current AASHTO fatigue-design curves should be equally applicable to predict the fatigue behavior of weathered as well as unweathered structural-steel components.

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Fatigue Strength of Weathered and Deteriorated Riveted Members

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ABSTRACT

A study has been performed on the fatigue resistance of corroded and deteriorated riveted members. The need for this study arose from the concern with the large number of riveted structures functioning today that have various degrees of corrosion and potential fatigue damage. The validity of AASHTO and American Railway Engineering Association category D that is generally used for riveted connections is uncertain, particularly near the fatigue limit. A series of fatigue tests was carried out on 80-year-old steel bridge stringers with a riveted built-up cross section. The stringers were significantly corroded along the compression flange and locally at the tension flange. The stress ranges that were applied were selected between the fatigue limits of design categories C and D. The corroded region of the tension flange proved to be the most severe condition, varying between categories C and E. The category D fatigue limit appears to be applicable to the rivet detail studied. The reduction of the compression flanges had no effect on the performance of the member. A strong frictional bond between section components was found to have a beneficial effect on fatigue life. A

series of reduced-temperature tests on a cracked stringer did not induce fracture of the cracked component and confirmed the redundancy of riveted built-up sections fabricated from mild steel.

Major concerns of bridge engineers today are the safety of old riveted structures and the potential fatigue damage that has accumulated. Many of these structures were fabricated and placed into service at the beginning of the century. The question of safety is of increasing importance as ever-intensifying traffic, deteriorating components, and accumulation of large numbers of cycles are a reality for highway, railroad, and mass transit bridges.

The criteria adopted for control of fatigue and fracture of new bridge structures are based on studies of modern welded construction and ongoing laboratory research on welded members. Most older bridges were constructed of riveted built-up members. Research is needed to establish better estimates of the fatigue resistance of riveted built-up sections.

Most of the previous laboratory work has been carried out on simple butt splices. A further limitation is that none of the previous testing has been performed with stress ranges less than 97 MPa. Both the AASHTO and the American Railway Engineering Association (AREA) specifications use a lower-bound estimate based on these limited data to classify the fatigue strength of riveted built-up members (1,2). This lower bound corresponds to category D in the joint classification system. A description and summary of the data base used are given in the commentary to the AREA specifications (2).

A recent literature survey by the authors confirmed the validity of category D as a lower-bound estimate for fatigue strength. Figure 1 shows the collected test results for normal clamping force and bearing ratios as permitted by the specifications. The test results for bearing conditions that exceed the specification limits are shown in Figure 2, which shows that some results fall below category D.

A pilot test program on the high-cycle fatigue behavior and fracture resistance of riveted built-up members in their weathered and deteriorated state is described. Stringers from an actual bridge were used in this study. The need for this investigation stems from the desirability to ascertain whether or not a difference exists between splices, built-up sections, cover-plate terminations, and other details of riveted built-up members. Category D fatigue restrictions impose an enormous penalty on fatigue resistance. It is of particular interest to investigate the applicability of this category near the fatigue or endurance limit because no experimental data are available at values less than 97 MPa.

FATIGUE TEST PROGRAM

Fatigue tests were conducted on four riveted built-up stringers taken from a railroad bridge. The purpose of the test program was to

1. Establish fatigue life data on rivet details, with particular focus on the high-cycle region; and
2. Establish the fatigue behavior of the deteriorated built-up member.

The second objective deals with the effect of force redistribution to the other section components as a result of crack extension at a rivet detail or corroded flange angle. This redistribution raises stresses and produces progressive crack initiation and propagation in those components. Note that riveted built-up members possess a degree of redundancy, which allows for an important increment of life after cracking develops in one component. The factor of redundancy did not appear in the majority of the previous experiments, which had been conducted on simple splices.

The project concentrated on high-cycle fatigue behavior; that is, near the constant cycle fatigue limit of AASHTO and AREA categories C and D of 69 and 48 MPa, respectively. This generally meant continuation of cycling beyond 10^7 cycles.

Test Specimens

The fatigue tests were conducted on members that had been removed from a railroad bridge. It was a three-span riveted truss bridge that supported a

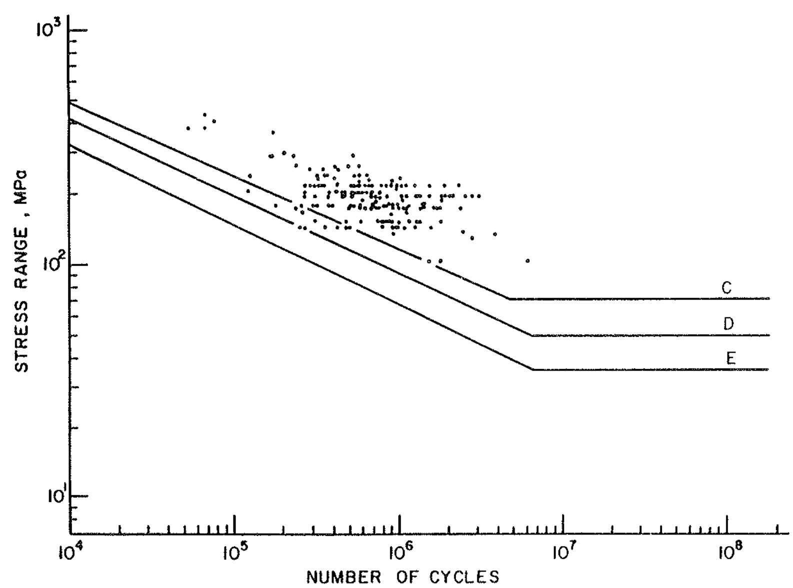

FIGURE 1 Fatigue resistance of riveted steel connections with normal clamping force and low bearing ratio (≤ 1.5).

