

Fatigue Cracking in Welded Steel Bridges

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A number of localized failures have developed in welded steel bridge components because of fatigue crack propagation, which in some instances has led to brittle fracture. Increases in the number of trucks and allowable weights has led to a rapid accumulation of loading cycles. A large number of structures experiencing cracking have welded details that have been identified as susceptible to fatigue crack propagation only after they were built. Oversimplification of member interactions and connection behavior allowed by design codes has resulted in a large number of cases of distortion-induced fatigue cracking. Many of the structurally deficient bridges are over 50 years old. Environmental corrosion has built up over time and has caused increasing amounts of damage. As the infrastructure ages and the costs for new construction escalate, maintenance and rehabilitation are becoming increasingly important to the continued operation of the transportation system. Existing bridge structures have been fabricated under a wide variety of practices and operate in a full spectrum of service conditions. As such, no single set of guidelines can adequately ensure the safety and reliability of all the existing structures. Periodic inspection for accumulated damage and deterioration is critical to acceptable operation of the nation's bridges. Continued technology transfer and education of engineers and qualified inspectors will increase the effectiveness of the inspection and maintenance process. Typical types of fatigue damage found in welded steel bridges are reviewed, specific examples are cited, common retrofit procedures are examined, and proper investigation practices are outlined.

Since the early 1960s, a number of localized failures have developed in steel bridge components because of fatigue crack growth that in some instances resulted in brittle fractures. Several hundred bridges are known to have developed one or more types of cracking. Often several types of fatigue cracking developed in a single bridge because different details existed on several types of structures (1-3).

The largest category of cracking is a result of unequal out-of-plane displacements, usually across a small unstiffened segment of girder web. Large numbers of distortion-induced cracks of this kind may form nearly simultaneously in a structural system because the cyclic stress is high and the number of cycles needed to produce cracking is relatively small. The problem of fatigue cracking induced by distortion has developed in a wide variety of bridge structures, including suspension bridges, two-girder floorbeam bridges, multiple-girder bridges, tied-arch bridges, and box girder bridges. In general, the cracks formed parallel to the primary tension from the applied loading and were not detrimental to the performance of the structure, provided they were discovered and retrofitted before turning perpendicular to the main stress field.

Another class of fatigue cracks are those related to connection restraint. Use of coped members such as stringers, floor beams, and diaphragms is common in bridge structures

in which members frame into one another. When rolled sections are used, these copes are often flame cut, resulting in residual tensile stresses along the cut edge. Often, the residual tensile stresses from the cutting operation approach the yield point of the parent material. With welded built-up members, terminating the flange outside the end connection is not unusual. Increasingly, cracks are being observed at these types of connections (1,3). A related type of cracking develops in the end connection angles. End rotation deforms the connection angle out-of-plane. This condition results in cracking of the angle, often at the fillet or the bolt-rivet restraint line. In cases where the angle is relatively thick, rivet or bolt heads may crack off.

Most of the remaining cracks resulted from details that were not known to have such a low fatigue resistance at the time of the original design, such as cover-plated beams, welded flange attachments, and web gusset plates.

LOW-FATIGUE-STRENGTH DETAILS

The possibility of fatigue cracks forming at the ends of welded cover plates was demonstrated at the AASHTO road test in the 1960s (4). Multibeam bridges subjected to relatively high stress range cycles (12 kg/in.^2) under controlled truck traffic experienced cracking after 500,000 vehicle crossings. In general, few crack details were known to exist until inspections revealed a cracked beam in Span 11 of the Yellow Mill Pond bridge in 1970 (Figure 1). Between 1970 and 1981, the Yellow Mill Pond multibeam structures in Bridgeport, Connecticut,

