Steel Bridges: Design for Fatigue

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Donald Parnell, P.E.

CED engineering.com
PDH FOR THE PROFESSIONAL

Continuing Education and Development, Inc.

P: (877) 322-5800
info@cedengineering.com

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Steel Bridge Design Handbook: Design for Fatigue

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1.0 OVERVIEW

1.1 Introduction

Fatigue in metals is the process of initiation and growth of cracks under the action of repetitive tensile loads. If crack growth is allowed to go on long enough, failure of the member can result when the uncracked cross-section is sufficiently reduced such that the member can no longer carry the internal forces for the crack extends in an unstable mode. The fatigue process can take place at stress levels (calculated on the initial cross-section) that are substantially less than those associated with failure under static loading conditions. The usual condition that produces fatigue cracking is the application of a large number of load cycles. Consequently, the types of civil engineering applications that are susceptible to fatigue cracking include structures such as bridges, crane support structures, stacks and masts, and offshore structures.

In the design and detailing of structures, details that might be prone to cracking should be minimized, or avoided if possible. The structures are inspected for cracks, both during fabrication to limit the size of initial flaws and also during service to ascertain any crack growth. However, it is inevitable that cracks or crack-like discontinuities will be present in fabricated steel elements. Thus, the engineer is responsible to consider the consequences of potential fatigue and subsequent brittle fracture. The fatigue behavior of a fabricated steel structure is controlled by the presence of pre-existing cracks or crack-like discontinuities, which most often occur at welded connections or other areas of stress concentrations. Consequently, there is little or no time during the life of the structure that is taken up with "initiating" cracks.

Probably the most common civil engineering structures that must be designed for fatigue are bridges. In North America and elsewhere, early steel bridge structures were fabricated using mechanical fasteners, first rivets and later high-strength bolts. In these cases, initial imperfections and stress concentrations are relatively small. In addition, loading and load frequency were also low by today's standards. Consequently, fatigue cracking in these early structures was infrequent. In the 1950's welding began to be used as the preferred method for fabrication of steel highway bridges. This had two principal effects related to fatigue. First, welding introduces a more severe initial crack situation than does bolting or riveting due to more critical stress concentrations and flaws. Second, the continuity between structural elements inherent in welded construction means that it is possible for a crack in one element to propagate unimpeded into an adjoining element.

Design rules as welding was being introduced had been developed from a limited experimental base and the mechanism of fatigue crack growth was not well understood. Furthermore, most of the experimental results came from small-scale specimens. This is now known to be a limitation in evaluating fatigue strength: reliance on small-scale specimens can result in overestimates of fatigue strength.

During the 1970's and 1980's there were many examples of fatigue crack growth from welded details now known to be susceptible to this phenomenon. Research revealed that the type of cracking observed in practice was in agreement with laboratory test results and supportable by theoretical predictions. Experience in the 1970's also exposed an unexpected source of fatigue cracking, distortions of the structure. This is also a phenomenon related largely to welded structures.

This publication provides the practicing engineer with the background required to understand and use the design rules for fatigue resistance that are currently a standard part of design codes for fabricated steel structures. Many passages in this publication are taken with permission directly from a much more voluminous primer on fatigue (1) which should be reviewed for a more complete discussion on fatigue and fracture.

Fundamentals are presented in a general way, but applications will refer to the specification for the design of steel bridges prepared by the American Association of State Highway and Transportation Officials (AASHTO) (2). This specification is widely used in the United States, and the governing Canadian specification is nearly identical to it.

1.2 Historical Perspective

Fatigue cracking was observed in railroad equipment over 140 years ago. Studies carried out at that time by Wöhler on railway rolling stock showed that stress concentrations and sharp angles in the axle configuration resulted in failures even though the stress in the material was well below its yield strength (3). The industrialization of society and the subsequent increased use of machinery and equipment led to other examples of failures resulting from fatigue cracking. As a result, studies into the phenomenon started in both Europe and in North America. For example, in North America the observation of cracks in railroad bridge truss hangers and in stringer end connection angles led to a number of laboratory investigations between 1930 and 1960 (4).

Welded details were first examined in the 1930's when tests were carried out on welded steel details. These and later studies following World War II formed the basis for the early fatigue design specifications in North America (4). Fatigue cracks forming in steel bridges at a road test program conducted in the USA in the 1960's (5) became the genesis of the fatigue test program sponsored by the National Cooperative Highway Research Program (NCHRP) that began at Lehigh University in 1968. Prior to the NCHRP program, the fatigue design rules that existed for welded steel bridge components were based on small specimens and on a limited quantity of test data. This made it difficult to establish the significance of stress variables, detail type, types of steel, and quality of fabrication. The early provisions for fatigue life evaluation proved to be inadequate for a number of bridge details. This explains, in part, the relatively large number of cases of fatigue cracking in bridges that were designed prior to about 1975.