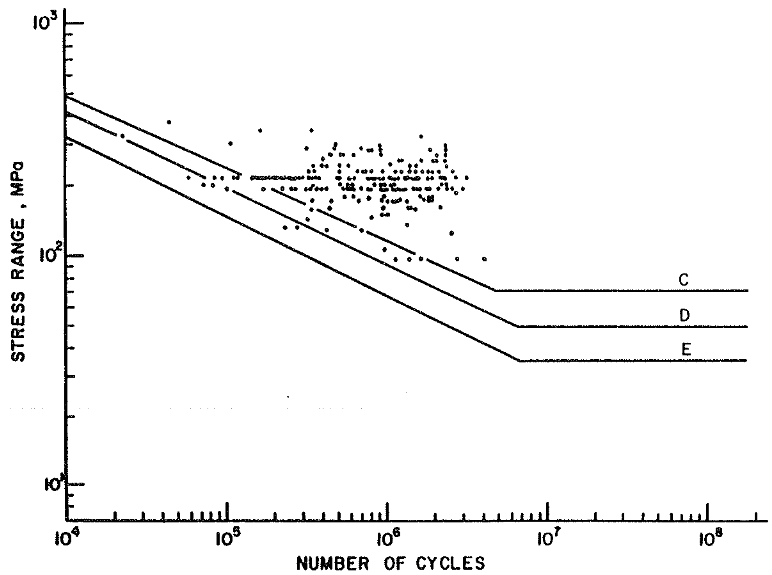

FIGURE 2 Fatigue resistance of riveted steel connections with normal clamping force and high bearing ratio (≥ 1.5).

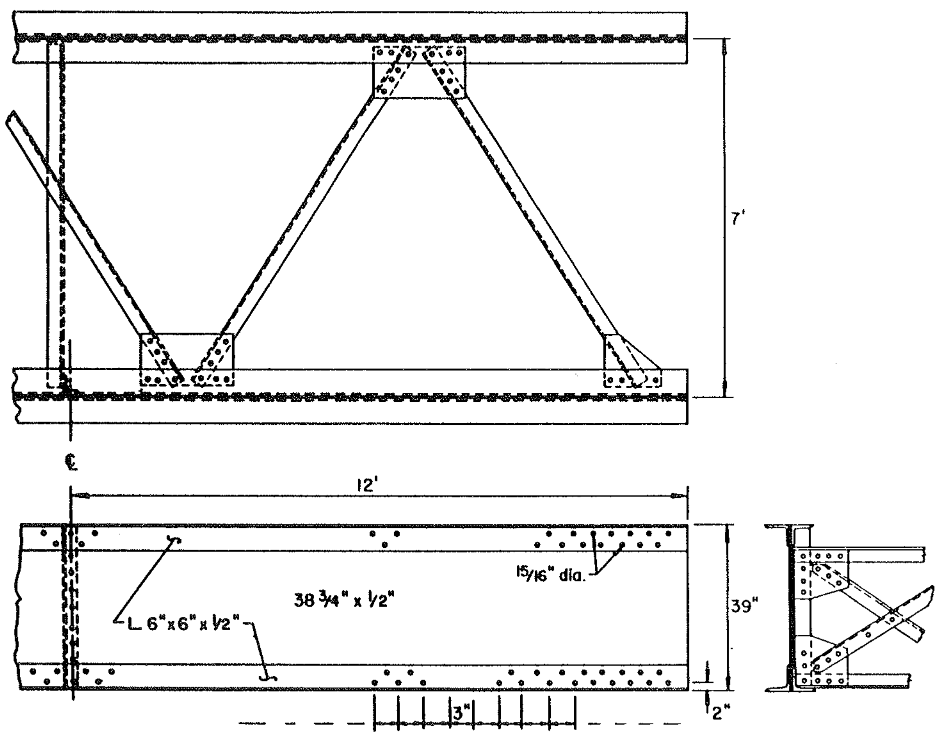

FIGURE 3 Stringers of the French Broad Ivy River Bridge.

single railroad track. The bridge was owned by the Southern Railroad Company and was located near Marshall, North Carolina, spanning the French Broad Ivy River. It was built in 1903 and demolished in 1982 for being obsolete rather than for malfunctioning. It was in service until demolition. Strain measurements made while in service indicate that about 1 percent of the stress cycles exceeded the category D fatigue limit; thus the cumulative fatigue damage from service was negligible (3). The bridge was constructed by the Phoenix Steel Company. The construction material used was medium steel, an early alloy steel. In 1903 alloy steels had just replaced wrought iron as the principal material for metal structures.

Fritz Engineering Laboratory acquired six of the stringers. They were built-up I-shapes that were 1 m deep and 0.32 m wide, and consisted of a web plate and four angles that were connected to the web by two rows of rivets (Figure 3). Their original length was 7.32 m. As cut from the bridge, they were about 6.10 m long.

The condition of the stringers appeared to be satisfactory. Their surface had been protected by a heavy tar coating and was eventually cleaned by sand blasting. Inspection indicated that the tension flange was relatively undamaged except at sections where the cross frame had been connected to the web (Figure 4). The bottom inside flange angle is seen to be severely corroded, so that a reduced thickness resulted as well as a partly eliminated rivet head at that section. The regions of reduced thickness contained a set of notches and associated stress concentrations, which proved to be rather severe.

Significant deterioration was generally present along the compression flanges, as illustrated in Figure 5, which shows a thickness reduction of the flange where the cross ties had rested on the stringers. Several long cracks were found in the compression flange at the location of the lateral bracing connection plates, as illustrated in Figure 6. It appears that the bracing connection had provided restraint to the top flange adjacent to a tie-bearing point.

Test Setup

The stringers were tested in a four-point bending setup. The repeated loads were applied by two Amsler 245-kN hydraulic jacks and one pulsator, as illustrated in Figure 7. The distance between the jacks was 1.53 m. The loading signal had a constant amplitude, and the frequency of the cycle was 520 cycles per minute.

Lateral braces were attached adjacent to each jack to prevent lateral movement caused by the slight distortion and lack of symmetry of the deteriorated section. The high frequency of the variable loads required special care to prevent vibration of the setup and the jacks.