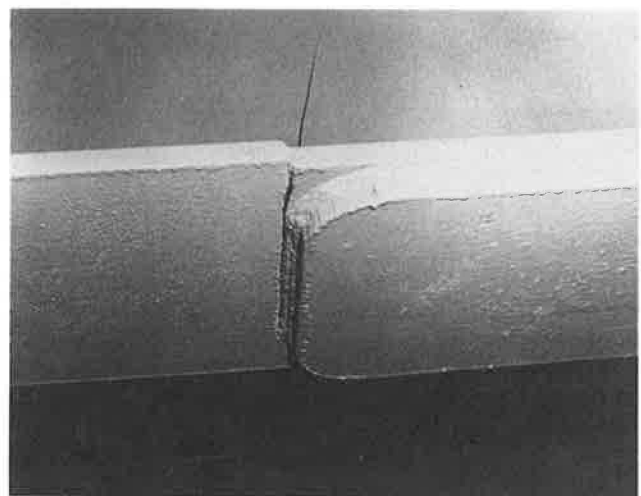


FIGURE 1 Fatigue crack at end of coverplate on Yellow Mill Pond bridge.

developed numerous fatigue cracks at the ends of cover plates (1). These cracks resulted from the high volume of truck traffic and the unanticipated low fatigue resistance of the large cover-plated beams (5).

Another low-fatigue-strength detail is the welded web gusset plate shown in Figure 2. These plates are particularly susceptible to crack growth when adjacent, but not attached, to transverse stiffeners and connection plates. A detrimental combination of cyclic stresses results from the expected in-plane deformation of the main girder and from unexpected out-of-plane web gap stress. This latter stress develops from the lateral forces that cause the gusset plate to twist and deform the unstiffened segment of the web between the gusset and transverse stiffener or connection plate.

Low-fatigue-strength details, such as cover-plated beams and welded web and flange gusset plates, should be avoided on new structures that will experience large numbers of stress cycles during their design life.

LARGE INITIAL DEFECTS

Large initial defects and cracks make up the next category of cracked members and components. In several cases, the defect resulted from poor quality welds that were produced before nondestructive test methods were well established. Many of these cracks occurred because the groove-welded component was considered a secondary member or attachment and no weld quality criteria were used nor were nondestructive test requirements imposed. Splices in longitudinal stiffeners frequently fall into this category.

In November 1973, an inspection revealed a large crack in the south fascia girder of the suspended span of the I-91 bridge over the Quinnipiac River (6). The bridge was approximately 9 years old when the crack was discovered. Figure 3 shows that the crack had propagated to the middepth of the web and had extended into the bottom flange at the time it was discovered. Detailed examinations of the fracture surface conducted during the course of the investigation revealed that the fracture began at the unfused butt weld in the longitudinal stiffener splice.

A similar condition has occurred when backing bars were used to make a groove weld between transverse stiffeners and

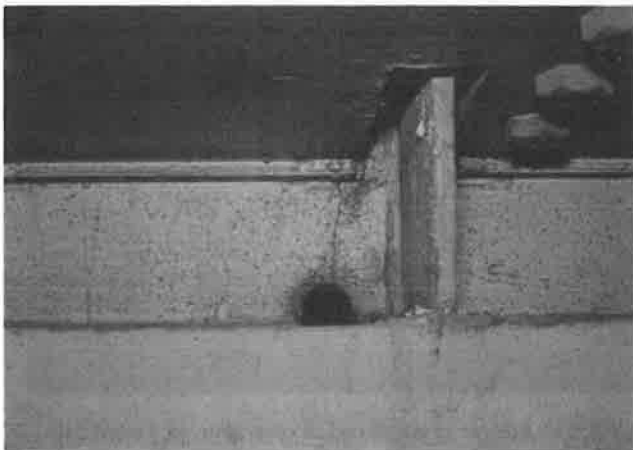


FIGURE 2 Fatigue crack at web gusset weld termination.



FIGURE 3 Crack originating from groove weld in longitudinal stiffener.

a lateral gusset plate. Lack of fusion usually exists adjacent to the girder web in the transverse groove welds. When the detail is such that the transverse welds intersect the longitudinal welds, a continuous path is created for the crack to enter the girder web. Longitudinal stiffeners on beams or girders of new structures should be fabricated with fully fused welds whose quality is verified by nondestructive examination.