The approach taken by modern-day specifications for the fatigue design of fabricated steel structures is based primarily on work done in Great Britain (6) and in the USA (7, 8, and 9) in the late 1960's and early 1970's. Although many other investigators have contributed to our understanding of the problem, both before and after the work cited, it was this research that identified the influence of residual stress on fatigue life, demonstrating that due to the tensile residual stress associated with welded details, stress range was the dominate variable greatly simplifying fatigue design. These studies also revealed the necessity to acknowledge that fabricated steel structures always contain cracks or crack-like discontinuities.

Fabrication inspection insures that cracks and larger discontinuities are not incorporated into the structure. Small crack-like discontinuities occur at the intersection of the fusion line of the weld and the plate surface at the weld toe. These small crack-like discontinuities are too small for detection. The effects of these micro-discontinuities are incorporated into the design specifications.

2.0 INTRODUCTION TO CRACK GROWTH

2.1 Crack Growth Regions

2.1.1 Introduction

Crack growth in metals requires two existing conditions: existing flaws and tensile stresses. This crack growth can be delineated into three distinct regimes: initiation, steady-state propagation and unstable fracture, as illustrated in Figure 1.

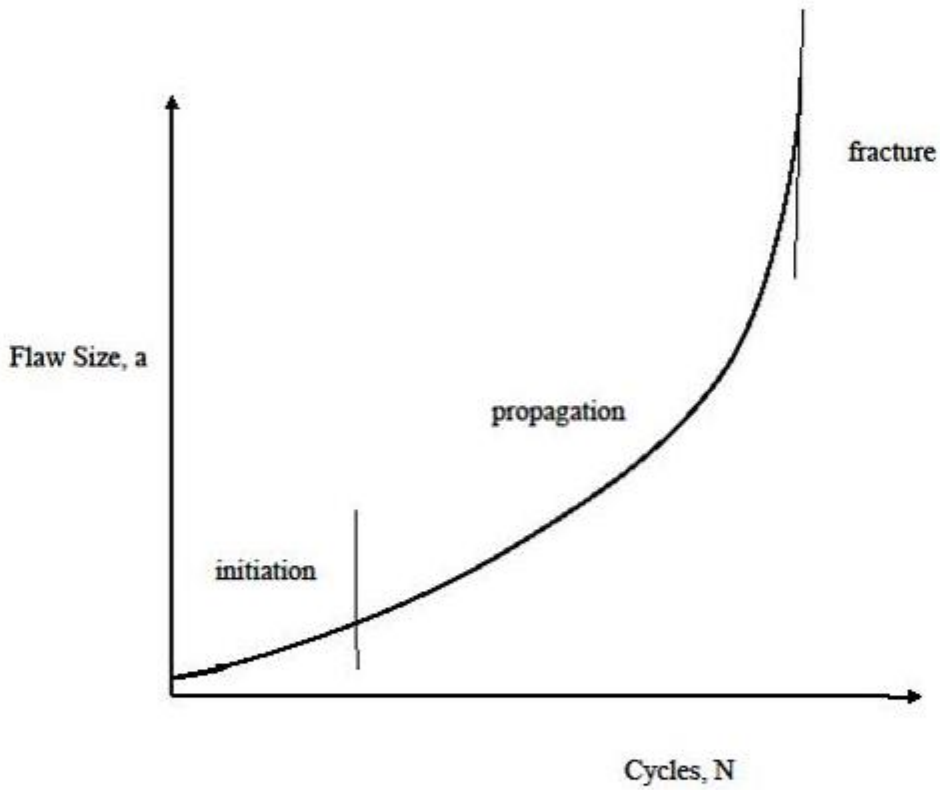


Figure 1 Regimes of Crack Growth

As has already been noted, the initiation portion of general crack growth in which existing flaws are sharpened into cracks is essentially non-existent for all fabricated steel structures and can conservatively be ignored. Thus, crack growth in bridges is delineated into two regimes: stable fatigue cracking and unstable fracture. Figure 2 graphically illustrates crack growth in bridges.

