Fatigue Strength of Steel Members with Welded Details

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Early studies on the fatigue of welded steel structures and components revealed the influencing factors such as stress magnitudes and geometry of structural details.[1,2] Approximate design relationships were developed in the 1940's and, overall, very few problems occurred with steel bridges and structures as a result of fatigue. The fatigue cracking of coverplated steel beam bridges of the AASHO Road Test[3] in the early 60's led to further examinations of the controlling factors and more extensive studies.

A substantial amount of experimental data has been developed on steel beams since 1967 from a study sponsored by the National Cooperative Highway Research Program (Project 12-7) and carried out at Fritz Laboratory, Lehigh University. These studies have shown that the most important factors which govern the fatigue strength are the stress range and the type of details.[4,5] For each type of welded structural detail, regardless of the strength of the steel, the applied stress range was found to cause propagation of fatigue cracks from initial discontinuities at the details. The fatigue life of a detail was nearly exhausted when the crack had propagated through the thickness of the flange plate. These findings were observed to be applicable to rolled beams, welded beams, beams with groove-welded flange splices, coverplated beams, and to beams with welded stiffeners and attachments.[4,5] Com-

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prehensive specifications based on stress range alone have been developed for bridge and building structures[6-8] utilizing these beam test data and other available information in the literature.[9]

This paper reviews very briefly the governing parameter of stress range, the experimental results on welded stiffeners and attachments, the phenomenon of crack propagation, and the data basis for the specifications. In addition, some secondary considerations which have caused fatigue cracking are examined and possible ways of retrofitting cracked structural details are suggested.

UPPER AND LOWER BOUND STRENGTH

Results from testing 374 beam specimens indicated that stress range was the controlling factor influencing the fatigue strength for each structural detail.[4] The stress range-cycle life relationships shown in Figs. 1 and 2 indicate that when the applied nominal stress range at the detail is used, it accounts for nearly all the variations in cycle life
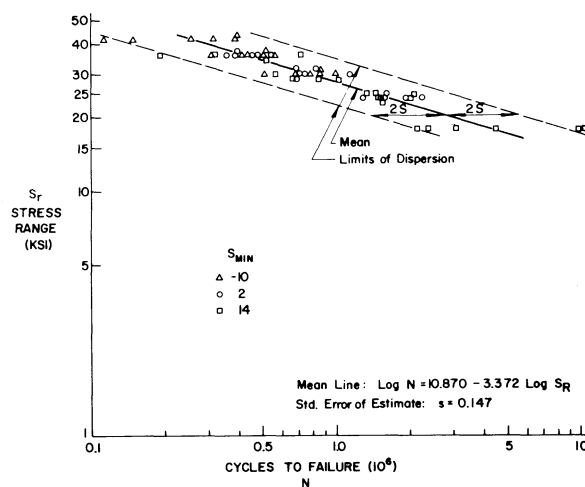

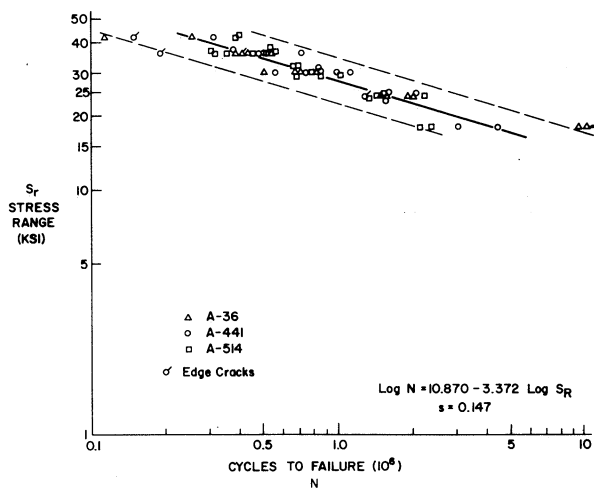Fig. 1. Effect of minimum stress on fatigue strength of welded beams

Fig. 2. Effect of grade of steel on fatigue strength of
welded beams

of welded beams and girders. Nearly all test data are within
the 95% dispersion band. The minimum stress (dead load)
and the type of steel had little effect on the fatigue strength
relationship.

The results for all rolled beams, welded beams, and
coverplated beams from these 374 test specimens are
summarized in Fig. 3. It is visually evident that the stress
range controlled the fatigue life for every type of beam. It
is also apparent that welded beams have well defined sur-
vival limits and that the attachment of welded cover plates
reduced the fatigue strength of the rolled and welded beams
a significant amount.

Examination of fatigue crack surfaces revealed that fa-
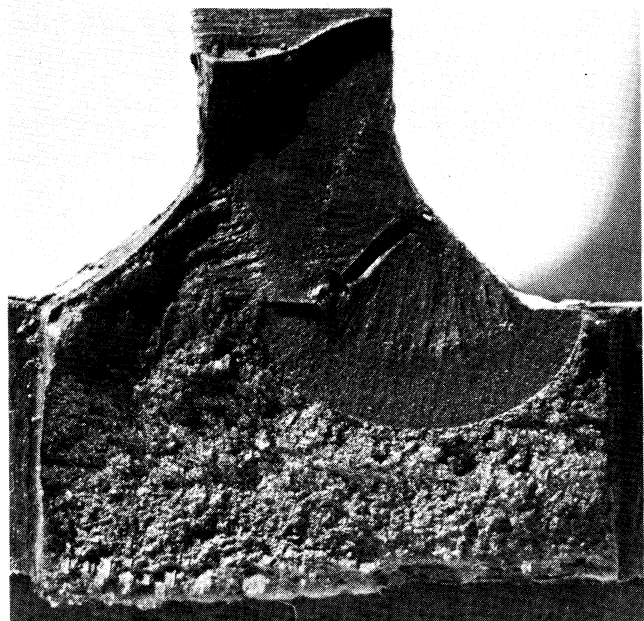tigue cracks grew from small initial flaws or discontinuities.