Cracks that have developed in the web at lateral connection plates have generally started at intersecting welds. Horizontal gusset plates used to connect diaphragms and lateral bracing members to a longitudinal girder are often slotted around transverse stiffeners. The Lafayette Street bridge over the Mississippi River at St. Paul, Minnesota, was one of the first known structures to exhibit this type of cracking (7). The primary problem was a large defect in the weld attaching the lateral connection plate to the transverse stiffener. Because this weld was perpendicular to the primary stress field and also intersected the vertical welds attaching the stiffener to the web and the longitudinal welds attaching the gusset to the web, a path was provided into the girder web. Detailed studies indicated that the crack originated at a large lack-of-fusion discontinuity in the weld between the gusset plate and the transverse stiffener. Intersecting welds at the corner permitted the transverse crack to penetrate into the girder web.

Development of these cracks indicates that considerable care must be exercised when web gusset plates are used for bracing or lateral systems.

Intersecting welds should be retrofitted and the local area carefully inspected for cracks. A related lack-of-fusion type defect and cracking, which has been among the most severe encountered, occurred at details where a plate component was inserted through an opening cut into a girder web. The resulting detail was usually welded into place with either fillet or groove welds. In either case, large cracks in the girder web resulted at the edge of the flange plate where short vertical weld lengths contained large unfused areas. This feature was illustrated by a large crack in one of the steel box bents supporting the elevated track of Chicago's mass transit Dan Ryan line. Discovered in January 1978, subsequent inspection showed that box adjacent bents were also cracked (8). Initial field examination of the fractures indicated that all of the cracks started at the junction of the plate girder flange tip to the box side plate, as shown in Figure 4. All three cracks completely severed the bottom flange of the box girders and the webs.

Extreme care must be exercised when flange, web, or diaphragm plates pass through the web of an intersecting member. Because it is nearly impossible to provide full fusion welds at the penetrating flange tips, installation of open holes are desirable at the ends of the slots to ensure adequate fatigue resistance. Figure 5 shows a retrofitted detail.

OUT-OF-PLANE DISTORTION

Several hundred bridges have developed fatigue cracks as a result of out-of-plane distortion in the small, unstiffened seg-

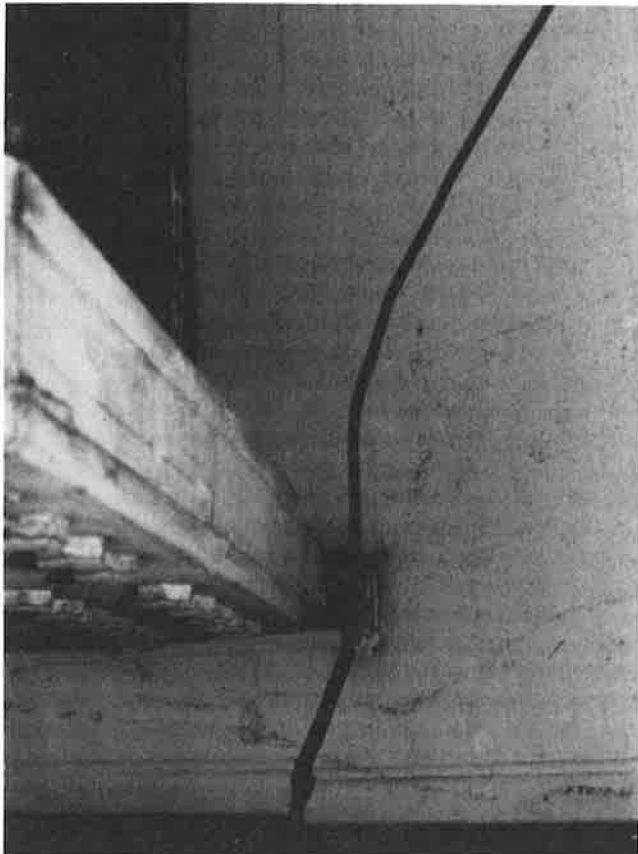


FIGURE 4 Crack in box girder web at intersecting flange tip.