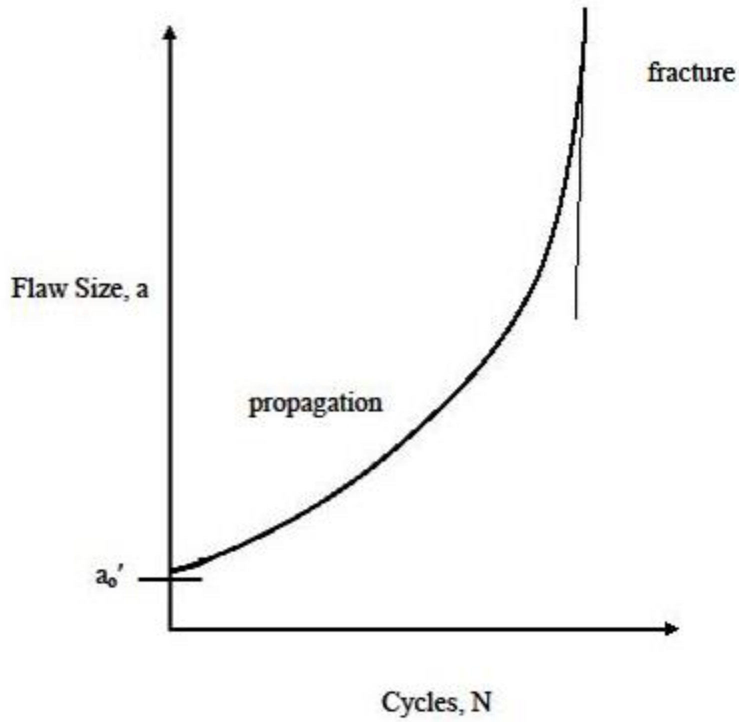


Figure 2 Regimes of Crack Growth in Bridges

The propagation regime of crack growth represents steady-state fatigue cracking. When a critical flaw size is achieved, the stable crack fractures in an unstable manner without an increase in stress. Unstable crack propagation can be in the form of a low energy brittle type fracture or if the material has sufficient toughness, a ductile tear. The critical flaw size is proportional to fracture toughness of the material which varies with rate of application of the load and the temperature of the material. The fracture toughness of the material is controlled by the Charpy V-notch toughness specified in the material specification.

2.1.2 An Illustrative Example

Fatigue can be defined as the initiation and propagation of microscopic cracks into macro cracks by the repeated application of stress. An initial crack grows a small amount in size each time a load is applied. Growth occurs at the crack front, which is initially sharp. Even at relatively low loads, there will be a high concentration of stress at the sharp front, and plastic deformation (slip on atomic planes) therefore occurs at the crack front. Continued slip results in a blunted crack tip, and the crack grows a minute amount during this process. Upon unloading, not necessarily to zero, the crack tip again becomes sharp. The process is repeated during each load cycle.

Figure 3 shows the fracture surfaces of a member that has an I-shaped cross-section. The web of the member, which was 10 mm thick, was fillet-welded to 13 mm thick flange plates. (The full thickness of the flange is not shown in Figure 3). The profiles of the fillet welds are generally satisfactory and the flow lines of the weld show good penetration of the base metal. In this illustration, an internal flaw at the root of the weld in the left-hand fillet weld (highlighted by the superimposed arrow) grew outward toward the free surface under the repeated application of stress until the crack penetrated the outside surface of the weld. Since this was a laboratory specimen, at this point the beam was deliberately overloaded so that the remaining cross-section fractured and could be exposed.

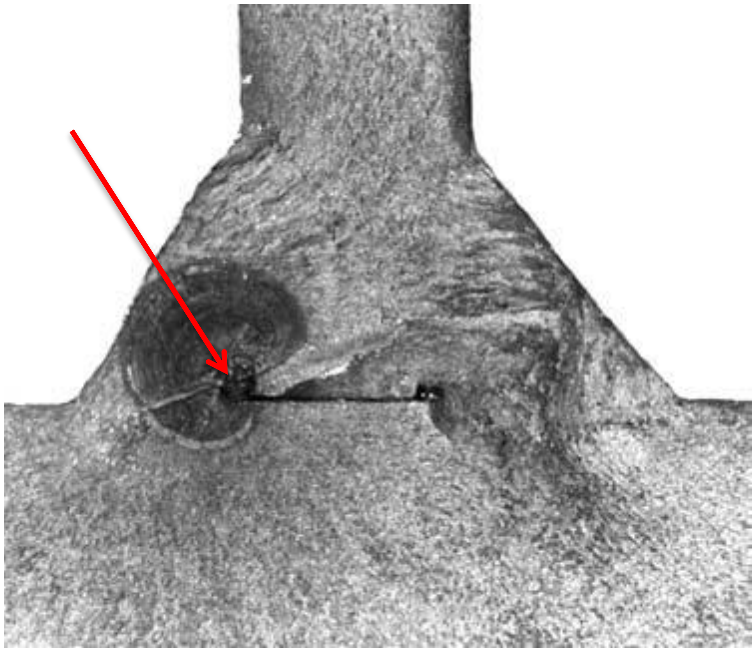


Figure 3 Fracture Surface of I-shaped Cross Section

In the case illustrated by Figure 3, the crack front eventually reached the exterior surface of the weld. Experience in the laboratory shows that as much as 80% of the fatigue life has been consumed by the time a fatigue crack emanating from an internal flaw reaches the surface and can be observed. The lack of fusion inherent in a fillet welded web to flange connection was not the discontinuity that caused the cracking. The plane of the lack of fusion is parallel with the direction of stress.

If the test that produced the specimen shown in Figure 3 had not been terminated by the investigators, failure could have occurred in one of two ways. One possibility is that the fatigue crack grows to such an extent that the loss of cross-section means the load simply can no longer be carried by the uncracked portion of the beam. In this case, failure occurs by yielding of the remaining material, or, exceptionally, by instability if the crack growth produces a grossly unsymmetrical cross-section. The other way that the beam can fail is by brittle fracture. Growth of a crack by fatigue can lead to brittle fracture if the crack reaches a critical size according to the particular conditions of material toughness, temperature, and loading rate.