Fig. 4. Fatigue crack growth from gas pocket in fillet weld

Figure 4 is a photograph of a fatigue crack which propa-
gated from a gas pocket in the web-to-flange welds to a
circular shape (with a missing first quadrant) and then
towards the flange edges of a welded beam. At ends of cover
plates which developed fatigue cracks (Fig. 5), the cracks
grew from the toe of welds. The crack surface in Fig. 6 il-
lustrates a crack which penetrated through the flange plate
thickness and then propagated across the flange width.

This phenomenon of fatigue crack growth from a small
discontinuity to failure, and the fatigue strength reduction
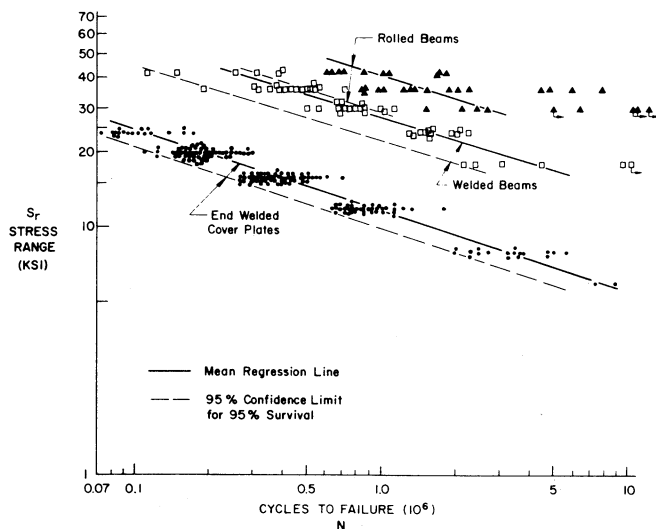due to welding of partial length cover plates to welded

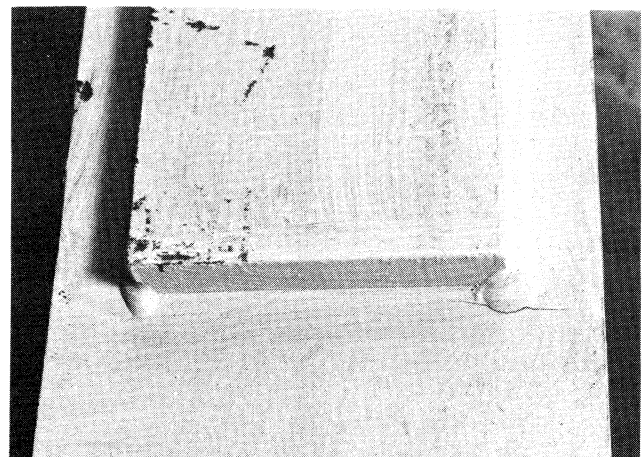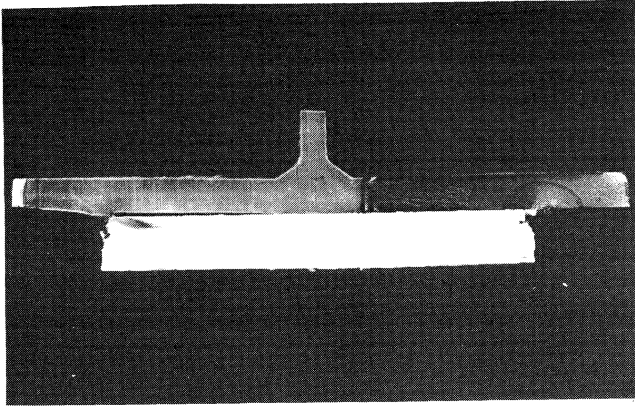Fig. 3. Fatigue strength of rolled, welded, and
coverplated beams

Fig. 5. Fatigue crack at end of cover plate

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Fig. 6. Fatigue crack growth from weld at end of partial length cover plate

beams, indicated that the fatigue strength of the welded beam was the upper bound that could be attained by a welded steel beam with or without welded attachments. This fact has been demonstrated through additional experimental results from 157 beams and girders, as well as from analytical studies utilizing the concepts of fracture mechanics and fatigue crack growth.[5] The fatigue strength of beams with partial length cover plates was found to be the lower bound condition for all steel beams with welded details.
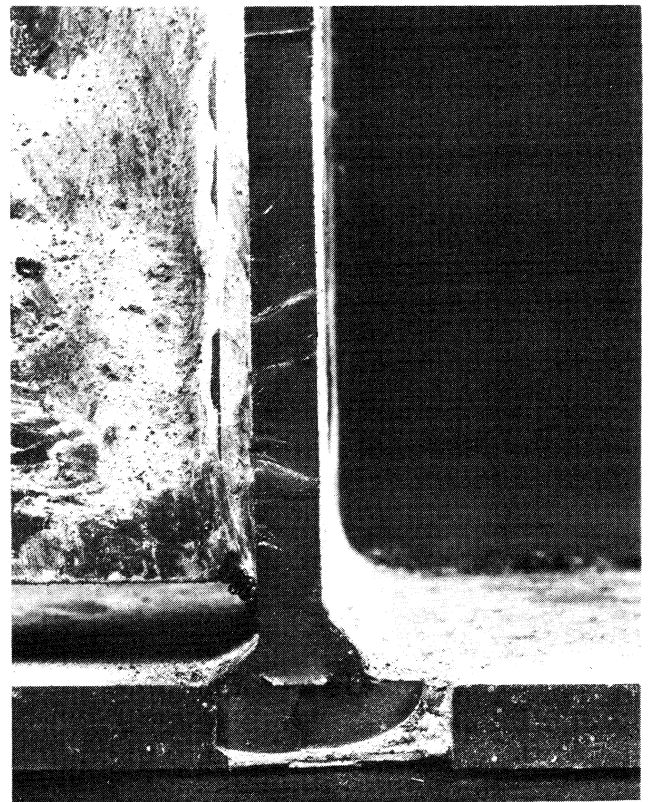
BEAMS WITH WELDED TRANSVERSE STIFFENERS

One of the details examined during the second phase of the NCHRP project was the attachment of full length transverse stiffeners to beams and plate girders. These stiffeners were welded to the web alone, or welded to both the web and the flange plate.