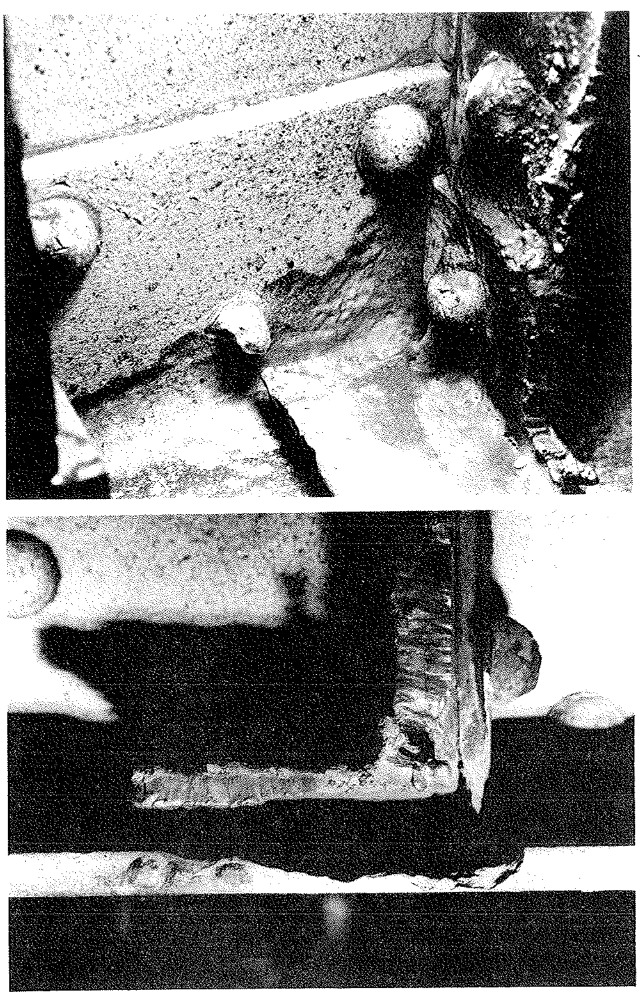

FIGURE 4 Flange angle corrosion at cross-frame connections.

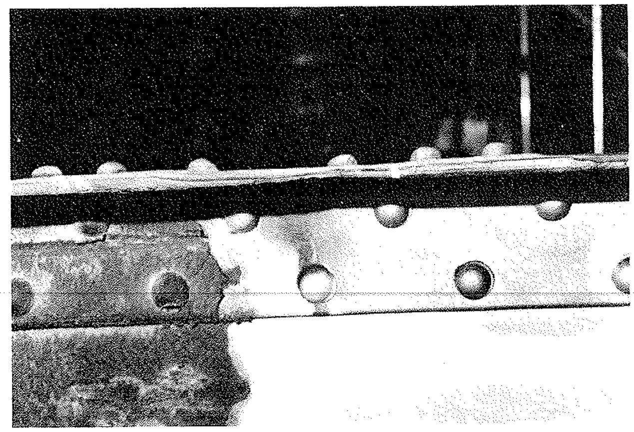

FIGURE 5 View of compression flange showing significant reduction in outstanding legs.

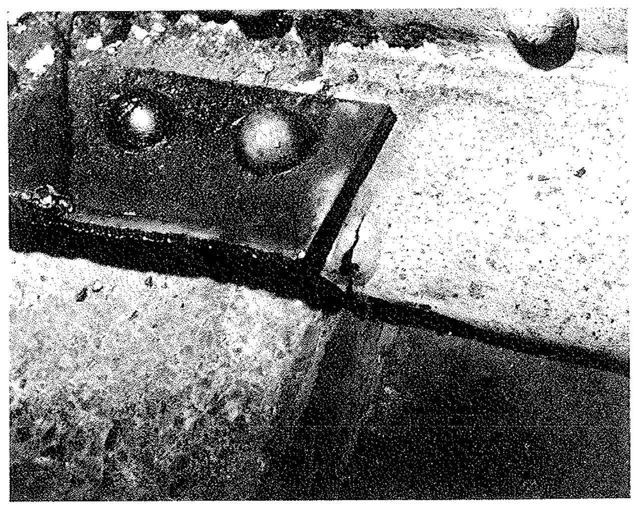

FIGURE 6 Preexisting crack in compression flange next to bracing connections.

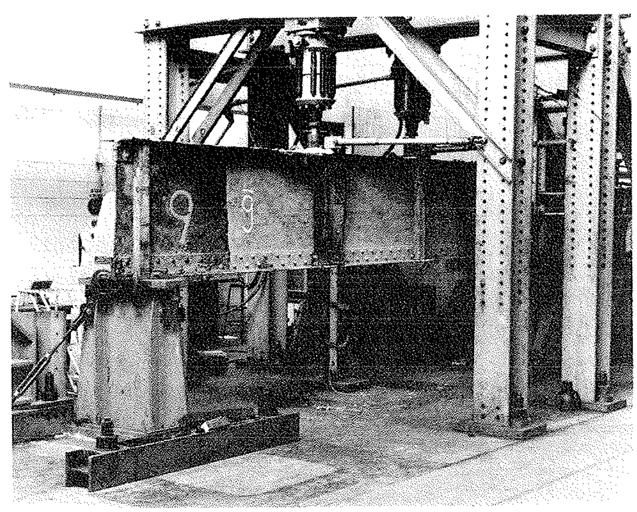

FIGURE 7 Fatigue testing setup.

Results of Fatigue Tests

General Remarks

Four of the stringers have been tested. The attention focused on the following details: (a) the corroded region of the flange angles, and (b) the riveted connection between the web and angles. The data in Table 1 summarize the test results for the

TABLE 1 Observed Cracking, Corroded Area

Test	Δσ Gross[a] (MPa)	Δσ Net[b] (MPa)	No. of Cycles (10^6)	Comment
1	73.4	75.2	3.77	Crack found >104 mm
			4.40	Angle severed
			4.99	Section failed
2	62.1	64.8	0.85	Angle severed
			1.45	Section failed
3	62.1	63.4	39.71[c]	No failure
4	55.2	57.2	1.19	Crack found >7.6 mm
			5.37	First hole drilled
			7.61[d]	Section spliced

Note: Ratio $R = \sigma_{min}/\sigma_{max} \approx 0.1$.
[a]Stress range at full section.
[b]Stress range at section reduced by corrosion.
[c]Test discontinued.
[d]Test stopped because of change in condition.

fatigue strength of the corroded region. For each test beam, the gross section stress range (at the full section) and the net section stress range (at the section reduced by corrosion) are tabulated. Results are tabulated for the number of cycles until first detected cracking, until failure of the corroded angle, as well as the number of cycles until failure of the section, if applicable.

