High Cycle Fatigue of Welded Bridge Details

INSPECTING STEEL BRIDGES FOR FATIGUE DAMAGE

by

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Report No. 386-15 (81)
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CHAPTER 1: INTRODUCTION

Sometimes cracks develop in bridge structures as a result of the repeated loading to which they are subjected. The primary factors leading to the development of such cracks are the frequency of truck traffic and the age of the bridge, the magnitude of stress range (live load and impact), the type of detail, the quality of the fabricated detail, and the material fracture toughness.

Generally the stress fluctuation, frequency, and type of detail are the most important factors. Welded structures tend to be less forgiving of small weld discontinuities than riveted structures, because welds are more sensitive to repeated stresses, and once cracking starts to develop, it can destroy the member as a result of the continuous path provided by the welded connections. Riveted construction generally utilizes multiple plate or shape components, and when one component cracks the crack does not directly enter into the other components.

Cracking has developed more frequently in welded bridge structures, because of flaws that escaped detection, the use of details more severe than assumed in the design, and as a result of secondary and displacement induced stresses. Except for secondary and displacement induced stresses, comparable conditions do not develop in riveted and bolted bridge structures. As a result, fatigue cracking is not likely to develop in riveted and bolted highway bridge members, because the repeated loads will not produce stress cycles large enough to cause crack growth. Excessive corrosion or secondary and displacement induced stresses can lead to cracking in riveted, bolted and welded structures.

The material fracture toughness will generally control the size of the largest crack that can be tolerated before a fracture of the member
or component will occur. The size of the crack will depend on the detail type and the stress level to which the component is subjected. Any crack found should be immediately reported to the bridge engineer, so that its severity can be assessed. Cracks that are in line with the direction of stress are usually not very serious. However, those that are perpendicular to the direction of stress are very critical.

There are several manuals and other sources of information (see References) available for the purpose of inspecting and rating bridge structures. These provide recommended ways to inspect and assess bridge structures, so that uniformity in the procedures and policies of determining the physical condition and maintenance needs are achieved. The sources include FHWA - Bridge Inspector's Training Manual\(^1\), FHWA - Steel Bridge Inspection Using Acoustic Crack Detector and Magnetic Crack Definer\(^2\), AASHTO - Manual for Maintenance Inspection of Bridges\(^3\), OECD - Bridge Inspection\(^4\). It is not the intent of this manual to replace these documents. This manual is intended to supplement existing inspection and rating manuals and to provide the field inspector with a readily available handbook which can be used on the site. It provides photographs and appropriate line drawings of details where fatigue cracks have been observed in bridge structures. These can be used to aid the bridge inspector in what to look for.

The most used inspection method has always been the relatively elementary visual inspection. Most cracks in steel bridges have been first detected by visual inspection. Thereafter, other nondestructive inspection methods such as dye penetrant, magnetic particle, ultrasonics, eddy current or radiography have been used to inspect comparable locations. More sophisticated visual observations can also be employed using a magnifying unit with the paint blast cleaned from the suspect area.
CHAPTER 2: CLASSIFICATION OF BRIDGE DETAILS

Members and details on bridges have been arranged into five basic classes for fatigue design purposes\(^5\). A number of typical bridge details were also classified in Ref. 6, and most of that information is contained in this report. The details shown in Figs. 2 to 27 are illustrative of situations which might develop fatigue cracks. In each figure, one or more classifications are indicated, depending on location in the member and direction of the applied stress shown by the double-ended arrows. The double-ended arrows shown in Figs. 2 to 27 are located at the critically stressed point for the detail stress category in the steel member adjacent to the weld. A more detailed description of design criteria and development is given in Ref. 7.

![Diagram of Welds and Joints]

**Fig. 1** Types of Joints and Welds

Welds are generally placed into two types of categories—fillet welds and groove welds. Figure 1 is a chart that shows schematically several common welds and also gives the symbols which are shown on the design and fabrication drawings for these welds. When flange or web groove welds are produced, the weld metal connecting the plates results in a build-up of
weld metal that is called weld reinforcement. In most flange tension
welds this reinforcement is ground off to produce a flat surface which
improves fatigue resistance and nondestructive inspection.

Category A

Structural components and joints that fall under Category A consist
of plain material with rolled or cleaned surfaces. This "base metal" con-
dition is for rolled shapes and plates without a welded detail. It provides
the maximum fatigue strength of any structural detail. Fatigue studies on
rolled beams have shown that the fatigue cracks originate from the surface
of the tension flange. Studies on plate specimens have also shown that
the fatigue cracks originate from the surface on the specimen. The point
of crack initiation is a microdiscontinuity at the surface of the material.
Generally, these occur at locally adhered mill scale or other similar sur-
face imperfections. If the surface discontinuity is at the edge of a
flange or plate, it is slightly more severe. The slight surface depres-
sions from rolled-in mill scale are not so detrimental to fatigue strength
as are sharp surface edge indentations and other examples, such as changes
in section at copes or blocked flanges (see Fig. 39) and any welded at-
tachment. In general, it is not necessary to look for cracks in the main
material, unless it is damaged or has one of the adverse conditions described.

Category B

Stress Category B applies to a variety of welded beam details. Typi-
cal bridge details that fall into this category are shown in Figs. 2 to 6.
They include groove-welded joints with the reinforcement removed and the weld soundness established by nondestructive inspection. Flange and web butt welds are illustrated in Figs. 2 and 5. Straight tapered transition details with slopes not greater than one to two and one-half in either the thickness or width are shown in Figs. 3 and 4.

Groove welds at transitions in width sometimes use a curved radius transition. If the reinforcement has been removed and the weld soundness established by nondestructive inspection, Category B is also applicable. Typical examples of curved transitions at gusset plates and at flange thickness and width transitions are shown in Figs. 4, 5, and 6.

Longitudinal welds in built-up plates or shapes that are continuous and parallel to the direction of the applied stress field also fall into stress Category B. This includes the groove and fillet web-to-flange welds in welded built-up girders shown in Figs. 2 to 4. As long as the longitudinal weld is continuous, the primary factors influencing the fatigue strength are the size and location of the imperfections that exist in the weldment. If crack growth occurs, it results in embedded disk-like cracks that originate in the weldment at these flaws.