For stiffeners welded to the web alone, regardless of the geometrical configuration of the end of the stiffener plate, fatigue cracks grew at or near the end of the weld. Two such cracks are shown in Fig. 7. The crack surface shown in Fig. 8 illustrates the unevenness of the crack in the web near the end of weld, suggesting strongly that several small cracks had formed there and then grown up the web plate and down into the flange plate. Inspection of a large number of crack surfaces confirmed the phenomenon of growth from small micro-flaws at or near the end of the stiffener weld. Again, for all test specimens, the fatigue life was essentially exhausted when the crack penetrated through the flange plate.

Fig. 7. Fatigue crack at end of transverse stiffener welded to web alone

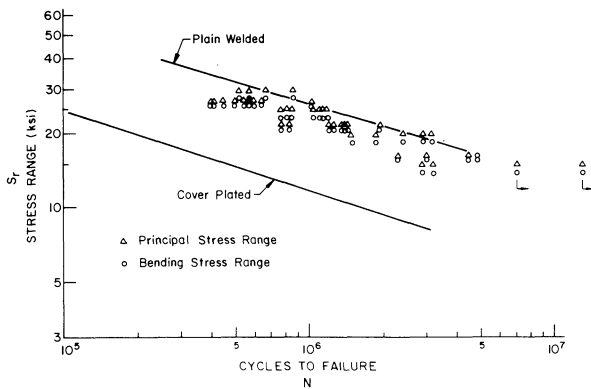*Fig. 8. Fatigue crack growth from weld near end of transverse stiffeners*

Fig. 9. Fatigue strength of transverse stiffeners welded to web alone—comparison of principal and bending stress range

Fig. 11. Coalescence of small fatigue cracks at the stiffener-to-flange weld

Evaluation of the stress range at the point of initial crack growth indicated about the same range of nominal beam bending and principal stresses at each individual crack. The presence of shear (thus the principal stress) caused the cracks to propagate at an inclination to the transverse stiffener and perpendicular to the direction of the major principal stress range. The magnitude of the bending and principal stresses, however, were about the same.

The applied stress ranges are plotted in Fig. 9 against the corresponding fatigue life for all stiffeners welded to the web alone. A number of conclusions can be drawn from these results. First, the stress range controlled the fatigue strength, just as observed for welded beams and coverplated beams. The dominance of stress range was further confirmed, and minimum stress was not a governing factor. Secondly, the principal stress ranges were only slightly higher than the bending stress ranges for the design con-
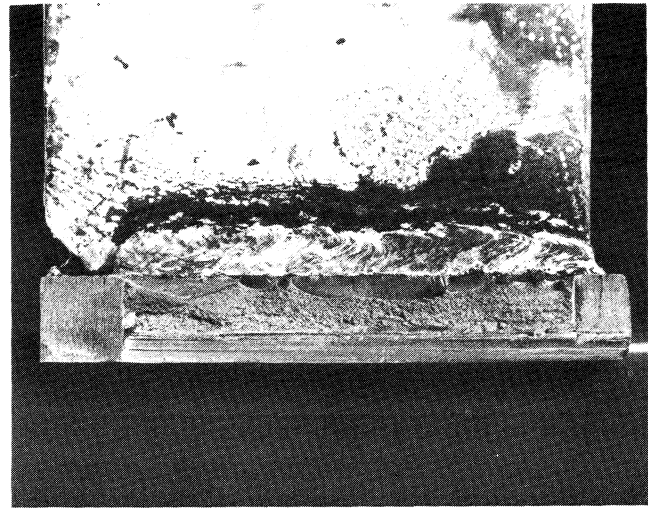
ditions normally expected in bridges. Finally, the fatigue strength of this type of stiffener was lower than the upper bound strength of welded beams, but was much greater than the fatigue resistance of coverplated beams.

For transverse stiffeners welded to both the web and to the flange, the fatigue crack typically grew from the stiffener-to-flange weld into the flange and toward the web. A typical crack is shown in Fig. 10. A number of small cracks may propagate simultaneously and join each other (or coalesce) as in the case of the stiffener-to-flange weld shown in Fig. 11, and of the stiffener-to-web weld illustrated in Fig. 12.
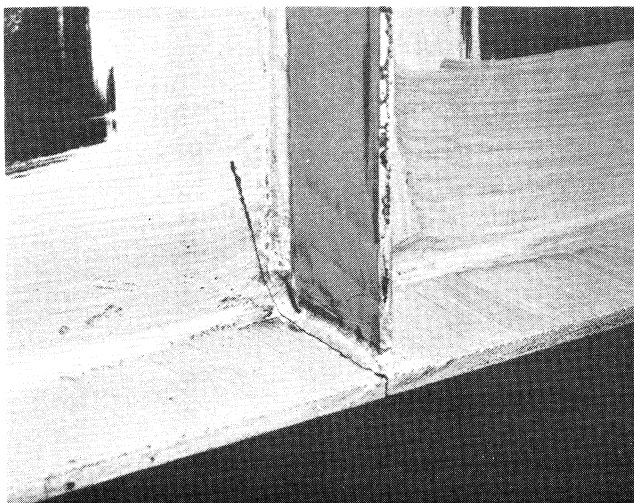

Fig. 10. Typical fatigue crack at end of transverse stiffener welded to both web and flange
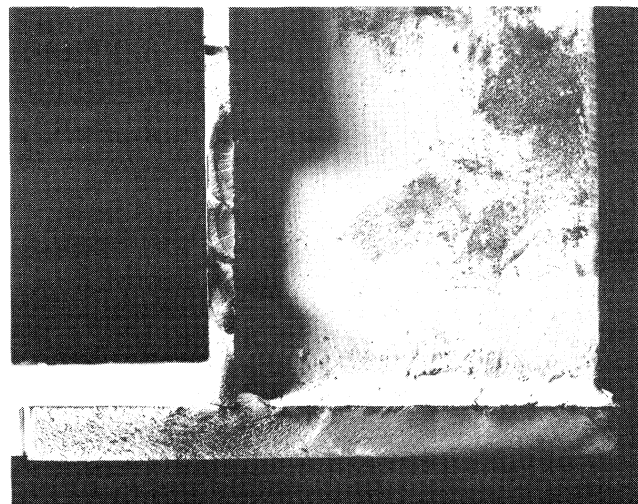