The test results on the rivet details are summarized in Table 2. Only the test data for those

TABLE 2 Summary of Test Results at the Rivet Details

Test	Δσ Net[a] (MPa)	No. of Cycles[b] (10^6) N_c	N_f	Comment
1	65.9	6.52		Angle, bottom hole, top of hole
		7.34		Angle, bottom hole, top of hole
		8.70		Angle, bottom hole, top of hole
		12.98		Angle, bottom hole, top of hole
			18.25	Test stopped
2	59.3	36.50		Web plate, bottom hole, bottom of hole
			36.50	Test stopped
3	56.2	18.26	38.69	Angle, bottom hole, fillet, top of hole
			39.72[c]	Angle, bottom hole, fillet, top of hole
			39.72[c]	Angle, bottom hole, fillet, top of hole
	46.8	25.94		Angle, bottom hole, top of hole, shear span
	45.8	25.94		Angle, bottom hole, top of hole, shear span

Note: Only cracks at rivet holes not affected by a cracked section are included.
[a]Stress range at section reduced by corrosion.
[b]N_c = life until first cracking, and N_f = life until failure of component.
[c]Test stopped.

fatigue cracks that were not affected by cracking of the corroded section or other adjacent sections are listed. It was observed that progressing cracking of a section caused force redistribution from the cracked section components to the other cross-sectional elements. This redistribution often generated rapid cracking in those elements. This effect is shown in Figures 8 and 9. Soon after

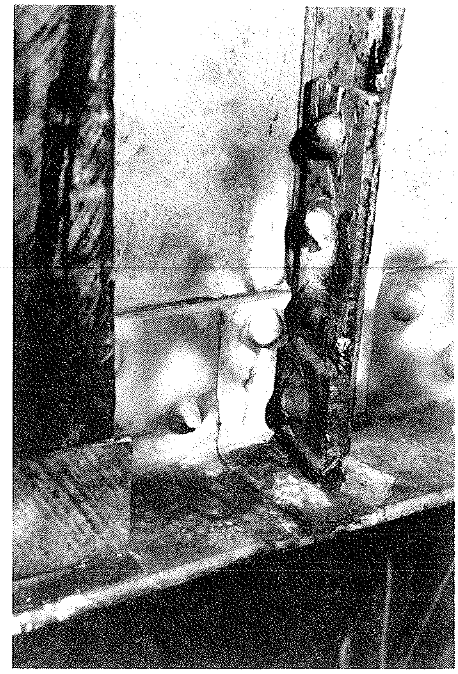

FIGURE 8 Crack at corroded area of test beam 1.

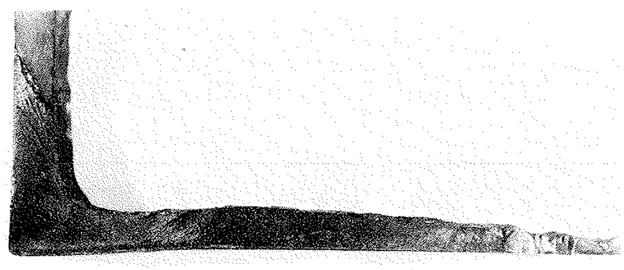

FIGURE 9 Surface of crack in corroded area of test beam 1.

severing of the corroded angle, the opposite flange angle developed several cracks within a short period of time.

Fatigue Resistance of Corroded Region

The four stringers tested all had the region of reduced thickness from corrosion. The degrees of reduction, however, were quite different and varied from 5 to 40 percent of the angle leg area. Figure 8 shows the cracked-reduced section of beam 1. The crack surface at this section is shown in Figure 10. Cracking was observed to initiate at multiple sites in the region near the edge of the angle where the thickness is minimal.

The corroded section of beam 2 after fracture is shown in Figure 11. Before the test a small existing crack had been detected in the corroded section, which was removed by grinding. This segment is shown in Figure 12.

The degree of corrosion of the third stringer is shown in Figure 13. No cracking was observed at

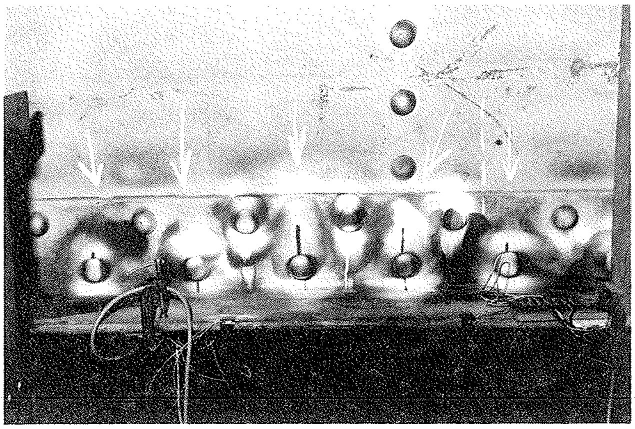

FIGURE 10 Set of cracks developed because of force redistribution in section.

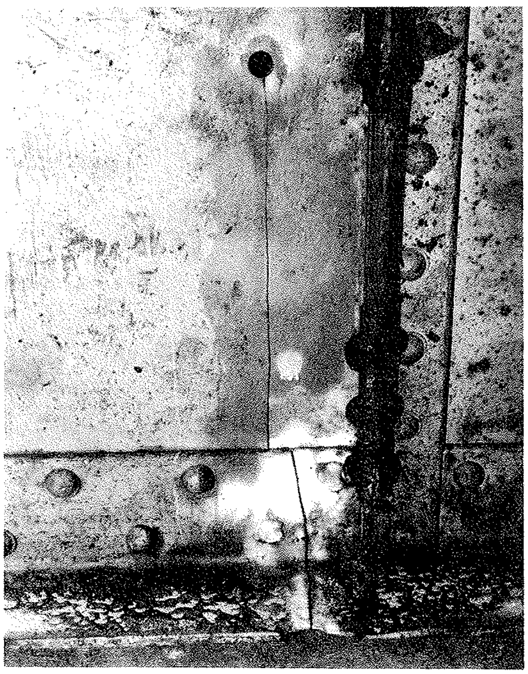

FIGURE 11 Cracked section at corroded area of test beam 2.