Stress Category B is also applicable to continuous longitudinal welds for attachments, gussets, and cover plates. Only the weld-toe termination at the end of the longitudinal weldment causes a substantial decrease in fatigue strength. Examples of the portions of the welds for which Category B is applicable are shown in Figs. 3, 5, and 6.
Fig. 2 Groove Welded Web and Flange Butt Joints
- Category B Weld Reinforcement Removed

Fig. 3 Groove Welded Flange Butt Joints at Tapered Transitions - Category B Weld Reinforcement Removed
Fig. 4 Groove Welded Flange Butt Joints at Curved Radius and Tapered Transitions - Category B Weld Reinforcement Removed

Fig. 5 Groove Welded Gusset with Curved Radius Transition and Groove Welded Butt Joints in Box Member - Category B
Fig. 6 Flange and Web Attachments with Curved Radius Transition - Category B

**Category C**

Stress Category C is primarily applicable to stiffeners and short attachments. These structural elements exhibit fatigue crack growth from their terminating weld toes. In addition, groove-welded connections perpendicular to the applied stress and having the reinforcement left in place and the weld soundness established by nondestructive inspection also fall into Category C. Examples of various groove-welded splices with the reinforcement in place are given in Figs. 7 to 9. Groove welds that are parallel to the applied stresses are governed by Category B and are comparable to continuous fillet welds.

A number of stiffener details that fall into Category C are shown in Fig. 10. As shown in Fig. 10, welds attaching transverse stiffeners to either the web or the flange are in the same stress category. Category C is also applicable to vertical gussets attached to the web, as indicated in Fig. 11, and to short flange attachments less than 2 in. long. A stud shear connector in the negative moment region reduces the strength of the flange to Category C. In all of these cases the critical point is in the base metal adjacent to the weld.
Fig. 7 Groove Welded Web and Flange Butt Joints - Category C, As-Welded with Reinforcement

Fig. 8 Groove Welded Flange Butt Joints at Tapered Transitions - Category C, As-Welded with Reinforcement
Fig. 9 Groove Welded Flange Butt Joints at Curved Radius or Tapered Transition - Category C, As-Welded with Reinforcement

Fig. 10 Transverse Stiffeners Welded to the Web (and Flange), Category C
Fig. 11 Transverse Web Gusset Plate - Category C

If attachments are fillet or groove welded to the edge of the flange, as indicated in Fig. 12, the stress category at the weld termination is dependent on the transition radius. If the transition radius is sharp (R nearly zero), a fillet or groove weld provides a right angle attachment similar to the flange attachment in Fig. 12, and stress Category E is applicable. If the transition radius is less than 24 in., but greater than 6 in., Category C is applicable, as shown in Fig. 12. The improvement in fatigue strength is caused by a decrease in the stress concentration condition with an increase in the transition radius R and the removal of the weld ends by grinding. The categories shown in Fig. 12 also apply to web
attachments. In most bracket attachments the transverse forces acting on the bracket nearly cancel each other and the weld attaching the bracket to the web or flange is primarily subjected to shear. Such weldments can usually be treated as other longitudinal welds.

![Diagram of gusset plate with welds and radius transitions](image)

Fig. 12 Web or Flange Gusset Plate with Radius Transitions
Ground Smooth 6" < R < 24 in. – Category C

**Category D**

Category D provides an intermediate level between short fillet or groove welded attachments and longer attachments (such as cover plates). If the attachment length in the direction of the applied stress is greater than 2 in. but less than 4 in., the controlling stress at the weld end is given by Category D (see Figs. 13 and 14).

Other possible applications of stress Category D would be short plate connections, or channel-type shear connections that are between 2 and 4 in. long.
Fig. 13 Short Web Gusset Plate $L < 4$ in. - Category D

Fig. 14 Short Flange Gusset Plate $L \leq 4$ in. - Category D
Category E and E'

A wide class of fillet and groove welded details are covered by stress Category E. It provides the lower bound fatigue strength of welded details and has been defined by experimental work on cover-plated beams and other comparable details. Studies on attachments have shown that as the attachment length exceeds 4 in. the fatigue strength rapidly approaches the lower bound cover-plated beam condition. The critical point is at the end of the longitudinal weld. Crack growth originates at microdiscontinuities at the weld toe and continues perpendicular to the stress field into the plate thickness.

A number of commonly used details have fatigue strengths comparable to the cover-plated beam. Included are the weld toe terminations of longitudinal stiffeners, as shown in Fig. 15, and of gusset stiffeners, as shown in Fig. 16. Although Category E applies at the end of a longitudinal stiffener, stress Category B is applicable at points away from the weld end (see Fig. 3). When attachments to the web or flange are greater than 4 in. in length in the direction of the applied stress (see Figs. 17 and 18), stress Category E is applicable. Category E is also applicable to gussets that are attached to flanges (see Fig. 19) or to other structural elements, as shown in Fig. 20.
Fig. 15 Termination of Longitudinal Stiffeners – Category E

Fig. 16 End of Stiffeners Welded to Gusset Plates – Category E
Fig. 19 Flange Gusset Plate - Category E

Fig. 20 Gusset Welded to Bracing Member - Category E

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A variety of cover-plated beam details with tapered or square ends are shown in Figs. 21, 22, and 23. Stress Category E or E' applies to the base metal at the ends of the cover plates. When the cover plate thickness exceeds 0.8 in., Category E' is applicable. The continuous longitudinal fillet welds along the edges of the cover plate are covered by stress Category B.

When discontinuous or intermittent fillet welds are used (see Fig. 24), stress Category E is applicable at the end of each intermittent weld. This is a conservative treatment of intermittent welds because of the lack of test data. It is probable that higher fatigue strengths exist when the intermittent welds are continuing (i.e., web-to-flange connections) so that the connected plates are about equally strained.

Beam-girder intersections often result in low fatigue strength details. This is shown in Figs. 25 to 27. Groove welded flange-to-web or flange-to-flange joints can cause substantial reductions in the allowable bending stress range. Because the length of the weld connecting the secondary beam to the web or flange will always be greater than 4 in., Category E or E' applies. Recent research has indicated that when flanges thicker than 1 in. frame into a girder web, Category E' governs the fatigue strength for both fillet welded and groove welded connections. Even lower fatigue resistance is possible if the stronger flange pierces through the girder web and is welded. If a continuous web-to-flange weld abruptly terminates, as shown in Fig. 25, Category E is also applicable. A more critical condition is provided by the flange welds in Fig. 27.
Fig. 21 Square Ended Cover Plates Narrower than the Girder Flange - Category E or E'

Fig. 22 Cover Plates Narrower than the Girder Flange with Tapered Ends - Category E or E'
Fig. 23  Cover Plates Wider than the Girder Flange with End Welds - Category E or E'

Fig. 24  Intermittent Fillet Welds - Category E
Fig. 25 Stringer Framing into Girder - Category E or E' 

Fig. 26 Stringer Framing into Girder Web - Category E or E'
Fig. 27  Stringer Framing into Girder Flange
   - Category E or E'
CHAPTER 3: DETAILS MOST SUSCEPTIBLE TO FATIGUE

Of all of the typical design details discussed in Chapter 2 those that fall into stress Category E or E' (see Figs. 15 to 27) are the most susceptible to fatigue crack growth in steel bridges. Critical cracks will only exist in tension areas. When details are located in compression stress regions and no possibility of stress reversal exists, there is no fatigue problem. Under these conditions any crack growth will be confined to the residual tensile zone unless out-of-plane deformations occur.