Fig. 12. Crack surface, growth from stiffener-to-flange weld and multiple cracks in web
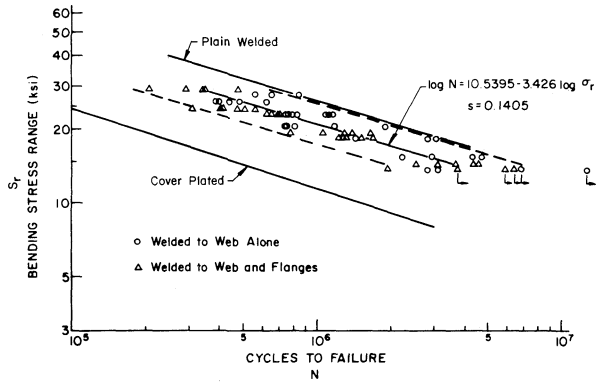
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Fig. 13. Fatigue strength of transverse stiffeners

The stress range-fatigue life results of beams and girders with transverse stiffeners welded to both the flange and the web or the web alone are summarized in Fig. 13. The test data for stiffeners which were welded to the web alone (Fig. 9) were not significantly different from those for stiffeners welded to the flange as well. Thus, for simplicity, only the primary bending stress range at the stiffener attachment needs to be considered.

BEAMS WITH WELDED ATTACHMENTS

Short steel plates are often attached to the flange or web of beams and girders, serving as gussets or other connection plates. The welds for these attachments are comparable to those used for transverse stiffeners and partial length cover plates. Therefore, fatigue cracks were expected to grow in the beam or girder from these attachments in the same way that fatigue cracks propagated from stiffeners and cover plate ends.

The laboratory testing of welded attachments included beams with small $1/4$-in., 2-in., 4-in., and 8-in. long plates
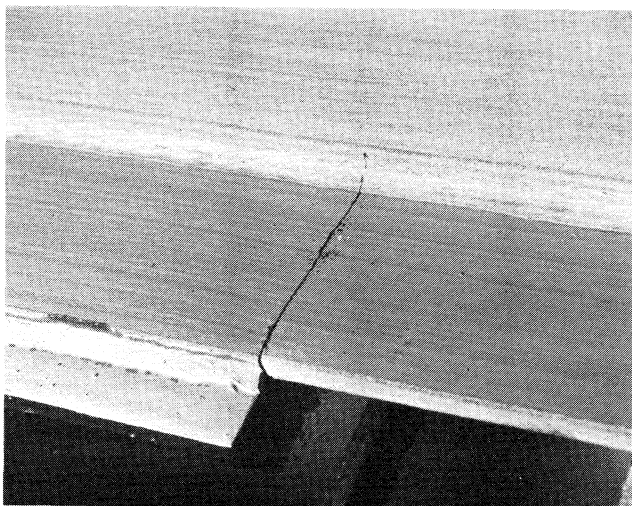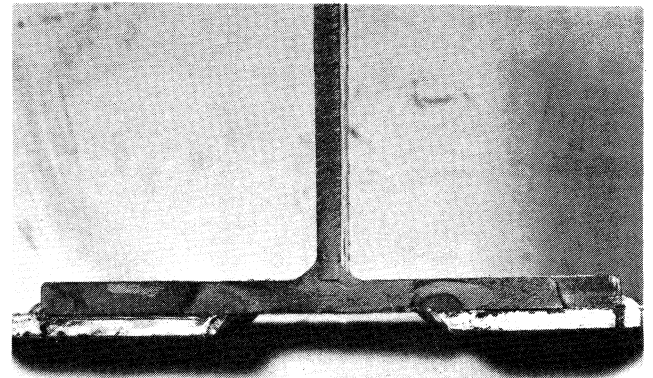

Fig. 15. Fatigue crack surfaces at ends of welded attachments

fillet welded to the flanges. The $1/4$-in. plates were attached edgewise and transverse to the beam length. Different combinations of longitudinal and transverse welds were used for 2-in., 4-in., and 8-in. plates. All fatigue cracks grew at the ends of the attachments, as expected.

Figure 14 shows a crack which formed at the end of an attachment. The crack surface shown in Fig. 15 is similar to the cracks that formed at the end of a partial length cover plate (see Fig. 6). Cracks grew from initial flaws at the weld end under the applied stress range. The fatigue strength was again controlled by the stress range alone. Most of the fatigue life of a beam was exhausted when the crack penetrated through the flange plate. The stress range-fatigue life relationship for beams with 4-in. and 8-in. attachments are summarized in Figs. 16 and 17. It is evident that the fatigue strength of each detail is between the upper bound and lower bound provided by welded and coverplated beams.
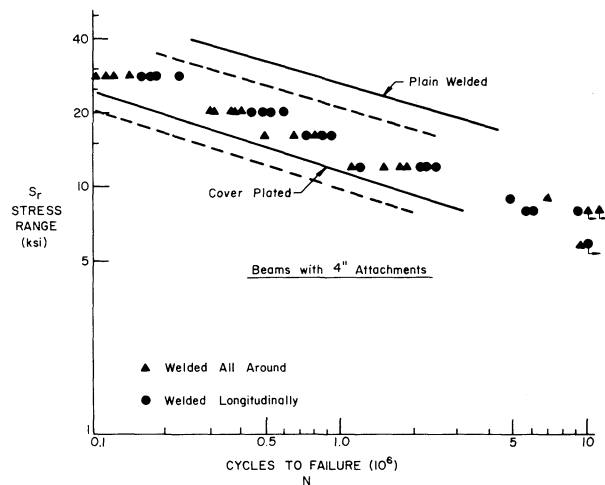