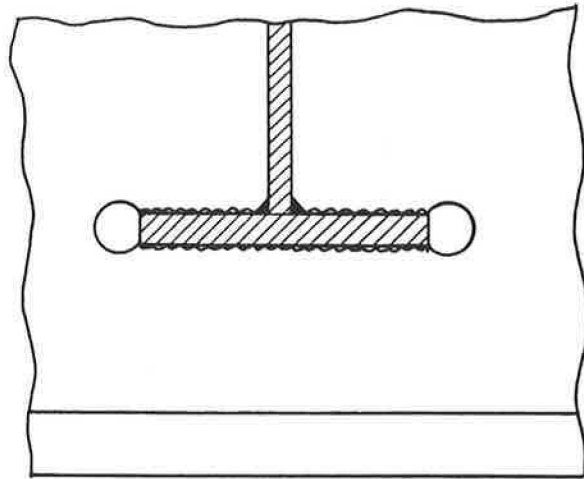


FIGURE 5 Flange plate passing through intersecting web with holes at flange tip.

ment of a girder web (1-3). When distortion-induced cracking develops in a bridge, usually large numbers of cracks form before correction action is taken because the cyclic stresses are usually high. As a result, many cracks form nearly simultaneously in the structural system. Early detection of this condition would permit other potential crack locations to be identified and retrofitted before significant damage develops elsewhere.

The problem of displacement-induced fatigue cracking has developed in many types of bridges, including truss, suspension, two-girder floorbeam, multibeam, tied-arch, and box girder bridges. Cracks have usually formed in planes parallel to the stresses from loading and have not been detrimental to the performance of the structure providing they were discovered and retrofitted before turning perpendicular to the applied stresses from loads. In some structures, the cracks stopped in low-stress areas and thus have served to relieve local restraint conditions.

Conditions that favor the formation of web gap cracks have most often developed because of the desire to avoid welding transverse connection plates to the tension flange. Figure 6 shows a typical condition that exists in floor beam girder bridges. As the floor beam rotates under traffic loading, the segment of girder web is pulled out-of-plane, producing a large stress gradient in the web gap. Such large cyclic stresses will result in fatigue cracking in a relatively small number of load cycles.

Other examples of web gap fatigue cracking abound. Examples include diaphragm connection plates in multibeam bridges, internal diaphragms in box and tie girder structures, and lateral connection plates that are welded to girder webs but cut around transverse stiffeners.

Current AASHTO specifications require new designs to provide a positive attachment between transverse connection plates for diaphragms and X frames and both girder flanges (9). This attachment decreases the web gap distortion to acceptable levels, providing web gaps at the copes are at least 2 in. or four times the web plate thickness, whichever is larger. Structures without positive attachment will eventually have to be retrofitted by providing this corrective measure.

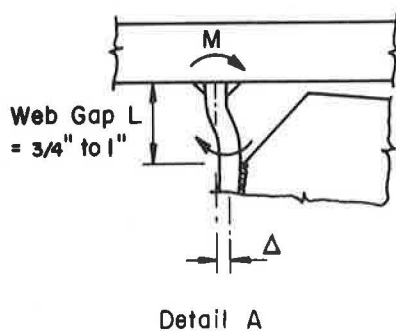
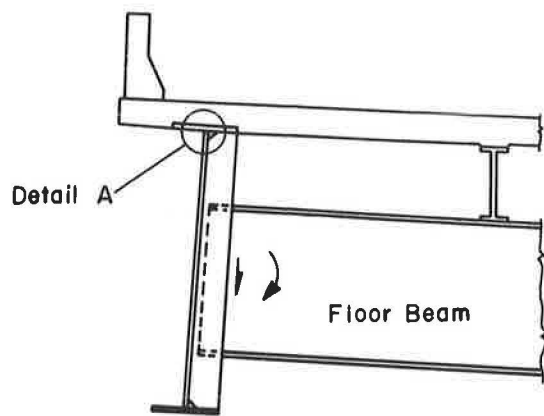


FIGURE 6 Schematic of distortion in web gap at end of transverse connection plate.