The usual and most reliable sign of fatigue cracks are the oxide or rust strains that develop after the paint film has cracked. This is readily apparent in nearly all the photographic examples given hereafter in Figs. 28 to 48. Experience has shown that cracks have generally propagated in depth to between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by blast cleaning the suspect area. Furthermore, at weld terminations, very small cracks are very difficult to detect by other nondestructive inspection techniques, unless the crack depth is between 1/8 in. and 1/4 in.

There are three major factors that result in the fatigue cracking of the details shown in Figs. 28, 29, 31, 32, and 33 - the type of detail, stress range and the frequency of occurrence. Hence, bridges on roads with large numbers of heavy trucks are most susceptible to fatigue cracking if they have low fatigue strength details. Many of these details were
not recognized as being fatigue critical when the structure was designed, and they were incorporated into it.

Even less fatigue resistance is possible if large discontinuities or cracks are fabricated into the structure at the detail. This is discussed in further detail in Chapter 4. These discontinuities may be the result of poor fabrication and quality control or may result because the consequences of using a specific detail in certain orientations was not understood at the time of the design.

3.1 Cracks at Ends of Cover Plates

Cover-plated beam bridges that have experienced large numbers of stress cycles have developed fatigue cracks. The fatigue strength of beams with flange thickness greater than 0.8 in. has the lowest fatigue Category E'.

As illustrated in Fig. 28, the fatigue crack will form in the rolled section at the toe of the fillet weld where it attaches the cover plate termination to the rolled section. Figure 28a shows the cracked paint film just shortly after it was broken. Not much oxide is apparent. In this particular beam, the crack was found to be about 3 in. long after the paint was blast cleaned from the weld toe.

Figure 28b shows a fatigue crack that is nearly the full width of the cover plate. The oxide film which is usually a sure indication of a fatigue crack is readily visible. A crack this long will be at least halfway through the beam flange.
Fig. 28 Schematic of Cover-Plated Beam Shows Location of Fatigue Crack

Fig. 28a Fatigue Crack that has just Resulted in Cracking of Paint Film
3.2 Gusset Welded to Transverse Stiffener

Category E attachments need not be welded to girder flanges to develop fatigue cracks. Figure 29 shows a cross-frame with gusset plates welded to the transverse stiffeners or connection plates. The beam action of the transverse connection plate resulted in high stress range conditions. This caused the fatigue crack to form at the end of the groove weld which connected the gusset plate to the transverse connection plate.

A thin oxide line can be seen in the paint circled area in Fig. 29a. Close attention should be given any Category E detail on cross-frame members. Often the stresses in the cross-frames and diaphragms are difficult to calculate. Hence the inspector may not be alerted to critical details by the engineer.

A more visible crack can be seen in Fig. 29b which is at a different location. A hole has been temporarily drilled at the end of the crack to arrest the propagation pending repair.
Fig. 29 Schematic of Transverse Cross-Frame with Gusset Plates Welded to Transverse Stiffeners (Connection Plates)

Fig. 29a Fatigue Crack showing as Thin Oxide Line at End of Gusset Plate

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Fig. 29b Large Fatigue Crack in Connection Plate With Hole Drilled at Crack Tip

3.3 Groove Welded Flange Transition with Reinforcement

Figure 30 shows the flange tip of a groove welded flange transition. This is a Category C condition and would not normally be expected to experience fatigue damage. At this particular joint crack growth has developed at an electroslag weldment. The crack has penetrated about 1/2 in. into the flange from the "toe" of the reinforcement (i.e. the weld passes that have not been ground smooth).

At the time this crack was found the paint film had not cracked. The crack was only detected after the paint was removed from the flange tip, and the area ground and polished. Figure 30a shows a photograph of the flange tip. The faint irregularly shaped crack is very tight and not readily apparent.

-28-
Fig. 30 Schematic of Butt Weld Transition

Fig. 30a Crack in Ground Edge of Flange at Electroslag Weldment
3.4 Beam Column Connections of Box Sections

When steel frames are used to provide piers, intersecting beams and columns result in rigid frame connections as illustrated in Fig. 31. This type of construction is susceptible to two potential types of crack problems. Since the flange of either the column or girder must be interrupted, it is necessary to weld the flange of one perpendicular to the other. This results in a highly restrained joint and can result in lamellar tearing of the through plate as can be seen in Figs. 31a and 31b.

The top flange of the girder is in tension at the intersection. The high stress concentration at the groove weld toe is a Category E design condition. Because of edge effects the corners are more susceptible to fatigue cracking. The cracking shown in Fig. 31a represents the most likely type of condition that will develop at this type of connection.

The crack shown in Fig. 31b is not serious, because it is subjected to compression forces. Nevertheless, all cracks should be brought to the engineer's attention.
Fig. 31 Beam-Column Connection at Rigid Frame Pier
Fig. 31a  Cracking at Top Tension Flange Weld Toe Connecting to Midthickness Lamellar Tear

Fig. 31b  Lamellar Tear in the Compression Flange Midthickness
3.5 Flange or Plates that Frame into or Pass through Webs

When flange plates intersect one or more girder webs and are permitted to pass through the webs in order to provide continuity through the intersection, slots have been used which have been welded to seal the opening. The flange tips represent a very severe fatigue condition. Laboratory studies\textsuperscript{9,10} have shown that the fatigue resistance of this detail can be less than 50\% of the resistance for Category E'. The degree of deviation is dependent on the thickness of the web plate, and the weld configuration connecting the flange plates into the web. "Seal" welds on one side of the web provide the lowest fatigue resistance.

Both field and laboratory experience suggest that this type of detail is very likely to develop fatigue cracks. Figure 32 shows a schematic of a typical detail, and Fig. 32a shows a photograph of the critical configuration.

