account for crack growth which is occurring simultaneously from the weld toe. Nevertheless, substantial improvement in fatigue strength can be expected even if the entire crack has not been completely remelted, if the crack initiation sites along the weld toe have been effectively reduced.

Ultrasonic inspection of the primary detail of Beam 2 (east end, eastbound roadway) and the secondary detail of Beam 3 (east end, eastbound roadway) produced a spot indication at a depth of 3.2 mm

(0.12 in.). These embedded discontinuities may be below the crack growth threshold. Since their size and shape is nearly impossible to estimate, an exact evaluation is not possible.

The beams with the large embedded cracks were inspected again during September 1977 and showed no evidence of further crack growth. The tungsten inert gas remelt pass was still intact. Approximately 3 million stress range cycles had been experienced since the original treatment.

Retrofitting Web Cracks at the Ends of Stiffener and Floor Beam Connection Plates

Web cracking has been observed in welded girders at the ends of transverse stiffeners and floor beam connection plates(10). These cracks are caused by out-of-plane movement at the web. Out-of-plane movement as shown schematically in Figure 12 is introduced from the end rotation and displacement of floor beams which frame into the web of the main longitudinal built-up girders and by horizontal deflections of bracing members connected with stiffeners. It can also be introduced by bending during handling and shipping. Out-of-plane movement causes large bending stresses in the short gap at the end of the stiffeners.

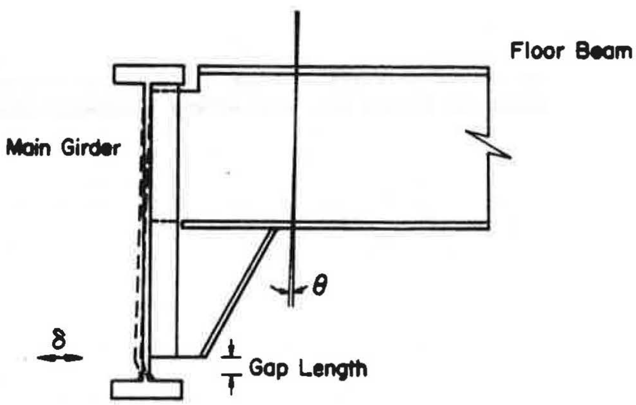


Figure 12. Schematic of deformation at end of floor beam connection plate

At the present time fatigue tests are being conducted at Lehigh University to evaluate the fatigue strength of comparable details. In these tests web cracking is introduced by simulating the out-of-plane moment that occurs at interior floor beams where the tension flange is restrained. The experimental test program includes various gap (distance between the end of the transverse stiffener and the flange) and the magnitude of out-of-plane deflection. Gap lengths equal to 2.5, 5, 10 and 20 times the web thickness are being examined when the out-of-plane deflection varies between 0.13 mm (0.005 in.) and 2.5 mm (0.1 in.).

The tests have shown that the specimens crack either immediately at the end of the transverse stiffener or at the end of the stiffener weld. No cracking occurred at the longitudinal weld between the web and the flange. In Figure 13 the preliminary results for several gaps under different deflections are shown.

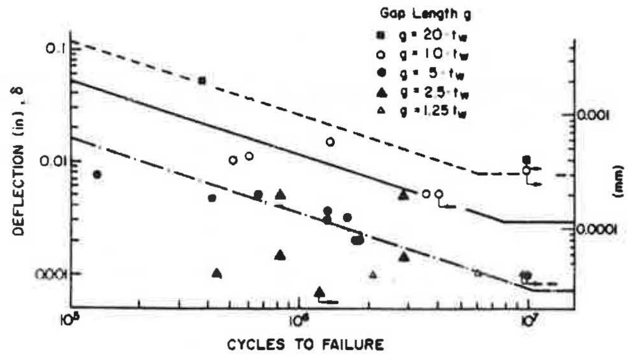


Figure 13. Experimental relationship between web gap length and out-of-plane displacement

The experimental data suggest that increasing the gap by a factor of two results in an order of magnitude increase in the cyclic life. If the stress range at the gap were only proportional to the deflection, i.e.

$$\sigma_r \propto A\,\delta\,\frac{EI}{g^2} \tag{1}$$

Then the cyclic life could be estimated from the relationship

$$N = \frac{1}{C}\int_{a_i}^{a_f} \frac{da}{\Delta k^3} \tag{2}$$

Since $\Delta K \cong \sigma_r\, F\, \sqrt{\pi a}$, this would result in

$$\Delta K \propto \bar{A}\,\frac{\delta}{g^2} \tag{3}$$

Substitution into Equation 2 yields

$$N \propto \bar{\bar{A}}\,\frac{g^6}{\delta^3}\left(\frac{1}{\sqrt{a_i}} - \frac{1}{\sqrt{a_f}}\right) = B\,\frac{g^6}{\delta^3} \tag{4}$$

The ratio of life for a given level of deflection can be estimated for this lower bound condition as

$$\frac{N_{g_1}}{N_{g_2}} = \frac{Bg_1^6/\delta^3}{Bg_2^6/\delta^3} = \frac{g_1^6}{g_2^6} \tag{5}$$

Considering the ratio of life for a gap length of 5 t_w and 10 t_w results in

$$\frac{N_{g_1}}{N_{g_2}} = \frac{(5\,t_w)^6}{(10\,t_w)^6} = 0.015 \tag{6}$$

The test data indicate that a ratio of 0.15 exists. This difference appears to be due to rotation of the flange and variance from the assumed model. This evaluation has indicated that increasing the gap does not increase resistance to fatigue cracking at as fast a rate as implied by Equation 1. However, the results also suggest that careful consideration should be given to details that will result in out-of-plane movement in gap regions.