RESTRAINT, COPES, AND FLANGE TERMINATIONS

Increasingly, cracks are developing at coped flanges and welded flange terminations. All simple connections provide some degree of end restraint (1-3). When rolled sections are coped by flame cutting, the burned edge has tensile residual stresses that approach the yield point. Because one or both flanges are removed, the web plate has a low section modulus compared with the member section. This condition can increase the bending stress in the web by 200 to 300 percent.

A similar condition results when the flanges of welded beams are terminated short of their end connections. This practice is typical in many floor beam girder bridges. Figure 7 shows a crack that developed at the end of a floor beam flange of the Woodrow Wilson bridge over the Potomac River. The fatigue crack at the cope of the floor beam resulted from a reduction of in-plane bending resistance, low-fatigue-strength detail created by termination of the flange plate welded to the web, and end restraint of the shear connection. Cracks can form at the top and bottom flanges because of the residual stresses at the flange-to-web weld termination and because of construction-induced stresses.

Coped flanges are also susceptible to distortion in certain types of applications. For example, at expansion joints it is not unusual to cope the top flange to accommodate the expansion joint. Lateral movement may develop between adjacent spans and produce large out-of-plane web bending stresses at



FIGURE 7 Fatigue crack in floorbeam web at end of welded flange.

the cope. Coped members need continued observation for cracking.

GUIDELINES FOR INVESTIGATING THE CAUSES OF CRACKING

If a crack is detected in a bridge component, the type of structure, crack location, and characteristics of the crack must be considered before steps can be taken to evaluate the causes of the cracking.

Of primary importance is the significance of the crack for the load-carrying capacity of the bridge. If the crack is moving perpendicularly to the main tension from the applied loads, holes should be drilled at the crack ends so as to blunt the tips and arrest growth. Installation of either a 3/4- or 7/8-in. bolt should be accommodated by the hole. Valuable information can be obtained from the region and the holes should be made with a hole saw so that the crack tips are available for subsequent study. After the holes are drilled, the hole surface should be checked by dye penetrant or some equivalent method to ensure that the crack tip has been removed. Usually, the drilling of holes can be considered only a temporary retrofit pending permanent repair. In addition, field personnel should spray-coat the crack surfaces with a clear acrylic lacquer so that the surface features can be preserved.

Removal of Crack Segment

In order to investigate the cause of the crack, part of the cracked segment is usually removed. However, before removal, data need to be acquired on size, location, and orientation of the crack. These data should be gathered by

- Detailed sketches showing crack location, dimensions of the crack, and orientation with regard to the primary stresses in the member, and
- Photographs showing visible crack conditions and location of the crack relative to the detail at which it is formed.

After documentation, a portion of the crack can be removed to permit evaluating the causes of the cracking. Generally,

two types of samples can be removed. If material characteristics need to be determined, then a larger portion can be removed. This method will allow for composition and material property determination. Often, only one crack surface needs to be removed, although removal of both surfaces is preferable.

During removal, steps should be taken to ensure that crack surfaces do not come into contact; otherwise, damage to surface features is likely and important information may be destroyed.

In a number of cases, much smaller cracks exist and removal of large pieces is not necessary. Core samples can be used to remove all or a major part of the crack. Figure 8 shows the polished surface of a core sample removed from a groove weld. The core was positioned near the end of the crack to see if fatigue crack growth had occurred. Figure 9 shows a core removed from a cover-plate termination where a partly through crack was found at a weld toe.

Data Gathering

In order to assist with the investigation, evaluation, and retrofitting of cracked components and members, the following information should be assembled and documented:

1. Date the crack was first detected;
2. Design stress conditions normal to the crack or detail, as applicable;
3. Average daily truck traffic (ADTT) estimate from the most recent traffic survey and where possible, an estimate of the ADTT for the period of time the bridge has been in service;
4. Estimated frequency and magnitude of any overloads, by permit or other basis;
5. An indication of the approach and deck conditions to permit an assessment of impact;
6. Yield point of the cracked plate or rolled section, as provided by mill reports or tests;
7. Charpy V-notch impact values and test temperatures from mill reports or other sources;
8. Minimum temperature that the structure has experienced during service life;
9. Minimum temperature that the structure experienced during the year preceding discovery of the crack;
10. An indication as to whether the cracked member was struck by a vehicle;
11. If the crack formed at a groove weld, determination of how these welds were inspected when the member was fabricated and all available inspection reports and a radiograph, if possible;
12. If the crack formed at a welded detail, identification of the applicable fatigue design classification according to AASHTO; and
13. Details of weld procedures, qualification tests, and fabrication procedures for the failed component.