![Diagram showing flange or plates that frame into or pass through webs.](image)

**Fig. 32** Schematic of Girder Flanges Passing through Box Girder Pier Cap
Figures 33a, b, and c show a detail at which fatigue cracking was detected in the flange tip region. The painted surface in Fig. 33a provides only a questionable indication of cracking. No significant break in the paint film is apparent. In Fig. 33b, the paint has been removed by blast cleaning, and a fine hairline crack is seen to exist. The area was then treated with dye penetrant which can be seen in Fig. 33c. This clearly revealed the existence of the crack. Because of the severity of this particular detail, a more extensive examination is warranted even when no oxide indication of cracking is observed in the paint film. When the flange framing into the web does not pass through it, the fatigue resistance is governed by Category E or $E^{10}$. The lower fatigue resistance occurs at thicker flanges (i.e. greater than 1 in.). About the same resistance is provided by fillet and groove welded connections.
Fig. 33a  Photograph of Tip of Flange and Pier Cap Web (See Fig. 32a)

Fig. 33b  Blast Cleaned Joint Showing Fine Hairline Crack
3.6 **Intersecting Welds at Lateral Connection Plates**

Sometimes when lateral connection plates are attached to the girder web and the transverse stiffener, an undesirable condition develops when the welds all intersect as illustrated in Fig. 34. The problem generally originates in the lateral connection plate-transverse stiffener weld as a result of lack of fusion or inadequately sized fillet welds connecting those two plates together. Since this transverse welded connection intersects the longitudinal and vertical welds in the corner at the girder web, a direct path is provided for any crack that propagates in the transverse connection into the girder web. Figure 34a shows a typical crack that has formed at one of the lateral connection plates. The propagation of this crack into the web resulted in brittle fracture of the girder.
Fig. 34  Schematic of Intersecting Welds at Gusset-Transverse Stiffener-Web Intersection

Fig. 34a  Crack in Girder Web at the Intersection of the Welds Visible After Hole was Cut into Gusset Plate
CHAPTER 4: INTERNAL FLAWS IN IMPROPERLY MADE GROOVE WELDS OR FILLET WELDS

Normal inspection during fabrication, if done properly, will ensure that initial flaw sizes are small. The possibility of fatigue crack growth is very sensitive to initial defect sizes. Inspectors must pay special attention to poorly made welds, as these will fail far in advance of other welds, other conditions being equal. If the structure has been in service for some time and nondestructive testing was carried out during fabrication, critical cracks are more likely to develop from small surface flaws than from internal defects. There are several important exceptions to this rule. This is particularly true of groove welds made in the 1940's and 1950's, when adequate nondestructive testing was uncommon. Furthermore, many groove welds were placed into structures without any nondestructive examination and with questionable weld quality, because they connected parts thought to have only architectural significance. One example is the longitudinal stiffener that has been placed in the tension region of fascia girders in order to provide an unbroken line.

Longitudinal stiffeners are usually supplied in varying lengths and butt welded. Since they are usually placed in a compression area, not much concern has been given to inspection in the past. At least one railway and several highway bridges have suffered failures from defects in these welds. This has occurred when the groove weld was located in a cyclic tension region. Other examples are discussed hereafter.

Several cracks have been detected in bridges that utilized A514 steel. This steel is difficult to weld, and cracking has occurred as a result.
4.1 Groove Welds in Longitudinal Stiffeners

Several bridges have had longitudinal stiffeners located in the tension area to provide symmetry and a pleasing appearance. This is illustrated schematically in Fig. 35. Since the stiffeners are fabricated from lengths of plate, several groove welds may exist throughout its length. The quality of these welds was questionable, as no weld quality criteria were established and no nondestructive test requirements were imposed on the longitudinal stiffener welds. This is readily apparent from Figs. 35a and 35b. The large flaws that existed in these weldments permitted fatigue crack growth which eventually propagated into the girder web via the longitudinal fillet welds which attached the stiffener to the web. The crack in Fig. 35a has started to penetrate into the web. Eventually, the crack will destroy the web and may penetrate into the flange. The inspector should be alert to obvious surface irregularities at such welds, as they provide an indication of the weld quality.

Fig. 35 Schematic of Girder Showing Longitudinal Stiffeners in Tension Stress Region with Butt Welded Splice
Fig. 35a Crack in the Longitudinal Stiffener Weld and partway through Web

Fig. 35b Obvious Poor Quality Groove Weld in Longitudinal Stiffener
4.2 Insert Plates

On occasion plates have been inserted into girder webs in order to patch or otherwise change the cross-section as illustrated in Fig. 36. Here a rolled section was haunched to increase its capacity by cutting the flange from the web and inserting a plate. This resulted in a transverse groove weld which was about 3 in. long. The quality of this weld was poor and resulted in fatigue crack growth. The fatigue crack has extended up the web from the groove weld and also propagated into the flange. The usual break in the paint film is apparent, as is the existence of oxide on the paint surface at the crack.

Any insert plate that exists in a tension region should receive careful attention. Focus should be on the welds that are perpendicular to the bending or axial stresses acting on the section.

![Diagram showing welded insert plate and transverse groove weld in web](image)

*Fig. 36* Welded Insert Plate Showing Short Length of Transverse Groove Weld in Web
4.3 Misplaced Holes Filled with Weld and Plug Welds

On occasion rivet or bolt holes have been misplaced in a structural component. Rather than leave these holes open or inserting bolts in them, it has sometimes been the practice to fill the holes with weldment. One such example is shown in Fig. 37.

Obviously not much weld quality control was maintained in this example, as the weld material has not been ground smooth and was unlikely to have received nondestructive inspection. Fatigue crack propagation and fracture have developed from the weld-filled holes.

All weld filled holes should be carefully examined. Any evidence of cracking should be grounds for immediate attention. It will usually be necessary to remove the weldment by drilling out the holes and removing all evidence of cracks.

Any plug welds should also be carefully examined. A plug weld provides a condition almost identical to the weld-filled holes.
Fig. 37 Schematic Showing Misplaced Holes Filled with Weld

Fig. 37a Cracking from Weld Filled Holes
CHAPTER 5: CRACKS FORMING AT RIVET HOLES

Generally, the fatigue resistance of riveted structures is good, because the stress range occurrences have been small. Hence, no major evidence of fatigue cracking has developed. However, on occasion fatigue cracking can develop in structures that are subjected to higher than average levels of stress range as a result of overloads or unusual conditions of load transfer.

Obviously, the condition of the hole is a major factor in the cyclic load performance of the riveted joint. Punched holes may have larger than average initial discontinuities which can result in premature cracking. Rivet loosening can also be cause for concern. A loosened rivet head creates a crevice and has the potential of aggravating the influence of corrosion on fatigue cracking. Moisture in a metallic crevice generally becomes acidic and creates an environment that enhances the rate of fatigue cracking. This is generally referred to as corrosion fatigue. Fortunately, most bridge steels are not sensitive to this type of behavior.

A major advantage of riveted built-up members is the multiple components which make up such members. This provides considerable redundancy, as a crack which forms in one component cannot directly grow into one of the other components in riveted and bolted structures.