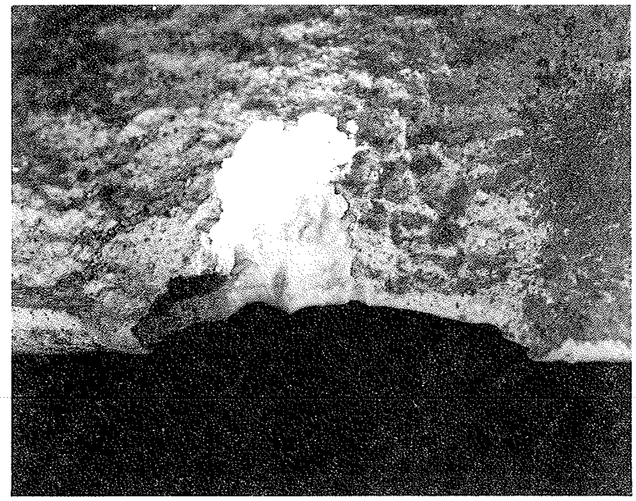

FIGURE 12 Initial crack in corroded area of test beam 2.

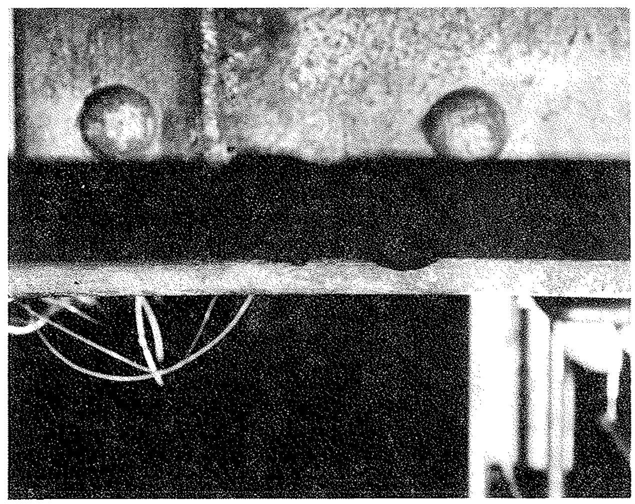

FIGURE 13 Corroded area of test beam 3.

FIGURE 14 Corroded area of test beam 4 with crack, reinitiated after having holes drilled.

this section throughout the test. The thickness reduction was less than half the original thickness.

Figure 14 shows the crack in the reduced section of beam 4. This crack was arrested by drilling holes centered on the crack tip after it had grown to a length of about 64 mm. The crack subsequently reinitiated after 2.2 million additional stress cycles, and the section was finally spliced to permit continued testing of the stringer.

The fatigue test data of the corroded region detail of the flange angle are shown in Figure 15. The stress range is defined on the net section.

The corroded areas of stringers 2 and 4 (see Figures 11 and 14) had a fatigue life at the first observed cracking that was lower than that provided by category E. The corroded detail of stringer 1 (see Figure 8) had a fatigue resistance slightly lower than category C. Stringer 3 (see Figure 13) did not crack at this location.

The deterioration of the flange angle of beam 3

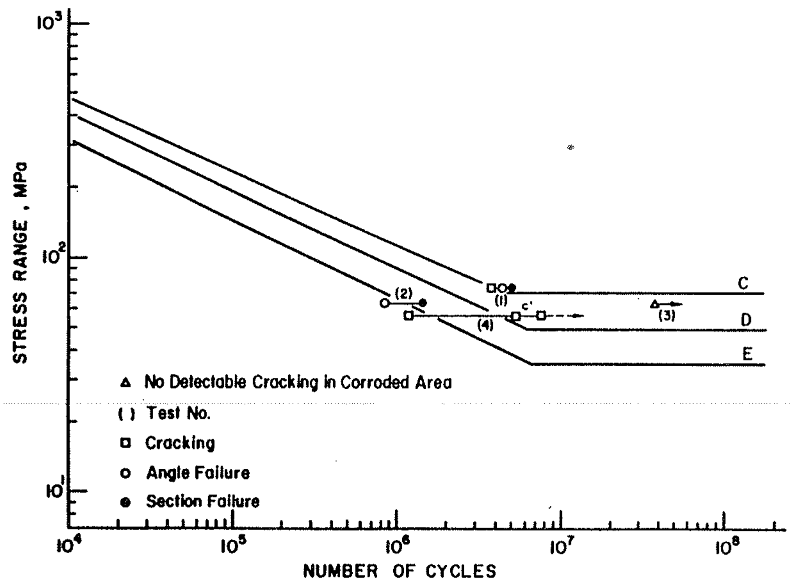

FIGURE 15 Fatigue resistance of corroded area details.

was not severe enough to have a crack initiate. The severity of the corroded regions of stringers 1, 2, and 4 were greater than that predicted by considering the loss of cross-sectional area only, as they all plot well below category A. Apparently, the roughness of the surface and the associated stress concentrations are the major factors in making this detail more severe than anticipated.

The mechanism of crack formation in the corroded area is as follows. First, several small cracks form on the rough surface at the deeper notches close to the angle tip. These cracks coalesce and form a long, shallow surface crack. This surface crack then propagates through the flange thickness at the tip and becomes an edge crack, whereas small cracks continue to form in the corroded surface and to coalesce with the edge crack. The crack length measured at the bottom surface significantly lags behind the length measured at the corroded top surface.

A final note should be made on the crack length at discovery in the various cases. For beam 4, the crack was small when detected--7.6 mm. This condition is identified in Figure 15 as cracking. The condition where the crack length has increased to approximately the length at discovery for beam 1 is represented by point C'. Hence the fatigue strength of the corroded sections of beams 1 and 4 are comparable.

All three stringers that developed cracks in the corroded section sustained a substantial number of additional stress cycles before the section failed. The numbers of cycles at failure of the angle of the cross section are indicated by angle failure and section failure, respectively.