Material Tests

Material in the cracked component will usually have documentation available on chemical composition and mechanical

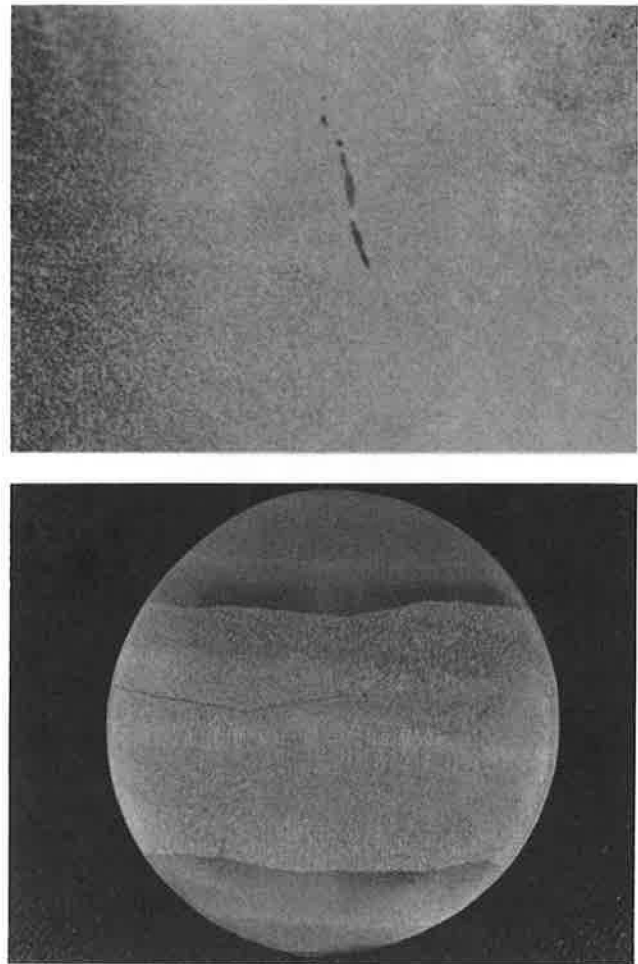


FIGURE 8 Crack in groove weld (*top*) and core sample removed from groove weld (*bottom*).

properties. However, fracture toughness values will often be unavailable. If material is available, extensive supplemental tests can be conducted. These tests can include standard Charpy V-notch tests. From 12 to 18 specimens should be prepared and tested at several temperature increments. At least three specimens should be tested at each temperature. Where limited amounts of material are available, tests should be carried out at AASHTO specified temperature.

Other tests, such as mechanical and chemical properties, may be useful but are not as essential because mill reports are often available.

For segments of rapid fractures, an estimate of the expected toughness can sometimes be made if the stress, crack size, and geometric conditions are available. This information will permit use of simple fracture mechanics models and allow an estimate of the stress intensity factor.

Metallographic Examination

Determination of weld profile and plate microstructure is often desirable. Investigation of a segment not in the proximity of the crack can reveal profile, heat-affected zone, and weld passes. In addition, information on plate microstructure can be obtained.