2.1.3 Flaws in Fabricated Steel Structures

The kinds of flaws that can occur in a fillet-welded detail are shown pictorially in Figure 4. These include partial penetration and lack of fusion, porosity and inclusions (the fatigue crack shown in Figure 3 started at a non-metallic inclusion), undercut or micro flaws at the weld toe, and cracking or inclusions around a weld repair or at start-stop locations or at arc strikes. Although the fabricator of the structure and those responsible for the fabrication inspection will attempt to minimize these defects, it is neither practical nor economically possible to eliminate them.

In the case of a typical flange-to-web fillet weld, as shown, partial penetration is not a defect and is of no concern as the applied stress is parallel to the weld. Surface defects are subject to visual and magnetic-particle inspection and are not a concern for bridges. The hidden flaws like porosity are accepted as the AASHTO LRFD Specifications was based upon tests of real welds with porosity flaws in them.

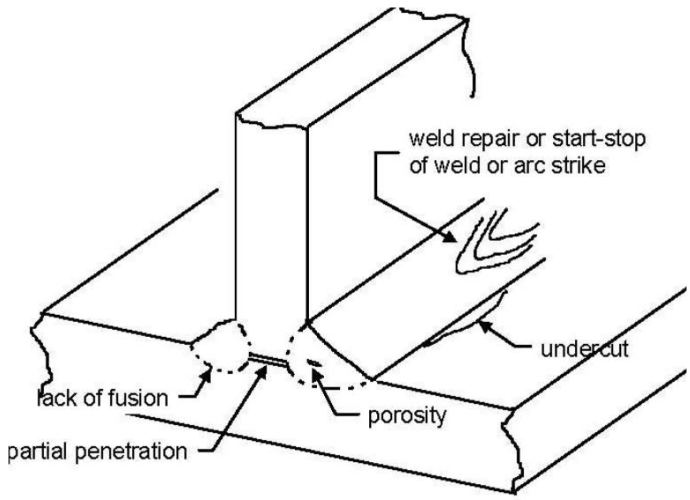


Figure 4 Discontinuities in a Fillet-Welded Detail

Test data on welded details have demonstrated that all fatigue cracks commence at some initial discontinuity in the weldment or at the weld periphery, and grow perpendicular to the applied tensile stresses. In a welded beam without attachments (simply two flange plates welded to a web), most laboratory fatigue cracks are observed to originate in the web-to-flange fillet welds at internal discontinuities such as porosity (trapped gases in the unfused area expanding and getting trapped in the solidifying weld), incomplete fusion, or trapped slag. Figures 5 and 6 show fatigue cracks that have formed from porosity (highlighted by arrows) in longitudinal submerged-arc fillet welds. These relatively large discontinuities are always present to some degree, irrespective of the welding process and techniques used during fabrication. The effect of these internal discontinuities is included in the design fatigue resistance which eliminates the need to for an internal inspection of fillet welds.

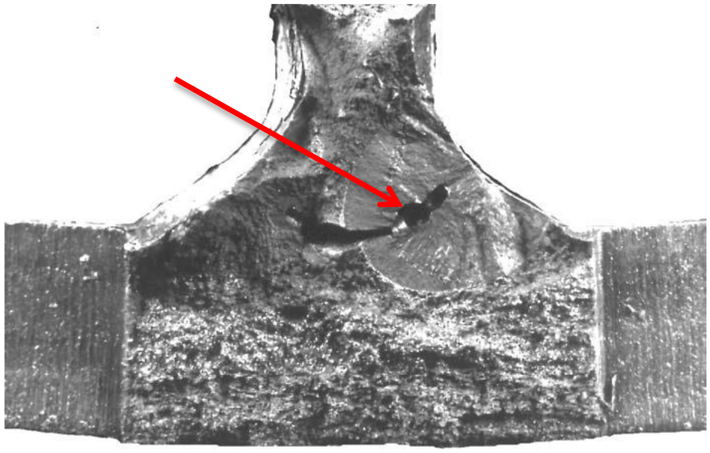


Figure 5 Fatigue Crack Forming from Internal Porosity in Web-Flange Connection

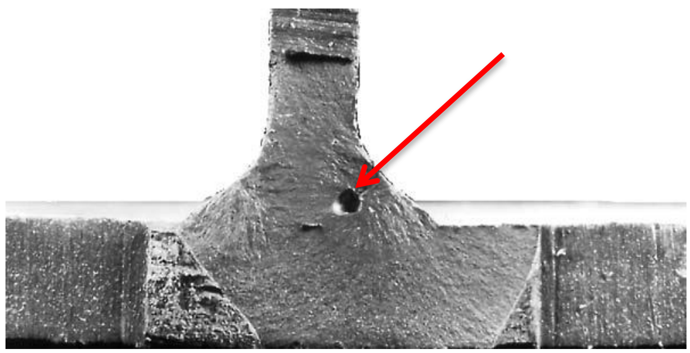


Figure 6 Fatigue Crack Enlarged to Three-Ended Crack from Internal Porosity

Attachments such as cover plates, gussets, stiffeners, and other components welded to a web or flange introduce a transverse weld periphery (toe), thus forming a line of elevated tension where fatigue cracking can start from small, sharp discontinuities. Figures 7 and 8 show a fatigue crack that has formed at a cover plate fillet weld toe. The crack surface in Figure 8 shows various stages of crack propagation. The first stage is a surface crack growing in the flange as a semi-circle until it penetrates the far side of the flange. The second stage is a through crack in the flange growing both to the right and left until it reaches the flange tip on the right. Finally, with the flange tip severed, the through crack grows toward the web on the left with the rougher texture on fracture surface suggesting more rapid propagation.