Fig. 14. Fatigue crack at end of welded attachment on flange

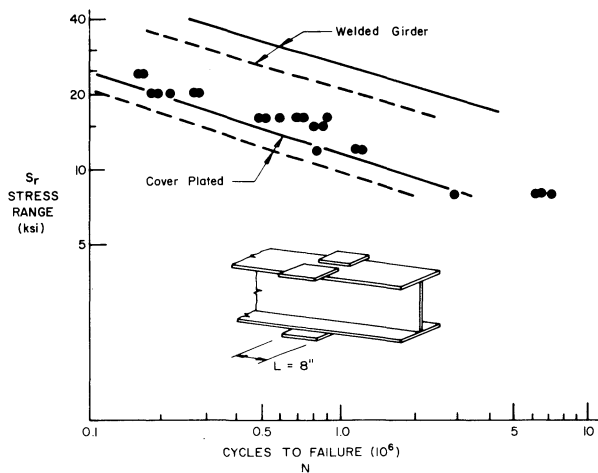Fig. 16. Fatigue strength of 4-in. long attachments

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Fig. 17. Fatigue strength of 8-in. long attachments

The comparison of the fatigue strength of beams with different attachment lengths resulted in a very important conclusion: the length of the attachment plate affects the fatigue strength of the beam. The test results in Fig. 16 for 4-in. long attachments are nearer to the upper bound than the data points in Fig. 17 for 8-in. long plates. The fatigue life of some of the 8-in. long attachments actually was lower than the mean life of partial length coverplated beams.

The lower confidence limits for all welded details are summarized in Fig. 18 for comparison. These are the 95% confidence limits for 95% survival from fatigue failure. All are straight (log-log) lines which are nearly parallel to each other. The reduction in fatigue strength from the upper bound of welded beams and girders is larger for details with longer attachment lengths. For welded attachments 8 in. or longer, the strength approaches the lower bound provided by coverplated beams, which are long welded attachments. The $1/4$-in. attachments and the transverse stiffeners have about the same geometry and hence the same fatigue strengths. Their results were combined to arrive at a single confidence limit and design relationship.
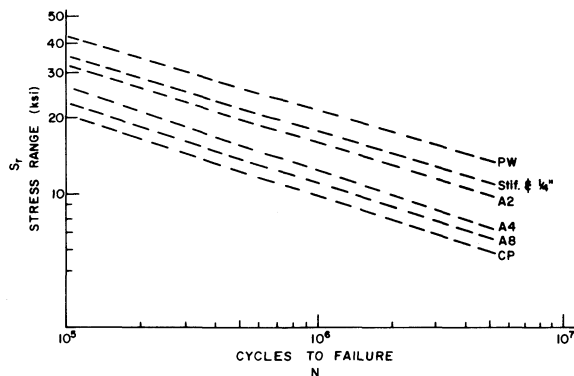

Fig. 18. Lower confidence limits for all welded details

The test data summarized in Fig. 18 show that any welded detail attached to a beam or girder reduces its fatigue strength. The longer the attachment length in the direction of the primary stress in the beam or girder, the larger the fatigue strength reduction. These results can be used as the basis for practical design limits.

FATIGUE CRACK PROPAGATION

To assist in classifying structural details where only a few test data were available and not well distributed throughout the desired regions of fatigue life, the concepts of fracture mechanics of crack growth were utilized. The extensive data available from more than 500 beam specimens was used to evaluate the applicable parameters.

The semi-empirical relationship proposed by Paris[10]

$$\frac{da}{dN} = C\Delta K^n \qquad (1)$$

yields an estimate for fatigue life in terms of the rate of crack growth and the stress intensity range ΔK. The number of cycles can be estimated from Eq. (1) when the stress intensity range is defined. This can be expressed as:

$$N = \int_{a_i}^{a_f} \frac{da}{C\Delta K^n} \qquad (2)$$

The stress intensity range can be expressed as:

$$\Delta K = S_r F(a)\sqrt{\pi a} \qquad (3)$$

where S_r = nominal stress range, $F(a)$ = a function of the geometry and configuration of the detail and crack, and a = crack size.

Correlations with the beam test results suggested that the crack growth rate for all the steels examined could be reasonably estimated using a coefficient $C = 2 \times 10^{-10}$ and an exponent $n = 3$ (Refs. 5 and 11). It is readily apparent in Fig. 18 that the stress range-cycle life relationship for each category results in a family of parallel curves with a slope approximately equivalent to $n = 3$.

Various studies have examined the function $F(a)$ in order to provide reasonable correlation with predicted and observed fatigue strength of welded details.[12-17] Equation (2) was used to estimate the stress range-cycle life relationships for several details.[5,11,13,15] An example is shown in Fig. 19 for transverse stiffeners welded to the web and flange.[5,13] The predicted strength is seen to be higher for welds with smaller initial discontinuities. Furthermore, the predicted fatigue strength agrees well with the test results and further confirms the fact that the fatigue resistance of welded details is represented by a family of parallel lines. The exponential relationship derived from Eq. (2) takes the form $N = GS_r^{-3}$, which is directly compatible with the results summarized in Fig. 18.

Since very few test data exist in the longer life region, i.e., $N > 10^7$ cycles, Eq. (2) was also useful in helping to establish a fatigue limit. The horizontal segments of the de-
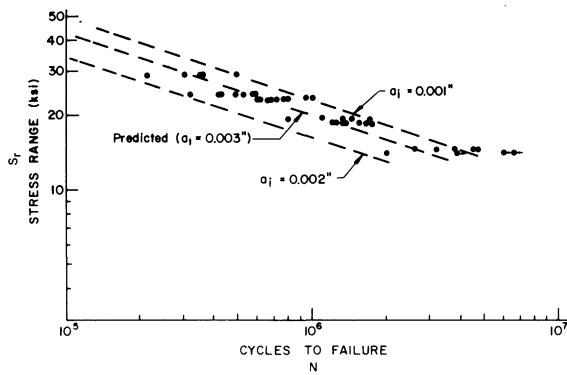
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Fig. 19. Comparison of predicted fatigue life with test data for stiffeners welded to web and flange

sign curves summarized in Fig. 20 were in part based on this analysis. The crack growth threshold lower limit has been established at about 3 ksi $\sqrt{in.}$ for structural steels.[5,11] This was used together with Eq. (3) to provide an estimated lower bound fatigue resistance which was also correlated with existing test data.