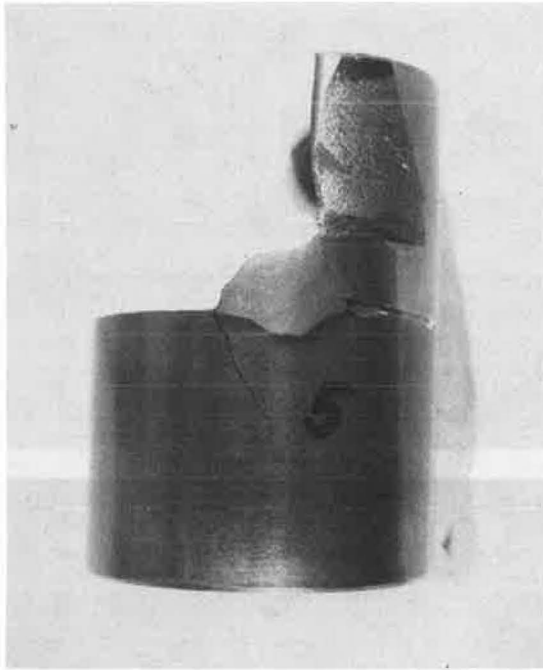


FIGURE 9 Core sample from end of coverplate showing crack at fillet weld toe.

If a crack is associated with a weld and a core is used to obtain a segment of the crack and adjacent material, a detailed examination should be carried out on the surfaces of the core before it is broken apart and cut into segments. Surfaces of the core should be polished and etched to reveal the location of the crack and nature of the surrounding material. Before polishing and etching, the crack should be sealed with an inert wax on the core surface to prevent damage to the crack surface.

Generally, microscopic examinations of core or plate surfaces are made at magnifications between 1 and 100 times normal. Suitable photographs of the exposed features should be obtained. If crack surfaces are not exposed during fracture, cutting the core or sample into segments will be necessary. Surfaces should be exposed only after the piece has been cooled in a liquid nitrogen bath, so the sample can be readily broken apart. Care should be exercised in exposing the crack so that saw cuts do not destroy the crack tips.

Careful attention to the saw cut surfaces, which can be polished and etched, can also reveal weld repair passes, method of fabrication, and other characteristics that may be important in assessing the fracture. Figure 10 shows a crack in a longitudinal box corner weld that has been removed with a hole saw. When the crack surface was exposed, the crack shown in Figure 11 was revealed.

When crack surfaces are exposed, polishing and etching a saw-cut plane parallel to the crack surface will generally be desirable. This plane should be relatively close to the crack surface, normally within $\frac{1}{2}$ to 1 in. Weld repairs, secondary cracking, and other features may be observed. Figure 12 shows the polished and etched surface of a plane parallel to the crack surface shown in Figure 11.

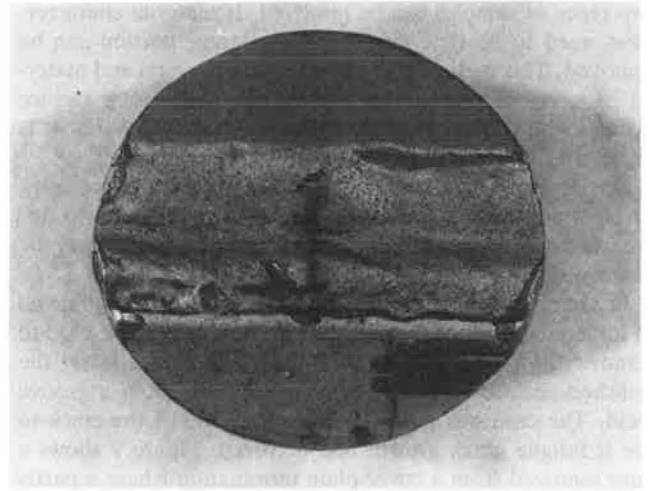


FIGURE 10 Core containing crack from box corner weld.

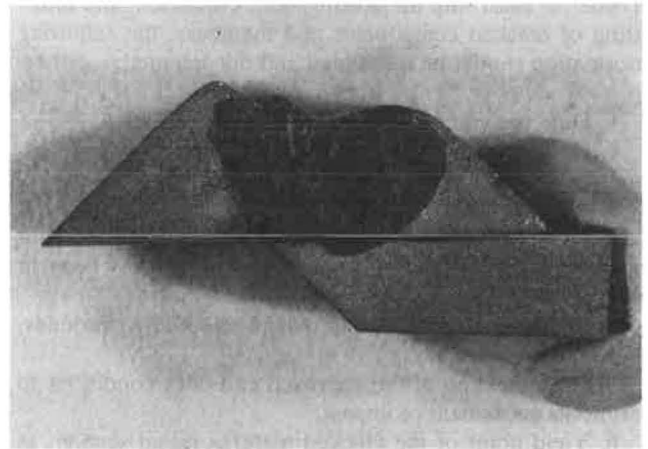


FIGURE 11 Exposed crack surface from box corner weld.