Except for secondary and displacement induced stresses which have resulted in cracking in a few riveted and bolted structures as discussed in Chapters 6 and 7, no significant cracking has been detected in riveted and bolted bridges. In general, riveted and bolted built-up members in bridges will not experience cracking unless excessive corrosion increases the cyclic stress or secondary and displacement induced stresses are developed.
5.1 Cracks in Riveted Built-Up Members

Fatigue cracks that develop in riveted built-up members will grow normally to the applied stresses at the rivet hole. Figure 38 shows typical fatigue cracks that have formed in the angles of a built-up member. The crack shown in Fig. 38a has formed at the upper edge of the rivet hole and extends about 1/2 in. from the rivet head. It is clear that any fatigue crack which forms under the rivet head cannot be seen until it grows beyond the head.

The crack in Fig. 38b has developed from both edges of the rivet hole. The crack has developed an effective length of about 3 in.

Note that the paint film has provided an adequate indication of fatigue cracking, even though it has aged considerably and exhibits significant deterioration.

![Diagram of riveted multiple plate member]

See Figs. 38a & b

Fig. 38 Schematic of Riveted Multiple Plate Member

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Fig. 38a  Fatigue Cracking Indicated by
Cracked Paint at Edge of Rivet Head

Fig. 38b  Obvious Cracking at a Rivet Hole
CHAPTER 6: RESTRAINT AND DISTORTION AT END CONNECTIONS

Secondary stresses develop at some connections because they behave differently than was assumed in their design. A detailed discussion of this behavior is given in Chapters 5 and 6 of Ref. 7.

Many connections have been designed and placed into bridge structures which are susceptible to fatigue cracking and can be expected to develop cracks as time passes and the cumulative cyclic loading increases.

A few examples of fatigue cracks as a result of secondary stresses and restraint are provided in this section. These types of cracks can form in either welded or riveted (bolted) construction.

6.1 Coped or Blocked Flanges

It has often been the practice to cope or block a beam flange (see Fig. 39) to provide clearance at the end connection, so that the component parts can be connected together with relative ease. Figure 39 shows the two conditions that are provided by the coped or blocked flanges. Unfortunately, removing the flanges from the beam web (coping) causes a significant reduction in bending resistance. This reduction can be as great as 80 – 90%.

Unfortunately, most simple connections develop restraint conditions at this weakened cross-section. This can result in cracking from the cope as illustrated in Fig. 40. It is common practice to cope flanges by flame cutting which creates high tensile residual stresses. This and the sharp notch condition at the reentrant corner make the cope very susceptible to
fatigue cracking. All coped flanges should be carefully inspected at the reentrant radius. Since the cutting operation is usually made from two perpendicular directions, the potential for gouges and other undesirable conditions is as great as is the susceptibility to fatigue cracking.

(a) Coped Beam Flange

(b) Blocked Beam Flanges

Fig. 39 Schematic Showing Coped and Blocked Beam Flanges
Fig. 40 Diaphragm with Coped Flanges Showing Likely Crack Development

Fig. 40a Fatigue Cracking that Developed at Coped Flange Detail
6.2 Prying on Rivet and Bolt Heads at Simple Connections

The prying of the outstanding legs on the rivet heads of stringer-to-floor beam and floor beam-to-hanger connections has caused fatigue cracks to develop in the rivets under the head as is illustrated in Fig. 41.

Generally, the first evidence of such cracking will be the development of a thin oxide film around the edge of the rivet head as the paint film is broken. Eventually the cracking will become sufficient, so that an obvious clearance develops between the rivet head and the leg of the connection angle. Unless observed and detected earlier, the rivet heads will be cracked off.

Any evidence of oxide and separation of the rivet head should be further explored. A sharp hammer blow may dislodge the rivet head if fatigue cracking has developed far enough.

Even tightened high strength bolts will experience fatigue cracking when excessive prying develops from flexible connected parts. Bolts that replaced the rivets in Fig. 41 have also cracked from the cistortion that developed.
Fig. 41 Schematic of Stringer-Floor Beam Connection

Fig. 41a Distortion of End Connection Results in Cracking of Rivet (or Bolt) Heads
CHAPTER 7: DISTORTION IN SMALL GAPS

One of the most common sources of fatigue cracks that has developed during the last ten years has resulted from unaccounted for out-of-plane movement in small gap regions. Distortion of this gap region results in high cyclic stresses on the surface of plate components such as the ends of welds which attach transverse stiffeners (or floor beam connecting plates) to girder webs. The cyclic stress range in the gap is often many times larger than stresses in the girder from normal loading.

An extensive discussion of the behavior observed in several structures is given in Refs. 7 and 8. Generally an out-of-plane distortion occurs in the gap region as the three-dimensional structure adjusts to movements of the connected components. The forces causing these out-of-plane movements are usually very small and have a negligible influence on the structure as a whole.

Fortunately, the cracking that occurs in gap regions, generally develops in a line that is parallel to the direction of the primary stresses from loads. As a result they do not adversely influence the members' strength, nor the performance of the structure. They do create maintenance problems, and if not treated, may eventually lead to more serious fatigue or fracture damage.

Several examples of cracks that have developed in gap regions are discussed hereafter.
7.1 **Floor Beam Connection Plates Near End Supports**

When a web gap exists near an end support, out-of-plane movement in the web gap can be expected since the support prevents lateral movement of the tension flange. Figure 42 shows the cracking that develops along the web-flange welds and at the end of the stiffeners and floor beam connection plate.

The crack along the web flange fillet will continue to grow until sufficient flexibility exists to accommodate the rotation at the end of the floor beam. The usual oxide that forms at the broken paint film can be readily seen in Fig. 42a. The crack that has formed at the end of the transverse stiffener also exhibits the telltale oxide.

![Diagram of floor beam connection plates near end supports](image)

*Fig. 42 Schematic Showing Crack in Girder Web at Floor Beam Connection Plates at Supports*
Fig. 42a  Fatigue Cracking Along Flange-Web Connection and at End of Transverse Stiffener

Cracks near the end reactions can be further aggravated by irregularities at the bearings.

7.2 Floor Beam Connection Plates in Negative Moment Regions

In the negative moment regions of continuous girders, the tension flange of the girder is prevented from moving by the slab. Usually the girder is partially embedded in the slab in order to provide lateral support.

As can be seen in Fig. 43, this has resulted in out-of-plane movement in the gap region and results in fatigue cracks at the web-flange weld and at the end of the connection plate and transverse stiffener. The obvious oxide is readily apparent in Fig. 43a. Repeated opening of the crack produced crack extension and further evidence of oxide which has run down the girder web. Conditions as shown in Fig. 43a can be seen from the ground either with the naked eye or with field glasses.

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All floor beam or floor beam truss connection plates are potential candidates for the cracking illustrated in Fig. 43. A recent survey in one state indicated that 50% of the bridges with this detail had fatigue cracks.