Further work is currently underway on this aspect of fatigue behavior. It is apparent that it provides a useful tool which can be used to supplement experimental data.

IMPROVING FATIGUE STRENGTH AND RETROFITTING FATIGUE DAMAGED MEMBERS

The 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 at cover plate end.

In addition, fatigue cracking has been observed in the field at coverplated beam bridges that carried unusually
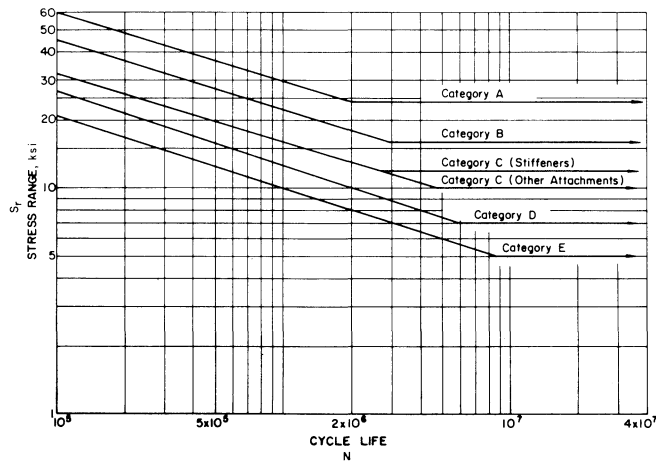

Fig. 20. Design stress for categories of structures and details

high volume of heavy truck traffic which caused large numbers of stress cycles.[18]

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 of arresting the progress of fatigue damage that occurs at the weld toes of severe notch producing details, where the probability of failure is greatest, were needed.

An experimental program was carried out on sixty steel coverplated 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.[19] Three of the most successful methods reported in the literature for as-welded details were utilized.[20-23] 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.

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, but none at all at the highest level of stress range.[19] Similar results were obtained in earlier studies on as-welded details which indicated that erratic results could be expected.

The results of earlier tests on ground coverplated beams showed that grinding accompanied by fine finishes decreased the stress concentration condition and resulted in substantial improvements in the fatigue strength. The results also indicate that substantial scatter can be expected from the ground details.

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 and low minimum stress levels and at higher stress range levels as well.

The results of all beams with peened details that were tested under a low minimum stress level, or that were peened under their minimum load, are summarized in Fig. 21. When 10 ksi minimum stress was applied to these beams, it eliminated most of the beneficial effects of the peening treatment. 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.
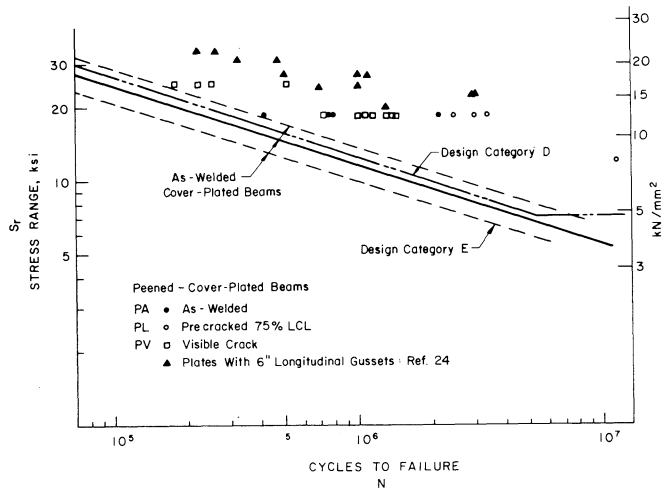
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Fig. 21. Improvement of fatigue strength of coverplated beams
by peening

Fig. 22. Improvement of fatigue strength of coverplated beams
by gas tungsten

Also shown in Fig. 21 are test results on small plate specimens with 6-in. longitudinal gussets welded to their surface. These tests were reported by Gurney and were made on as-welded specimens.[24] The studies on welded attachments reported in Ref. 5 have demonstrated that the attachment length has a significant effect upon fatigue strength. Hence, these 6-in. longitudinal gusset plates were expected to exhibit slightly more life than those provided by coverplated beams. This was confirmed by the test data. All of the peened plate specimens fell near the upper limit provided by peened coverplated 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.

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 and application of the gas tungsten arc remelt process did not succeed in completely fusing the fatigue crack, 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 Fig. 22. Except for those failures in precracked beams that occurred because of failure to incorporate the complete crack into the gas tungsten arc remelt, 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.

Data available from other sources is primarily on small plate specimens with transverse gussets that provide a non-load-carrying joint.[20,21] The studies on NCHRP Project 12-7 have indicated that this type of specimen provides fatigue behavior that is similar to stiffener type details.[5] No data was available on coverplated beam details
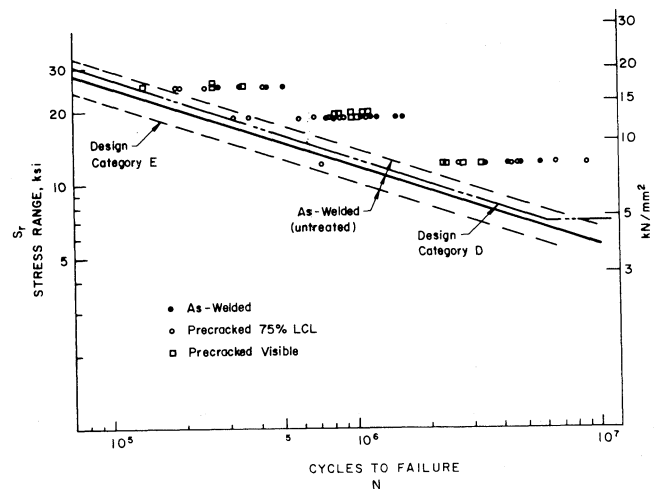
that had been subjected to gas tungsten arc remelting at fillet weld toes.