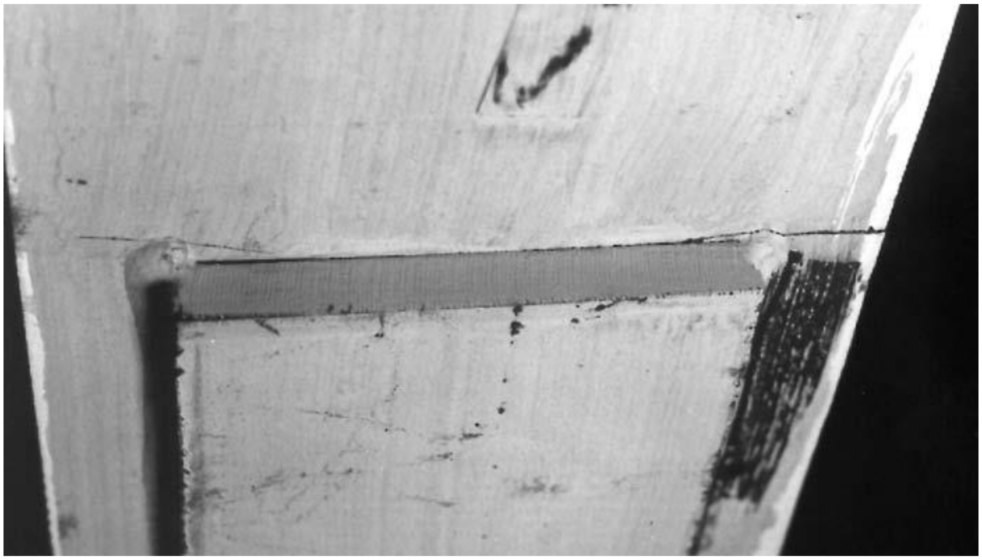


Figure 7 Fatigue Crack at End of Cover Plate Fillet Weld Toe

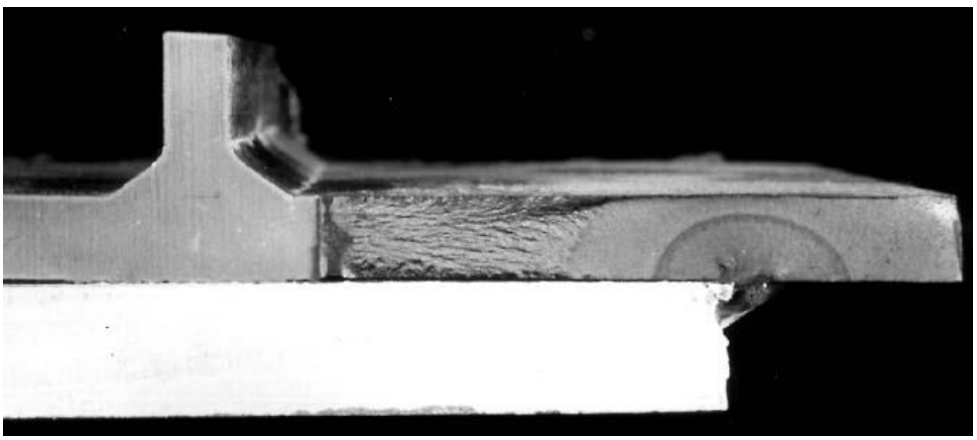


Figure 8 Crack Surface Showing Fatigue Crack Growth

In some cases, a "defect" is an expected result of the type of fabrication process and has no effect on the life of the member. For instance, the partial penetration shown in Figure 3 is a natural consequence of the fillet-welded connection: it is not expected that the two fillet welds will fuse in the central region of the connection. Furthermore, since the crack represented by the lack of penetration is parallel to the direction of the (bending) stress field, the crack will not open up under the application of stress and failure by fatigue will not occur.

Consider a detail involving mechanical fasteners—an I-shaped beam with a cover plate fastened to the beam flange with bolts. The region between bolt lines could be described as a "flaw" or "crack," but, since the discontinuity is parallel to the stress field, the "crack" does not grow and therefore its presence does not affect the fatigue strength of the member.

The flaws that exist in all fabricated steel structures are a consequence of the manufacturing process of the steel itself and the normal fabrication processes. Flaws in rolled shapes arise from surface and edge imperfections, irregularities in mill scale, laminations, seams, inclusions, etc., and from mechanical notches due to handling, straightening, cutting, and shearing. These irregularities are controlled by product and fabrication specifications (such as the specified roughness of a flame cut edge). In a rolled shape, fatigue crack growth can start from one of these sources. Comparatively, the "unaltered" rolled shape presents the most favorable fatigue life situation. However, there are not many practical cases in which a rolled shape does not have some kind of attachment, connection, or some other kind of alteration.