Fig. 43 Schematic of Crack in Girder Web at Floor Beam Connection Plates in Negative Moment Region

Fig. 43a Fatigue Crack Along Web-Flange Connection
7.3 Floor Beam Connection Plates

in Negative Moment Regions of Skewed Bridges

In skewed bridges, the distortion in the web gap will often be much greater because of additional end rotation due to the differential vertical movement between the two ends of the floor beam. This increased out-of-plane movement results in more severe and serious cracking at a web gap as can be seen in Fig. 44.

Cracks are seen to radiate from the web gap region and to turn and move down the girder web. As a result, they become susceptible to further crack growth from the bending stress in the girder.

The telltale oxide (rust) is readily apparent at each crack that has formed. Figure 44a shows the protected interior surface of the girder. The amount of oxide is not as great as exhibited in Fig. 43a which shows an exterior surface.

Fig. 44 Schematic of Cracks in Girder Web and Transverse Connection Plate Welds at Floor Beam in Negative Moment Region of Skewed Bridge
7.4 Diaphragm Connection Plates in Skewed Bridges

Diaphragms and cross-frames often possess geometric conditions that result in out-of-plane distortions in the web gaps at the end of the transverse stiffeners to which they are usually attached. Since the diaphragm is generally installed perpendicular to the longitudinal girders, the difference in movement at the two ends of the diaphragm (or cross-frame) introduces an out-of-plane movement into the web gap. If the girder flange is not flexible enough to allow lateral movement, the stresses in the web result in fatigue cracks as illustrated in Fig. 45.

The cracks have generally started at the bottom of the transverse stiffener and form a "U" shaped crack around the bottom of the stiffener which then extends horizontally into the girder web as is apparent in Fig. 45. Again the oxide that forms at fatigue cracks is readily apparent. The "working" (i.e. the relative movement of the crack surfaces) of the crack results in oxide which runs down the paint on the girder web.
Cracks sometimes will form along the web-flange weld, so that location should be checked as well.

Fig. 45 Schematic of Transverse Diaphragm Attached to Cut-Short Welded Connection Plate Showing Web Cracking in Gap

Fig. 45a Fatigue Crack in Girder Web at End of Diaphragm Connection Plate
7.5 Small Gaps in Riveted Structures

Cracking at small gaps which are subjected to out-of-plane movement is not confined to welded structures. Cracking can also develop in riveted and bolted structural components when an out-of-plane movement is introduced in a short restrained region. An example of such cracking is given in Fig. 46. This condition has developed where a riveted stringer is framed into a floor beam. Rotation at the end of the stringer moves the floor beam web out-of-plane between the two vertical lines of rivets that attach the stringer end seat and the transverse stiffeners to the web.

Cracking such as shown in Fig. 46 is likely to take longer to develop in riveted structures than in comparable welded components. This results because the stress concentration and discontinuities at the weld toe are more severe than the plate surface.

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Fig. 46 Schematic Showing Stringer Framing into Riveted Floor Beam with Crack Between Seat Angle Connection and Adjacent Web Stiffeners
Fig. 46a Crack in Girder Web Gap Between Seat Angle and Transverse Stiffeners
CHAPTER 8: TACK WELDS

Although tack welds were generally prohibited from use on riveted structures, particularly in regions of high cyclic stress, there are numerous cases where they are known to exist. Often they were used to temporarily hold parts together while rivets were being placed. In welded structures, the AWS Specification requires that tack welds be properly removed or incorporated into the permanent weld.

Any tack weld provides undesirable characteristics at the end of the weld. The possibility for larger than usual discontinuities is greater, because the weld length is small and the care and quality control are not as good as with other weldments. If the tack weld has been used to attach a connection plate to a girder or truss member and the weld is near the end of the plate, the stress field at the weld will generally be larger than the nominal calculated stresses because of the stress concentration effect of the riveted and tack welded plate.

Cracks that form at tack welds will be similar to other welded connections with the fatigue crack forming from the end of the weldment. Another possibility is for the tack weld to crack in two from larger than anticipated cyclic shear stresses acting on the welds. Either case provides conditions that are undesirable, although cracking from the weld end into the connected plates is the most serious case.

Tack welds are also dangerous, because in the absence of preheat and postheat, the tack weld produces a region of untempered martensite which is susceptible to stress corrosion cracking. Weld shrinkage produces
yield point tensile stress across the tack weld which can assist crack initiation and extension. Often the surfaces which are partially joined by a tack weld create a crevice. This provides a condition which assists corrosion and is likely to expedite crack initiation and enhances fatigue crack growth rate.

8.1 Tack Weld of Floor Beam - Cantilever Bracket Splice Plate

A number of bridges have experienced fatigue cracking as a result of in-plane bending being introduced into the transverse members by the bending deformation of the main longitudinal girders. In at least one of the structures the cantilever bracket connection plates were tack-welded to the cantilever bracket during fabrication prior to the rivets being installed.

Figure 47 shows a fatigue crack that has formed in the splice plate of this type of structural system. The fatigue crack is readily visible as the paint has been chipped away from the region with the crack. On a number of plates the usual oxide (rust) condition was readily apparent and assisted in locating the cracks.

Any tack weld should be carefully examined to determine whether or not fatigue crack growth has developed.

8.2 Tack Welds on Built-up Truss Members

A second example of fatigue cracking of a tack-welded member is shown in Fig. 48. Gusset plates for a riveted truss with riveted built-up members were obviously temporarily attached to the tension chord member with tack welds. Fatigue cracking has developed in the channel flange and cover plate from the tack weld which connected the gusset to each of these member components.
Fig. 47  Schematic of Longitudinal Girder, Floor Beam and Outrigger Bracket Connection Showing Tack Weld Location on Tie Plate

Fig. 47a  Crack in Tie Plate at End of Tack Weld
Fig. 48  Schematic of Hanger-Chord Connection Showing Location of Tack Weld and Crack in Chord

Fig. 48a  Small Fatigue Crack in Channel Section at Tack Weld Attaching Gusset Plate to Chord Member

The examples shown in Figs. 47 and 48 demonstrate the need to closely scrutinize plate components that are bolted or riveted to members. All tack welds should be identified as potential locations for fatigue cracks.
CHAPTER 9: STEPS TO FOLLOW WHEN FATIGUE CRACKS ARE OBSERVED

Fatigue cracks are most detrimental to the safety and performance of a structure or component when they lie in a plane perpendicular to the applied stress. If a crack is detected, the following steps are recommended:

For the Inspector

1. Report the fatigue crack to the District Bridge Engineer as quickly as possible. A sketch and/or photograph is desirable, so that the crack location, size and orientation can be evaluated.