An etched cross section of the transverse end weld is shown in Fig. 23. The remelt penetration is visually evident at the weld toe. It was possible to provide up to 0.2 in. penetration in the gas tungsten arc remelt. Figure 23 also demonstrates the reason that an upper bound to fatigue strength was observed for welded coverplated 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.

Work is continuing on retrofitting procedures for fatigue damaged members. Several bridges have been retrofitted using peening and gas tungsten arc remelt procedures. The results of these studies will be reported in the future.
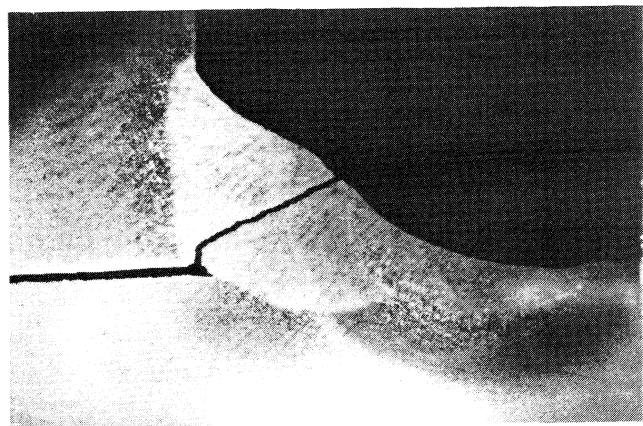

Fig. 23. Etched cross section of fillet weld after gas tungsten
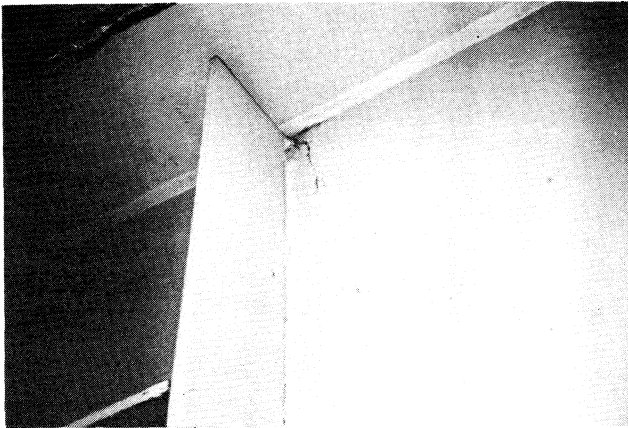arc remelting of toe

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Fig. 24. Fatigue crack at end of transverse stiffener opposite floor beam in negative moment region

Fig. 26. Schematic of floor beam rotation and web displacement

SECONDARY STRESSES AND FATIGUE STRENGTH

Bridges and buildings are essentially linear structures designed for in-plane loading and deflections. Out-of-plane behavior and interaction between members have been considered more frequently in recent years when evaluating the static strength of structures.[25] Greater attention must also be given to these factors when designing steel structure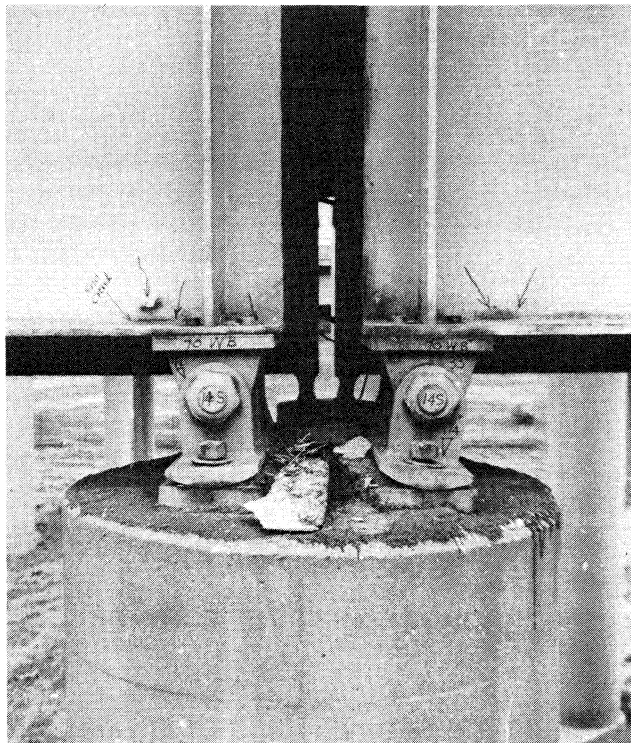s against fatigue failure. Most fatigue cracks in structures have been caused by secondary stresses which were induced by out-of-plane actions.[26]

An example of secondary stresses and the resulting fatigue damage that has developed is illustrated in Figs. 24 and 25. Figure 24 shows a fatigue crack that has occurred at the upper end of a transverse stiffener which is opposite a web connection plate. A floor beam frames into the connection plate as shown in Fig. 26. These cracks were observed to develop throughout the negative moment region at connection plate ends which were not welded to the top tension flange. Cracks would typically form at the end of the stiffener and along the web-to-flange weld toe in the girder web.

Cracks were also observed to occur near end supports when the connection plate was not attached to the bottom flange, as illustrated in Fig. 25. At these locations the bottom tension flange was restrained from moving laterally by the support. Any lateral movement of the floor beam connection plate was concentrated at the web gap between the end of the plate and flange.

Fig. 25. Fatigue cracks at end of transverse stiffeners opposite floor beam connection plate at supports

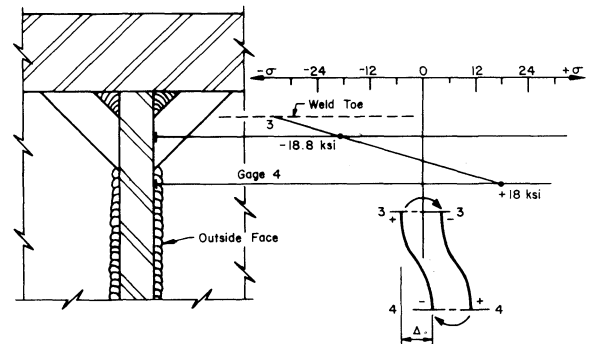Fig. 27. Measured plate bending stress distribution in web gap at end of transverse stiffener

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Measurements at several locations on the bridge confirmed that the cause of the web cracking was due to the end rotation of the floor beam, which in turn pushed the web out-of-plane, as shown schematically in Fig. 26. A typical measured stress distribution in the web gap region is shown in Fig. 27. The resulting strain measured on each side of the web confirmed that the web was being forced out-of-plane as vehicles loaded the floor beam. The stress gradient shown in Fig. 27 occurred under a loaded truck. Even automobiles were observed to cause a stress range of 2 or 3 ksi.