Fatigue Resistance of Rivet Details

Fatigue cracks formed at rivet holes, as illustrated in Figure 16. The crack surface of one of these details is shown in Figure 17. The fatigue strengths of the rivet details that cracked independently of each other are summarized in Table 2 and plotted in Figure 18 as a function of the net section stress range. In Figure 18 cracking denotes first observed cracking. Failure of the flange angle developed for stringer 3 only. The cracked angles of the remaining stringers were retrofitted before failure could

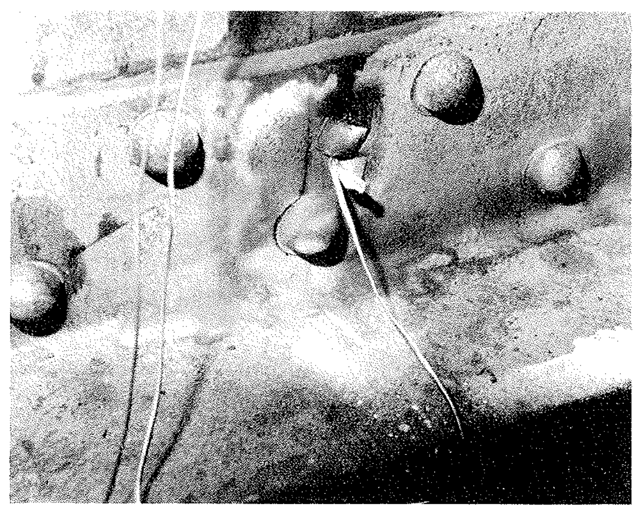

FIGURE 16 Fatigue cracked riveted section in test beam 3, after several reduced temperature tests.

FIGURE 17 Cracked surface of crack at rivet hole running into bottom flange.

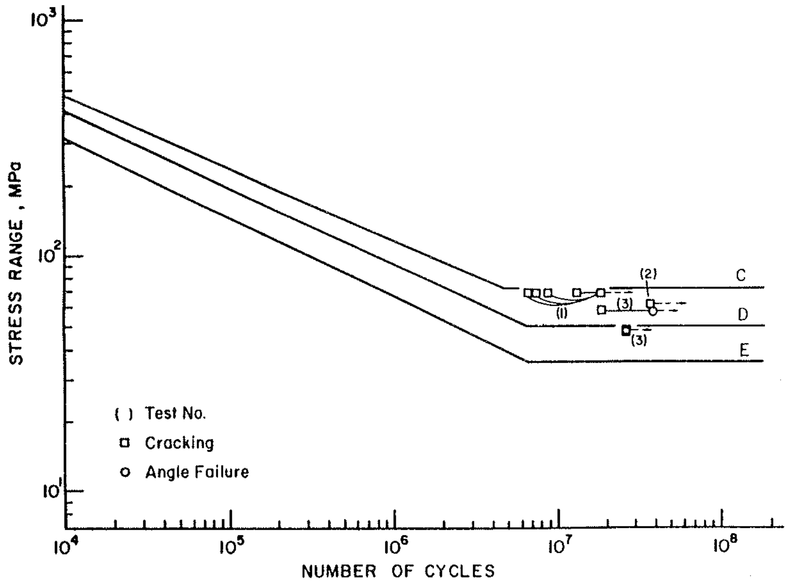

FIGURE 18 Fatigue resistance of rivet details.

occur. No tests were continued until failure of the riveted section because of cracking at rivet holes.

Fatigue cracking was observed to occur below the fatigue limit for category C (69 MPa). Two cracks developed below the fatigue limit of category D (48.3 MPa). Both of these cracks were located in a shear span. The stress condition that corresponds to bending and shear is slightly more severe than bending alone if the rivets are in bearing.

The literature review demonstrated that clamping force and bearing ratio were the principal variables influencing the fatigue behavior of riveted joints (4). Most of the cracked-rivet details are located in a constant moment region, so that the rivets do not transmit a bearing force. This is a favorable condition. At the same time, the rivets appear to be tight, which is favorable as well.

The available shop drawings do not indicate the method of hole preparation. The apparent distortion of the holes may be a result of punching or driving of the rivets.

There was no clear evidence of cracks existing at the beginning of the tests. In one or two instances a dark oxide was found on the crack surface, but sometimes this oxide was located away from the hole. In at least one instance it appeared this had occurred when the angles were flame cut from the section in order to expose the crack surfaces. None of these cracks corresponded to an appreciably different fatigue life than others without any indication of oxide.

The majority of the rivet details were observed to develop first cracking at the top of the hole, even though the nominal stress range is higher at its bottom. Most cracks initiated at the inner surface between web plate and angle at the edge of the hole. It is possible that fretting has aided crack initiation.

Observations during the test indicated that there was a significant influence of a frictional bond between the web plate and angles. This bond was from a paint and corrosion product. The significance of this influence lies in the effect it had on the propagation velocity of the crack. The frictional resistance between web plate and angle, which was adjacent to the crack as well as ahead of the crack front, reduced the compliance of the cracked plate and the crack opening displacement. This decreased the crack growth rate and extended the fatigue life. Crack growth measurements indicate that the crack growth rate was fairly constant with increasing crack length, as long as the crack propagated in the vertical leg of the angle. This phenomenon is treated in more detail elsewhere (5).

The fatigue resistance of the riveted stringers is compared with the category C and D resistance lines, which are shown to represent riveted members (2) in Figure 19. Also plotted are the results of other tests on riveted members. Fatigue life is defined as first observed cracking in all cases. The truss joints tested by Reemsnyder (6) and the hanger members tested by Baker and Kulak (7) were subjected to much higher stress range levels than the stringers and therefore yielded much shorter fatigue lives. All three sets of test data confirm that category D is a lower bound for the fatigue life, as defined by the development of a small crack.

Cracking of Deteriorated Compression Flanges

The significant loss of section of the compression flange apparent in Figure 5 was typical of every test beam. In extreme cases this loss was sufficient to result in the development of fatigue cracking in the outstanding legs of the angle. Figure 20 shows one such fatigue crack. The crack surfaces are shown in Figure 21.

The cracks always started at the section that had the largest thickness reduction. The explanation for this phenomenon is that the compressional stress cycle resulted in yielding on the net ligaments of the reduced section as a result of stress concentration and reduced area. During the unloading part of the stress cycle a residual tension stress field formed, which permitted crack initiation. In these circumstances the crack propagated toward the heel of the angle and arrested after having grown out of the region of yielding. None of the cracks propagated past the heel. Hence the cracks in the compression flange did not adversely affect the load-carrying capacity of any of the test beams.