Mechanical details, in which holes are drilled or punched and forces are transferred by means of rivets or bolts, present a somewhat more severe fatigue life situation than the bare rolled shape. Drilled or sub-punched and reamed holes give some reduction in fatigue life as compared with an unaltered member, but the difference usually is not very great. If preloaded high-strength bolts are used, the disturbing effect of the hole is largely mitigated by the presence of the high local compressive stresses introduced by the bolt. Punched holes give a greater reduction in fatigue life than do drilled or sub-punched and reamed holes because of imperfections at the hole edge arising from the punching process. In this case, the crack usually starts at the edge of the hole.

Broadly speaking, any mechanical detail has a better fatigue life than does its equivalent welded detail. The types of flaws and the large stress concentrations associated with weld toes transverse to the direction of stress introduced when welding is used have already been discussed. In addition to the fact that more flaws will be present when welding is used, inspection for defects is more difficult than is the case when mechanically fastened details are used. Repairing defects in welded details is also difficult. Prohibiting the use of welded details in fatigue situations is not usually a practical option, however.

The task of the structural engineer is to be able to proportion those structural members that have a potential for failure by fatigue crack growth so that they have a fatigue life exceeding the design life of the structure. As will be seen, this will be done in the environment that some probability of failure must be accepted: in real terms, there is no structure that can be designed for zero probability of failure. The design will be carried out in the expectation that flaws will be present initially in all fabricated steel structures and that all such members will contain residual stresses of relatively high magnitude. A concomitant feature is that in the design process it is possible to identify the size of flaws that are permissible and then to use this information as the basis for both initial inspection of the structure as well as periodic inspections. This latter feature is not yet well-developed in design specifications, and the usual procedure is to accept as permissible flaw sizes consistent with the specifications that accompany the fabrication processes, e.g., the welding specifications.

2.2 Load-Induced Fatigue

2.2.1 Stress Range as the Dominant Stress Parameter

Steel structures that are fabricated by welding contain "residual" or "locked-in" stresses that are a consequence of the welding process. These have considerable influence on the propagation of fatigue cracks. The main effect is to significantly reduce the effects of the mean stress levels. For modern codes, this has brought about a return to the simple stress range vs. cycle life model for fatigue strength suggested by Wöhler over one hundred years ago. Furthermore, independence of steel grade justifies the use of the relatively large data base of laboratory results taken from tests of different steel grades produced in different countries. During recent modifications to fatigue codes, code developers have taken advantage of such opportunities and, consequently, fatigue design guidelines have been greatly simplified and harmonized internationally.

Consider a weld laid down as shown in Figure 9. As the weld cools, it tries to contract. However, since the plate and the weld must maintain compatibility of length, the plate restrains the weld during the cooling and contraction process. This puts the weld and a relatively small volume of plate adjacent to the weld into tension. Conversely, the main portion of the plate is compressed by the contracting weld, thereby placing it into compression. The resulting stresses from this process are called residual stresses. Since there are no external forces applied during this process, the equilibrium condition of the cross-section must be reflected in the balance between residual tensile stress and the area over which it acts and the residual compressive stress and its associated area. The actual distribution and magnitude of the residual stress pattern depends upon such factors as the strength of the steel and the weld metal, the sequencing of the welds, the geometry of the connected parts, and the size of the weld relative to the connected parts. The important fact, however, is that the magnitude of the tensile residual stress can reach the yield strength of the material.

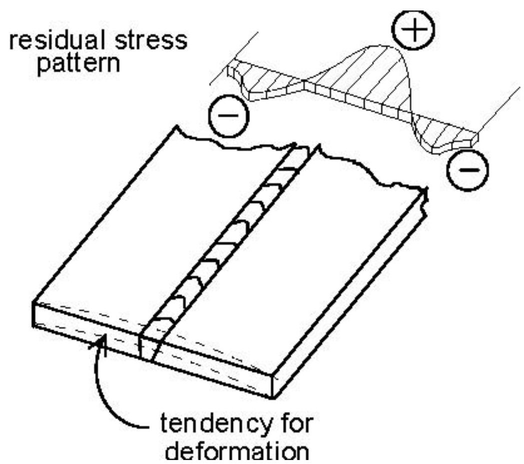


Figure 9 Residual Stresses in Groove-weld Connected Plates

It follows, of course, that the rolled shapes or built-up members used in structural applications also contain regions of high residual tensile stress. For example, very large residual tensile stresses are present at the junction of the flange and web of a beam that has been built up by welding the component parts together. This junction is also the location of the flaws that are likely to be the source of fatigue crack growth, which means that the flaw is under a condition of initial stress even before load is applied. For the usual condition wherein this initial stress is at or near the yield stress level, this means that stress range, rather than the maximum applied stress, the stress ratio (ratio of maximum stress to minimum stress), or some other parameter of applied stress, is the governing condition describing fatigue crack growth.