2. Determine the visual ends of the crack. The crack tip will extend beyond the crack in the paint film and beyond the oxide indication.

3. Examine other identical details on the bridge if they exist. Any fatigue cracks will likely occur at the same relative location at the detail. Those details attached to members located under the most heavily traveled truck lanes should be examined first in multiple girder bridges.

4. When examining other details look carefully for breaks in the paint and the formation of oxide (rust) at the weld end where the crack originated.

5. If a suspect area is located, a more detailed examination should be carried out, such as blast cleaning the area and applying dye penetrant or a visual examination with a 10X power magnifier.
6. Evaluate the significance of the crack on the load-carrying capacity of the bridge, considering the crack size, known material characteristics, and temperature.

7. If the crack is moving perpendicular to the stress field in the member, it is desirable to arrange to have holes drilled at the crack ends as soon as practicable. The holes should be large enough to install a 3/4 in. or 7/8 in. A325 bolt. The edge of the holes should be placed at the presumed end of the crack. After holes are drilled, it is desirable to check the hole to insure that the crack tip has been removed and does not pass through the hole. This is generally a temporary retrofit pending development of a permanent repair. Also, instruct the field personnel to spray coat the crack surface with a clear acrylic lacquer such as Kryolon. After the hole(s) have been checked, high strength A325 bolt(s) should be installed in the hole(s) and tightened by one-half turn-of-the-nut.

8. Determine if special nondestructive tests are desirable at other locations, such as dye penetrant, ultrasonic testing or a more thorough visual examination.

9. Review results of examination of other locations on the bridge structure. Determine if a pattern develops related to truck traffic lanes and geometry of structure.

10. Determine if the crack or cracks have developed from normal fabrication conditions or as a result of an unusual flaw.
11. Develop a repair and retrofit scheme for the fatigue damaged area.

12. Determine whether or not other structures exist with similar details and conditions. Those structures located on the more heavily traveled roads with the highest average daily truck traffic should receive the highest priority for any subsequent inspection.

A discussion of how to go about inspecting the bridge structure and how to evaluate the results acquired are discussed in greater detail in Chapter 10.

Suggested Reporting Format

In order to assist with the investigation, evaluation, and retrofitting of cracked components and members, it is recommended that the following items be assembled and documented:

1. The date the crack was first detected.

2. Detailed sketches showing the crack location and dimensions of the crack and the member. These sketches should also indicate the crack widths and the orientation of the crack with regard to the primary stresses in the member.

3. Photographs showing the visible crack conditions and its location relative to the detail at which it formed.

4. The design stress conditions normal to the crack or detail as applicable. Both the stress range from live load and impact, and the maximum stress should be reported, along with the basis for these stresses (i.e. HS20 truck loading or lane loading).
5. The average daily truck traffic (ADTT) estimate from the most recent traffic data. Where possible, an estimate of the ADTT for the period of time the bridge has been in service.

6. The estimated frequency of any overloads, by permit or other basis.

7. An indication of the approach and deck surface conditions to permit an assessment of impact. The length of time that these conditions have existed.

8. The yield point of the cracked plate or rolled section as provided by mill reports or tests.

9. Charpy V-Notch impact values and test temperature from mill reports or other sources.

10. The minimum temperature that the structure has experienced during its service life. This data can be approximated from the nearest weather station.

11. The minimum temperature that the structure experienced during the year preceding discovery of the crack.

12. An indication as to whether the cracked member was struck by a vehicle.

13. If the crack formed at a groove weld, determine how these welds were inspected when the member was fabricated. Obtain all available inspection reports and the radiographs if possible.

14. If the crack forms at a welded detail, identify the applicable fatigue design classification according to the details described in Chapter 2.
CHAPTER 10: GUIDELINES FOR DEVELOPING AN INSPECTION PROGRAM
FOR A BRIDGE STRUCTURE

The bridge inspector should discuss the details that are known to exist on a bridge with the bridge engineer, so that the lowest fatigue resistant details can be identified. These details warrant a more thorough examination than other details on the structure. Chapter 3 described details that are known to possess low fatigue resistance. Any bridge structure that contains these details and is subjected to a high volume of truck traffic will need these details examined closely.

There are several levels of inspection that can be carried out. A visual examination can be made of the suspect details often from the ground. It is recommended that binoculars be used to see the details focusing on the location where cracks will likely grow. Chapters 3 to 8 have provided examples of cracks that developed in bridge structures, and the photographs of these cracks can be used to help identify the proper location to focus the inspection on. As indicated in these photographs, cracks can often be identified from the oxide (rust) lines that form after the paint film has broken. Only large cracks will be detected at this level of inspection.

The next level of inspection is a close visual inspection of the detail. Careful attention should be given to the weld toe of attachments to the flange or web. The colored prints shown in Chapters 3 to 8 provide guidance on the location to focus on. Generally any visual crack will likely be several inches long and at least 1/4 in. deep in order to break the paint film.
Suspect details that show no visual evidence of cracking through the paint film should be carefully examined with either a magnifying glass, ultrasonic probes, magnetic particle, or dye penetrant inspection procedures. This level of inspection will not be required unless the engineer has determined that sufficient stress cycles have accumulated to warrant such a close examination. Appendix A provides guidelines that can be used to estimate the fatigue damage in a given bridge structure.

It will be necessary to remove the paint along the weld toe by blast cleaning or flame cleaning the weld toe region. Only a narrow strip needs to have the paint film removed. Because of oxidation it is usually necessary to limit the blast cleaning, so that no more than twelve hours elapse before the point is examined. In some regions it may be desirable to immediately examine the blast cleaned area. A careful visual examination with a 10X magnifier such as an illuminated Coddington will be as reliable as other nondestructive test methods.

Once a crack has been located, it is essential that its dimensions and location be accurately determined. Sketches and photographs should be prepared showing the crack, its location in the member and its size. This information will be needed by the engineer to assess the significance of any crack that is detected.

Only by working together can the inspector and engineer insure that safe and reliable bridge structures will be available for the traveling public.
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APPENDIX A: A PROCEDURE FOR ESTIMATING FATIGUE DAMAGE IN EXISTING STRUCTURES

1. The first step is to develop a reasonable estimate of the stress range spectrum for the traffic using the structure. This must include regular traffic, any permit overloads and other overloads (see Fig. A2). It should be kept in mind that most stress cycles will be caused by passage of a single vehicle. Consideration should also be given to the fact that not all passages will result in maximum impact. Measurements have generally indicated that impact follows a Rayleigh type skewed distribution as shown schematically in Fig. A1.