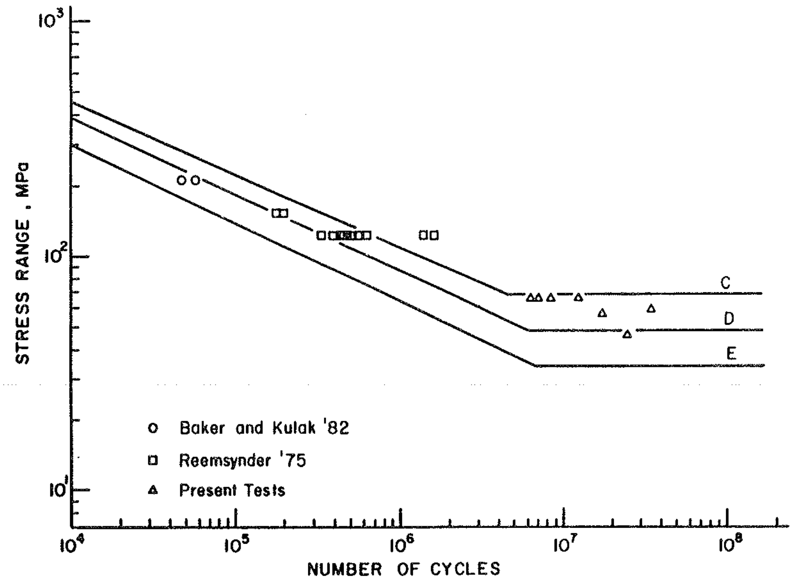

FIGURE 19 Fatigue resistance of riveted members.

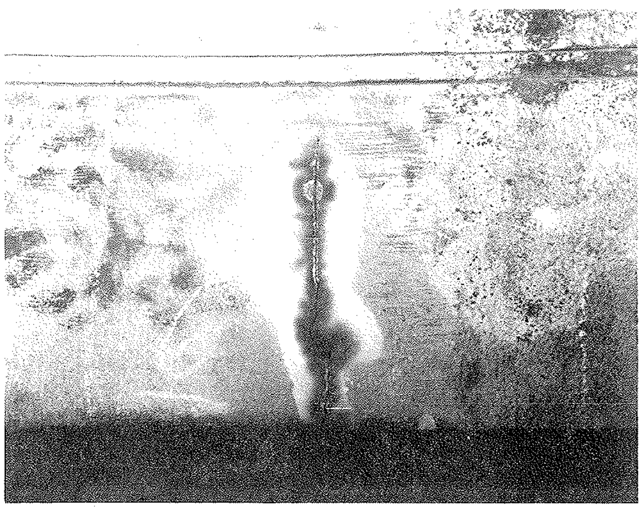

FIGURE 20 Fatigue crack in deteriorated compression flange of test beam 1.

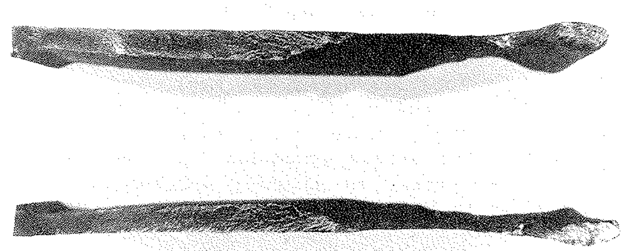

FIGURE 21 Fatigue crack surface in deteriorated compression flange of test beam 1.

REDUCED TEMPERATURE TEST

Objective and Procedure

A reduced temperature test was carried out on the third stringer. The objective was to evaluate the behavior of the member, given a fatigue crack, under low temperatures. More specifically, the objective was to determine if brittle fracture of a component would occur and how that would affect the behavior of the cross section. At the start of the test a fatigue crack extended from the top of the hole to the edge of the angle and from the bottom of the hole to the angle fillet, as shown in Figure 16. No cracking was observed in the other flange angle or in the web plate at that cross section.

The test procedure was as follows. The cyclic test was interrupted so that an insulating box of Styrofoam could be built around the section, which contained a grid of copper tubes with openings at regular distance through which liquid nitrogen could be injected into the closed space. By injecting the nitrogen, the temperature of the enclosed section of the stringer was decreased to approximately -40°C. The temperature was monitored by three temperature gauges attached to the web plate at mid-depth and to the top and bottom angles. After reaching the required temperature, the temperature was maintained by regulating the nitrogen flow.

With the cracked section being at this low temperature, a static load that corresponded to the maximum load of the repeated load cycle was applied. The cyclic loading was then resumed at a frequency of 260 cycles per minute. The crack front advanced in a stable, fatigue mode. The cyclic loading was continued for a period of approximately 0.5 hr at the reduced temperature. Then the crack was propagated at room temperature for an additional 12.5 to 25 mm. This procedure was then repeated.

Material Fracture Characteristics

An indication of the fracture characteristics of the material was obtained by performing a series of Charpy V-notch impact tests on 18 specimens taken from a tension flange angle of one of the stringers. Temperatures varied from -18° to 66°C. The results are shown in Figure 22.

A large variation in absorbed impact energy can be seen at test temperatures between 21° and 44°C.

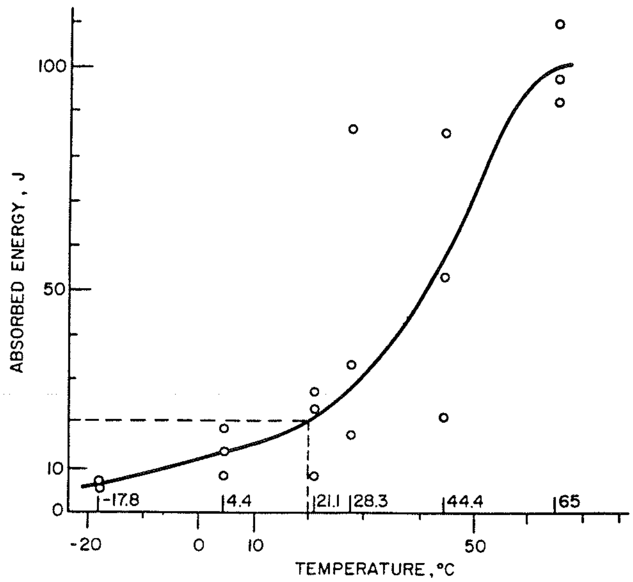

FIGURE 22 Charpy V-notch characteristics of flange angle.