To summarize, in large welded structures there are very high tensile residual stresses near fatigue crack sites and their presence significantly reduces the effects of mean stress level of the applied stresses and steel grade upon crack propagation for standard weldable structural steels. As a result, it is generally agreed that stress range is the dominant stress parameter for fatigue design.

2.2.2 Categorization of Details

The AASHTO LRFD design approach is simply to arrange standard structural details into categories relative to their expected fatigue life, based on nominal stress. For example, illustrated in Figure 10 are the fatigue life representations for two different categories—beams which have cover plates that include a weld across their ends and beams made up of three plates welded together (in other words, a plate girder), such as the beam illustrated by Figure 3. The vertical axis is the nominal stress range at the location of the weld and the horizontal axis is the number of cycles to failure. Clearly, if the designer had one of the two types of members shown in Figure 10, it would be possible to determine the fatigue life of that member.

It should be noted that cover-plated rolled beams exhibit relatively low fatigue resistance. If a designer found that a rolled beam did not have adequate section properties for strength consideration, using a built-up section plate girder would have a much higher fatigue resistance than adding a cover plate to the rolled beam. At a stress range of 100 MPa, the average fatigue life of a cover-plated rolled beam is 1 million cycles while the built-up girder life would be over 8 million cycles.

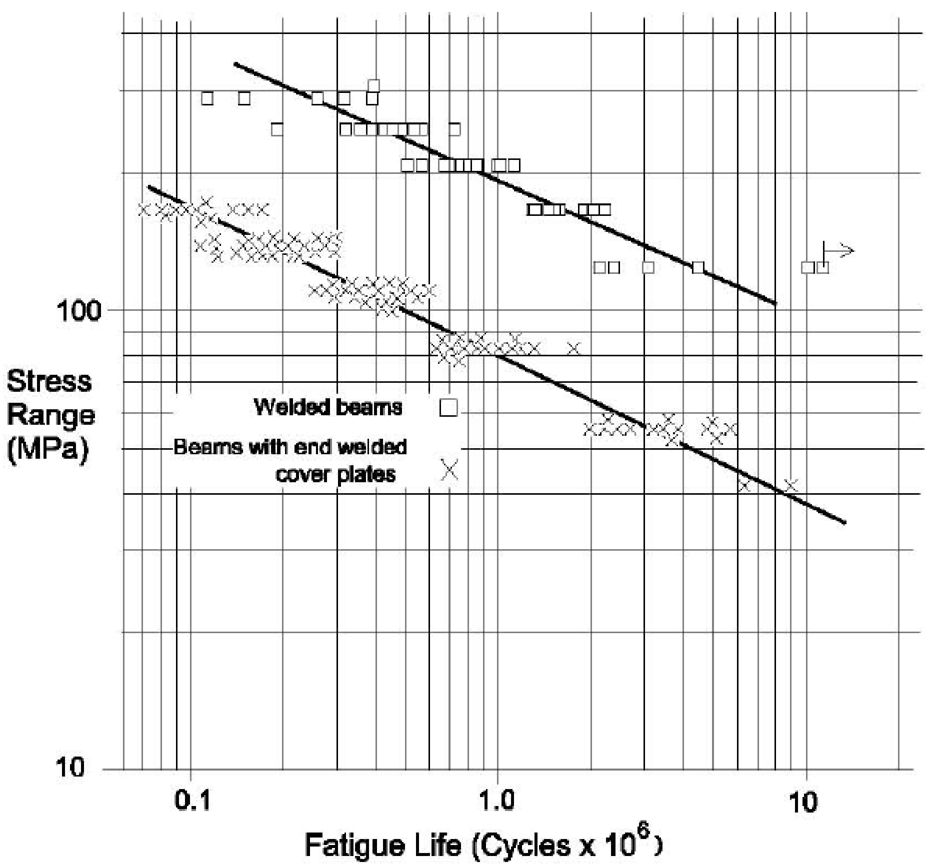


Figure 10 Fatigue Strength of Welded and Cover-plated Beams

In summary, the fatigue life of a fabricated steel structure is determined by three factors. These are:

1. The number of cycles of loading to which the member is subjected;
2. the type of detail under examination; and
3. the stress range at the location of the detail.

It has been implicit in the discussion so far that the stresses, which are the driving force behind crack growth, are those corresponding to the loads on the structure. This is indeed an important case, and for this situation the stress range to be calculated is simply that corresponding to the nominal stress at the location of the detail. This is valid because selection of the detail itself implies inclusion of the stress concentration for that detail. There is another source of stresses in the structure that can produce crack growth, however. This is the stresses (actually strains) that are produced as a result of displacements. Displacement-induced fatigue cracking is as important as load-induced fatigue cracking, and it will be discussed in a separate section.