The curves in Figs. A1 and A2 can be used to estimate the stress range spectrum as follows. The peak value $S_{r_{\text{max}}}$ can be approximated from the maximum gross vehicle weight and the design impact. The other occurrences $S_{r_i}$ can be estimated from the product of the RMS impact value and the gross vehicle weight $\text{GVW}_i$. On certain structures it may be desirable to utilize a factor larger than one to account for more than one vehicle on the structure. An alternative estimate can be obtained by approximating $S_{r_i}$ as a percentage of the design stress range base of HS20 loading. The resulting values of $S_{r_i}$ should be decreased by a factor $\alpha = 0.7$ for longitudinal members and 0.8 for transverse members.

On bridges which warrant a more refined estimate, it is preferable to obtain stress history measurements under normal traffic, so that a better estimate of $S_{r_{\text{Miner}}}$ can be obtained.
2. After developing the best estimate of the stress range spectrum (see Fig. A3), the peak stress range at the detail in question should be compared to the fatigue limit for the detail. This value is tabulated as the allowable stress range ($S_{rall}$) for more than 2,000,000 cycles in the AASHTO Specification.

If $S_{rmax} < S_{rall}$

Then fatigue crack growth will not occur and no further evaluation is necessary.
3. When \( S_{\text{rmax}} > S_{\text{rall}} \), all available information indicates that fatigue crack growth will develop with large numbers of accumulated stress cycles. In order to assess the degree of cumulative damage, the effective stress range must be determined from the stress range spectrum. The effective stress range should be calculated as

\[
S_{\text{rMiner}} = \left[ \sum \gamma_i S_{ri}^3 \right]^{1/3}
\]  

(1A)

This will indicate how probable fatigue damage is at the present time. The closer the cumulative stress cycles are to the fatigue resistance curve, the greater the probability of fatigue cracking. When the cumulative cycles are 80–90% of the distance to the fatigue resistance curve, visible cracking should become apparent (see Fig. A4) and detectable at several details.
4. **If large cracks exist** (i.e. rejectable internal flaws greater than 1/4 in. and visible weld toe cracks) **because of fabrication or some other event**, the design fatigue resistance curves are **no longer applicable**. The residual fatigue resistance in these cases must be assessed by fracture mechanics models. The crack must be modeled, and the stress intensity range estimated as

\[
\Delta K = F_e F_s F_w F_g S_{r\text{Miner}} \sqrt{a}
\]  

(2A)

**Where:**
- \( F_e \) = crack shape correction
- \( F_s \) = front free surface correction
- \( F_w \) = back surface correction
- \( F_g \) = stress gradient correction
- \( a \) = crack size
The cycles (and time) required to propagate a crack $a_i$ to some larger increment $a_f$ can be estimated as

$$\Delta N = \int_{a_i}^{a_f} \frac{da}{3.6 \times 10^{-10} \Delta K^3}$$  \hspace{1cm} (3A)$$

Where: $S_{r,\text{Miner}}$ (ksi) and $a_i$ (inches)

Generally, large cracks have a negligible stress concentration effect, $F_g$, and it can be taken as unity in this approximation.

It is also necessary to check the fracture resistance for the large crack. This is given by

$$K_{max} = F_e F_s F_w S_{max} \sqrt{\pi a}$$  \hspace{1cm} (4A)$$

When the crack tip is in a residual tension region, $S_{max} = \sigma_y$. The estimated maximum stress intensity factor, $K_{max}$, must be less than $K_c$ of the connected material.

Example: Yellow Mill Pond

Stress spectrum was acquired by strain measurements on the bridge which yielded

\[ S_{r,\text{Miner}} = 1.9 \text{ ksi} \]

\[ S_{r,\text{max}} = 6.9 \text{ ksi} \]

Fig. A5 Measured Stress Range Spectrum
The variable stress cycles $N_v$, were estimated from the ADTT

$$N_v (1976) = 35 \times 10^6$$

Fig. A6 Average Daily Truck Traffic Using the Structure

Detail is Category E'; $S_{rall} = 2.6$ ksi

Therefore $S_{rmax} > S_{rall}$

Fig. A7 Comparison of Cumulative Stress Cycles with Fatigue Resistance

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Largest crack ~ 0.5 in.; check for fracture.

\[ b = 5.462 \ a^{1.133} = 2.49 \text{ in.} \]

\[ F_e = 1 \]

\[ F_s = 1.211 - 0.186 \sqrt{\frac{a}{b}} = 1.128 \]

\[ F_w = \left[ \frac{2t}{\pi a \tan \frac{\pi a}{2t}} \right]^{1/2} = \left[ \frac{2 \times 1.26}{\pi \times 0.5 \tan \frac{0.5 \pi}{2 \times 1.26}} \right]^{1/2} \]

\[ K_{max} = F_e F_s F_w S_{max} \sqrt{\pi a} \]

With rolled shape use

\[ S_{max} = \frac{\sigma}{2} + \sigma_{DL} + \sigma_{LL} \]

\[ = 33 + 4 + 6.9 \]

\[ = 44 \text{ ksi} \]

Hence

\[ K_{max} = (1) (1.128) (1.153) (44) \sqrt{\pi 0.5} \]

\[ = 71.7 \text{ ksi } \sqrt{\text{in.}} \]

Charpy V-Notch Data: 30 ft-lbs. @ 70° F

15 ft-lbs. @ 40° F

\[ T_s = 215 - 3 (66) = 116° F \]

\[ 3/4 T_s = 87° F \]

\[ K_{Id} = (5 E CVN)^{1/2} = 67 \text{ ksi } \sqrt{\text{in.}} \quad (30 \text{ ft-lbs.}) \]

This suggests that \( K_{max} \ll K_c \) at -17° F if residual stress is \( \sigma \)

Under DL + LL, \( K \approx 18 \text{ ksi } \sqrt{\text{in.}}. \)

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DISCLAIMER

Prepared in cooperation with the Pennsylvania Department of Transportation and the U. S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation or the U. S. Department of Transportation, Federal Highway Administration. This report does not constitute a standard, specification or regulation.

ACKNOWLEDGMENTS

This study was part of PennDOT Research Project No. 72-3, a study of high cycle fatigue of welded details, sponsored by the Pennsylvania Department of Transportation and the U. S. Department of Transportation - Federal Highway Administration.

The author wishes to gratefully acknowledge the assistance of the staff at Fritz Engineering Laboratory in the preparation of this report, in particular, Mr. Richard Sopko who provided the photographic coverage and was responsible for coordinating the reproduction of the report. The manuscript was typed by Mrs. Ruth Grimes, and the figures were drafted by Messrs. John Gera and Richard Troxell and Mrs. Sharon Balogh. Thanks are also due Dr. Hans Hausammann for his assistance with sketches and Dr. G. R. Irwin for helpful suggestions.

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