Figure 14. Holes drilled at ends of crack

After each transverse stiffener on the test girder was fatigue cracked by the out-of-plane movement, holes were drilled at the crack tip. Figure 14 shows a typical retrofitted stiffener end. After retrofitting by drilling holes, the girder was subsequently subjected to cyclic loading at a stress range corresponding to category C [i.e. 96.5 MPa (14 ksi) at the stiffener end].

All cracked stiffener ends sustained cyclic loading until the test exceeded the lower confidence limit corresponding to Category C. No cracks were detected in the drilled holes perpendicular to the bending stress in the girder. Fortunately the cracks from out-of-plane movement are mainly parallel to the cyclic stresses. Hence the drilled holes were very effective in preventing further cracking and provided a detail comparable to the fatigue design condition.

Summary and Conclusions

Extensive laboratory experimental work on welded details has demonstrated that fatigue damaged details can be retrofitted and their fatigue life extended. Three repair or improvement methods were studied experimentally and observed to be effective to varying degrees in extending the fatigue life of welded details. Grinding was not as effective as peening under dead load and the gas tungsten arc remelt pass.

Peening was observed to produce good results with both uncracked as-welded details and fatigue damaged details with surface cracks less than 3 mm (1/8 in.) deep. The application of a high minimum stress after peening caused a reduction in the effectiveness of peening as it caused a decrease in the compression residual stress. As a result of the laboratory results on cover-plated beams which had experienced fatigue damage, peening was used to retrofit fatigue damaged bridge details that showed no significant crack growth.

The gas tungsten arc remelt pass was the most effective method examined in the laboratory and was also effective in repairing fatigue damaged details with surface cracks less than 5 mm (3/16 in.) deep. The procedure was used to retrofit bridge beams with cracks up to 12 mm (1/2 in.) deep. This did not permit the fusion of all of the crack surface. However, it was predicted that the embedded crack would provide several years additional life. Bridge beams with cracks at cover-plated weld toes that were 5 mm (3/16 in.) or less deep could be retrofitted and the fatigue crack removed by the remelt procedure. This was confirmed in the field.

Acknowledgments

This paper is based on research conducted under the National Cooperative Highway Research Program under Projects 12-15(1) and 12-15(2), Transportation Research Board, National Academy of Sciences and sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration.

The study was conducted at Fritz Engineering Laboratory, Lehigh University, Bethlehem, PA. Appreciation is expressed to the staff at Fritz Labortory, in particular, Robert Dales and Hugh Sutherland for experimental work, Richard Sopko for photography, John Gera for drafting and Ruth Grimes for preparation of the manuscript.

References

1. D. G. Bowers. Loading History Span 10 Yellow Mill Pond Bridge I-95, Bridgeport, Connecticut. Highway Research Record No. 428, TRB, 1973.
2. E. E. Brine et al. A Note on the Effect of Shot Peening on the Fatigue Properties of Plain Plate and Welded Joints in 18% Ni Maraging Steel and Al-Zn-Mg Alloy. Technical Note No. 5/68, Military Engineering Experimental Establishment, Christchurch, New Zealand, May 1968.
3. D. Millington, TIG Dressing to Improve Fatigue Properties in Welded High-Strength Steels. Metal Construction and British Welding Journal, Vol. 5, No. 4, April 1973.
4. F. Watkinson et al. The Fatigue Strength of Welded Joints and Methods for its Improvement. Proceedings of Conference on Fatigue of Welded Structures, The Welding Institute, 1971, pp. 97-113.
5. T. R. Gurney. Effect of Peening and Grinding on the Fatigue Strength of Fillet Welded Joints. British Welding Journal, Vol. 15, No. 12, 1968.
6. J. W. Fisher et al. Improving Fatigue Strength and Repairing Fatigue Damage. Fritz Engineering Laboratory Report No. 385.3, Lehigh University, Bethlehem, PA, December 1974.
7. A. Leone et al. A New System for Inspecting Steel Bridges for Fatigue Cracks. Public Roads, Vol. 37, No. 8, March 1974.
8. H. Tada et al. The Stress Analysis of Cracks Handbook. Del Research Corporation, Hellertown, PA, 1973.
9. N. Zettlemoyer. Stress Concentration and Fatigue of Welded Details. Ph.D. Dissertation, Lehigh University, Bethlehem, PA, 1976.
10. J. W. Fisher. Bridge Fatigue Guide - Design and Details. AISC, 1977.