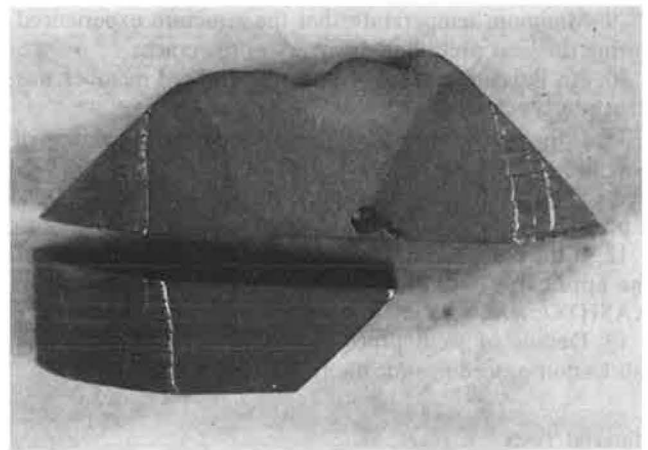


FIGURE 12 Polished and etched plate behind crack surface.

Crack Surface Examination

Where possible, a direct visual examination of the fracture surface is often beneficial in defining the cause of cracking. However, severe corrosion can destroy fatigue and fracture surface markings, as shown in Figure 11. Adequate photographic documentation should be obtained before cutting the surface into segments or cleaning off corrosion.

Normally, fatigue crack growth surfaces will be flat and smooth. Brittle fracture surfaces do not often exhibit the flat, smooth characteristics of fatigue cracks. Chevron-type markings are often apparent on the surfaces of bridge steel that has rapidly fractured. These chevrons point back to the origin of the fracture. Often, shear lips will be apparent on the surfaces of a brittle fracture, examination of which can assist in the estimation of material toughness.

Once the as-received surfaces are examined and photographed, the clear lacquer protective coating and loose corrosion can be removed with an organic solvent such as acetone. This process will often reveal surface features that were obscured. A soft toothbrush can be used to remove the coating and any loose corrosion.

Considerable success has been achieved in crack surface examinations by stripping the areas clean using solvent-softened replica tape. The tape is pressed against the surface and removed after hardening. Oxides and other corrosion will be stripped away. If this procedure is repeated, most of the debris can be eventually cleaned from the crack surface. Final replicas can be examined with a transmission electron microscope because the replica will carry a reverse impression of the crack surface.

Areas of fatigue crack growth may exhibit striations on the replica surface (8). These marks are a series of lines or bands that fatigue crack growth exhibits as the crack advances. Other modes can also be verified as cleavage facets may be apparent. Obviously, use of either a scanning or transmission electron microscope requires experienced personnel. These instruments are used to provide a microscopic examination of selected areas of the crack surface.

On the basis of visual observations of the crack surface, establishing whether or not there is evidence of a large initial flaw or crack may be possible. Of particular focus should be evidence of fatigue cracking. Such observations can be used to decide if the cracking can be understood in terms of final flaw size, stress, expected toughness, and so forth. Estimates

of stress range and cumulative cycles can be used to decide whether or not the observed crack extension is consistent with flaw size and crack growth kinetic data.

CONCLUSION

Once fatigue cracks are found, investigating the cause of the failure and retrofitting the structure is important. A majority of cracks in welded steel bridges may be a result of low-strength details, large initial defects because of geometric conditions and fabrication, distortion, or restraint problems. Investigation may reveal a consistent and logical answer that can then be used to assess damage in other locations and to develop an economical and effective fix. If an understanding is not achieved, consideration should be given to seeking expert help.

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