The large stress ranges were mainly confined to the top web gap in the negative moment region where the concrete slab restrained the top girder flange (Fig. 24), and near end supports where the connection plates did not coincide with the bearing stiffeners and were not welded to the bottom tension flange (Fig. 25). At other positive moment locations the girder flange was sufficiently flexible that no large out-of-plane web stresses were introduced. However, care must be exercised in these regions as well. In several instances large movements coupled with stiffer girder flanges have resulted in cracking in the positive moment regions.[9]

Tests[27,28] on specimens simulating out-of-plane web bending stresses have shown that the fatigue strength of the web at the web-flange weld connection is the same as for transverse stiffeners. It is readily apparent from the large stress range shown in Fig. 27 that cracking would be expected in a relatively short period of time.

In the studies reported in Ref. 5, a series of tests were conducted with lateral bracing members attached to the stiffeners, which exerted modest lateral forces and deformation into the girder. However, these deformations were introduced near the center of the span and the girder flange did not offer significant out-of-plane resistance. The results of these tests are plotted in Fig. 28. It is readily apparent that the test data are well above the lower confidence limit for Category C, which is the design condition for the stiffener end. No reduction in fatigue strength is apparent. However, these tests did not introduce the flange restraint provided by the slab and end supports.
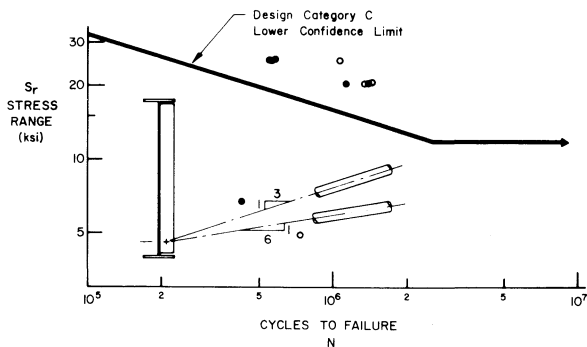
Since the cause of the fatigue cracks shown in Figs. 24 and 25 was web plate bending from relative lateral deflections between the stiffener and the flange, connecting the stiffener to the flange will eliminate the plate bending stresses and prevent such cracking. This has been confirmed by the behavior of connection plates welded to the top flange in positive moment regions where the concrete slab has restrained the flange as well. Existing cracks might propagate under the girder bending stress range if the crack front is not parallel to the stress field. To insure that such cracks will not grow, holes can be drilled in the web as illustrated in Fig. 29. Such retrofitting has been carried out on several bridges with good success. Laboratory studies now underway are intended to provide further information on this procedure.

SUMMARY

This paper summarizes the results of studies on the fatigue behavior of beams and girders with welded details. Stress range was found to account for the variation in cycle life for each welded detail. The type of steel and the dead load stress level had no appreciable influence on fatigue strength. The type of detail was a major factor. Transverse stiffeners welded to girder had about the same fatigue strength as short ($<$ 2 in.) attachments. However, an increase in attachment length resulted in significant decreases in fatigue strength. An attachment only 8 in. long in the direction of applied stress was about the same as a partial length coverplated beam.

Design guidelines were developed from these results and other data available in the literature in terms of stress range and cycle life for various categories of welded details. The concept of fracture mechanics of stable crack propagation was used to evaluate the test data and help define fatigue limits.

Fatigue cracks have been observed at the ends of welded cover plates in a few highway bridges which have been subjected to very high numbers of stress cycles. Work is currently underway in this extreme life region. Because of the possibility of fatigue damage, methods to improve fatigue strength and retrofit fatigue-damaged steel members have been examined. Grinding the weld toe provided some
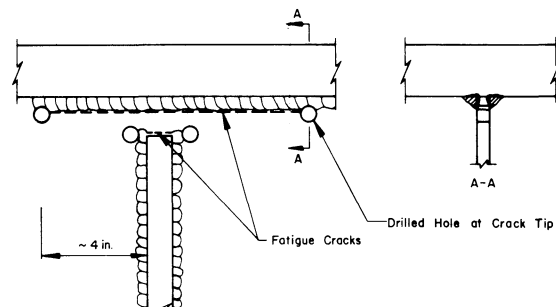

Fig. 28. Test data of girders with lateral bracing at transverse stiffener

Fig. 29. Retrofit of fatigue cracks of web plate bending

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improvement at low levels of stress range, but the results were erratic. Peening the weld toe introduces residual compressive stresses at the stress concentration and at small cracks. It was found to be effective in improving fatigue strength and retrofitting fatigue-damaged details if carried out with the dead load in place and the fatigue crack less than $\frac{1}{8}$-in. deep. The most reliable retrofit procedure was found to be gas tungsten arc remelting at the weld toe. Remelting removed discontinuities at the weld toe and fused small fatigue cracks up to 0.2-in. deep.

An example of fatigue cracking from secondary stresses was reviewed. The relative lateral displacement in the web at stiffener ends was shown to be the primary reason for formation of fatigue cracks in the webs of bridge girders. Lateral displacements occurred as a result of floor beam deflections and introduced out-of-plane bending into the web gap. Field measurements confirmed the existence of high web gap bending stresses. Preventing relative displacement will eliminate the development of such cracks. Retrofitting such fatigue damaged members was also reviewed.

Cyclically loaded members can be proportioned to provide adequate levels of fatigue strength through proper design and selection of weld details. Consideration must be given to the applied loads and the deformation introduced into the structure.

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