The estimated 20.3 J transition temperature was about 20°C. Hence the material would satisfy the impact energy requirement for zone 2 of the AASHTO and AREA specifications.

Results of Reduced Temperature Test

The reduced temperature test was performed for a number of different crack lengths. These lengths and the number of stress cycles experienced under reduced temperature are given in Table 3.

TABLE 3 Reduced Temperature Test Results

Test	a_i (mm)	ΔN (cycles)
1	—[a]	19,000
2	27.9	0[b]
3	39.4	22,000
4	61.0	12,000
5	88.9	9,000
6	114.3	15,000
7	142.2	17,000
	152.4[c]	

Note: $T \simeq -40°C$. $(\sigma_{max})_g = 61$ MPa. $\dot{\epsilon} \simeq 2.6 \times 10^{-3}$ (S^{-1}). a_i = crack length at start of test measured at bottom of flange from angle corner to crack tip.

[a]Crack in fillet.

[b]Single static test.

[c]Test 7 discontinued; no fracture.

No unstable crack extension occurred in the cracked angle at any stage. During the process of crack extension the gross section stress increased by about 30 percent. It is apparent from the data in Table 3 that significant numbers of stress cycles were applied at each crack length increment.

The maximum estimated stress intensity factor during the test was 45 MPa$(m)^{1/2}$ if an edge crack was assumed from the heel of the angle. The test results suggest that a significant temperature shift is applicable because the dynamic fracture toughness at 21°C was this order of magnitude (8).

In addition, the results indicated that the frictional restraint between the cracked angle and the uncracked web, which was noted in the section on Fatigue Test Program, was beneficial because the crack opening was small. The associated stress intensity factor was probably smaller than predicated by assuming an edge crack.

No adverse behavior was experienced at the section when the flange angle cracked in two under the reduced temperature. The remaining resistance of the cross section was sufficient to carry the applied load.

SUMMARY AND CONCLUSIONS

The experimental studies carried out on four weathered and deteriorated riveted members provided information on the extreme life behavior of such members. In addition, they have also provided information on the behavior of severely corroded regions and their susceptibility to fatigue crack growth. The principal findings are as follows.

1. The extreme life fatigue resistance of the web flange riveted connection appears to be close to the category D fatigue limit. Several fatigue cracks were found to develop in the rivet details at stress ranges between 46 and 66 MPa after 8 to 30 million cycles.
2. The fatigue resistance of the corroded section was observed to vary between category E and C, depending on the severity of the corrosion and the loss of cross-sectional area. This degree of severity cannot be accounted for by considering only the loss of area. The four test results suggest that when the thickness of the outstanding flange angle is reduced until less than half remains, the proximity of the corroded area to the opposite surface reduces the fatigue resistance. Those stringers that had more than half of their flange thickness removed by corrosion initiated fatigue cracks near the category E resistance curve. When about half the thickness was available, initiation was observed to occur near the category C resistance curve. A lesser level of thickness did not result in crack initiation.
3. Severing of a component of the built-up section did not immediately impair the cyclic loading capacity of the members. Between 0.5 to 1 million cycles of a stress range of 62 to 69 MPa on the gross section were required before the load-carrying capacity was completely destroyed. Cracks formed slowly in the other angle and in the web plate. All four test beams exhibited redundant behavior once cracks developed that severed a flange angle.
4. Significant bond was observed to exist between the angles and web plate in their painted and corroded condition. This reduced the opening of the crack and extended the fatigue life.
5. Fatigue cracks that formed in the deteriorated legs of the compression flange were observed to arrest near the heel of the angle. None of these cracks affected the load-carrying capacity and the fatigue resistance of the stringers.
6. Reduced temperature tests at periodic intervals of extension of a crack grown from a rivet hole into the outstanding leg of an angle did not result in unstable crack growth. Even with 95 percent of the angle section cracked, the crack extension mode was stable.

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Stresses in Hanger Plates of Suspended Bridge Girders

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ABSTRACT

Hanger plates in suspended span bridges are briefly examined for in-plane bending. The hanger plates are designed as tension members; however, measurements indicate that bending occurs as well. This bending is produced by the relative rotation at the ends of the plate and is caused by the frictional bond between the hanger plate, pin, and girder web assembly. A finite-element model of the hanger plates indicates that a nonuniform stress distribution exists across the width of the plate at the pinhole. The maximum stress concentration factors were calculated to be 4.6 and 1.8 for 6.9 MPa (1 ksi) axial and bending stress, respectively. A fatigue strength analysis was conducted to determine the life of the hanger plates.

A frequently adopted arrangement for short- and medium-span steel bridges is the three-span structure with a suspended portion in the middle (see Figure 1). The suspended steel girders are usually connected to the overhanging girders by hinges and hanger plates. This arrangement renders the suspended girders in a simply supported condition.

The hanger plates, which are attached to the girders by pins (as illustrated in Figure 2), are primary bridge components. The plates are designed to undertake the reactions of the rocker supports of the suspended span. Because the pins of hanger plates are assumed to rotate freely, the hangers are assumed to sustain tension forces.

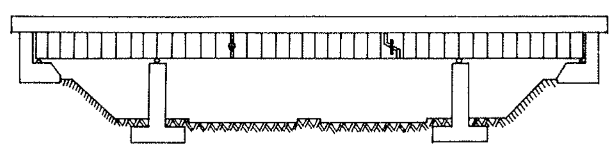

FIGURE 1 Three-span bridge with suspended girders.

In actual cases hanger plates may be subjected to in-plane bending because of friction at the pins, and to out-of-plane bending because of skewness of the bridge or other reasons. Broken hanger plates have been found in bridges (1). Some results of a brief study on in-plane bending of hanger plates are presented here.

MEASURED STRESSES

Because hanger plates are assumed to take tension forces, live-load stresses in hanger plates are expected to be tensile in nature. Measurements by electrical resistance strain gauges on hanger plates of two highway bridge girders indicated that strains varied from tension to compression, or vice-versa, as vehicles traversed the bridges (2).

Examples of recorded strain-time traces are shown in Figure 3. These results imply that the hanger plates were subjected to more than simple tension.