2.3 Fracture

Unstable crack growth or fracture occurs when the effects of total stress and flaw size exceeds a critical value, commonly referred to as the fracture toughness. Design to preclude fracture is based on proper material selection, or when applicable, the use of a Fracture Control Plan per the AASHTO/AWS-D1.5 Specifications. The designer should choose a steel with a fracture-toughness level that is sufficiently high for the intended application. The fracture toughness depends upon such factors as microstructure and composition of the material, service temperature, loading rate, plate thickness, and fabrication processes. A Fracture Control Plan is a holistic approach that includes everything that affects the potential for fracture, including aspects of design, detailing, materials, fabrication, and inspection.

An accurate determination of the fracture toughness is complicated, especially in most structural engineering design situations. Less sophisticated approaches are used for practical problems in structural engineering. The most widely used method for approximating the toughness quality of a steel is a procedure that was developed over 80 years ago, the Charpy V-Notch (CVN) impact test. In brief, this method measures the energy absorbed by the rapid fracture of a small bar containing a machined notch. The bar is broken by a swinging pendulum and the absorbed energy is measured by the difference in swing height before and after fracture. The effect of temperature is examined by repeating the test using physically identical specimens that have been cooled to various temperatures. Several tests provide a relationship between absorbed energy and temperature for the steel under investigation.

A plot of CVN absorbed energy is shown in Figure 11. The plotted data demonstrates how steel becomes brittle characterized by lower absorbed energies with lower temperatures. The bridge steel specifications require an energy level in the transition region of the curve at temperature that is based upon the lowest service temperature.

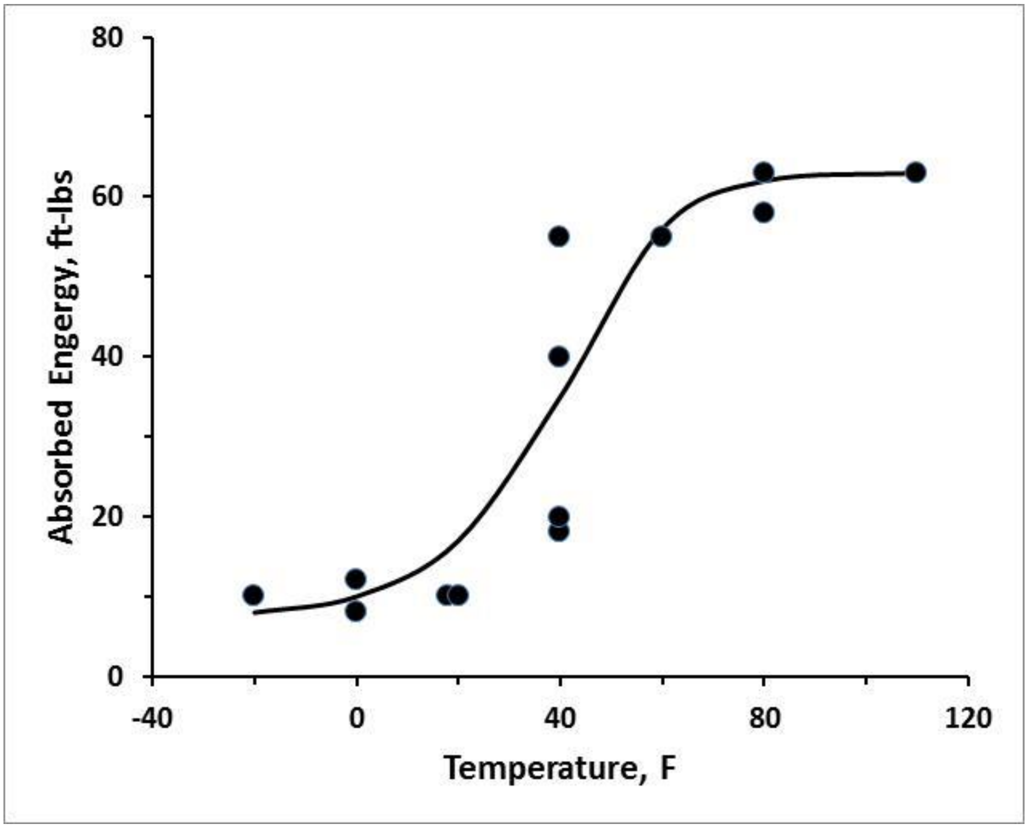


Figure 11 General Charpy V-Notch Absorbed Energy as a Function of Temperature

2.4 Distortion-Induced Fatigue

2.4.1 Introduction

Most of the topics so far have discussed the effect of stresses acting on pre-existing flaws, cracks, and geometrical discontinuities with respect to fatigue lives of fabricated steel elements. The assumption has been that these stresses can be calculated, usually at an elementary level. The loads used are the same as those associated with the strength design of the members. In many instances, however, fatigue crack growth results from distortions not typically considered by designers. Although it is possible in some of these cases to calculate a stress range, this is usually performed after the fact and requires that field measurements be made. Designers are not likely to be able to identify the need for such calculations in the course of their work. As will be seen, this type of fatigue crack growth results from the imposition of relatively small deformations, usually out-of-plane, in local regions of a member. These deformations are not anticipated in the design process. The main defense against this source of cracking is proper detailing, and this, in turn, is dependent on experience.

2.4.2 Examples of Distortion-Induced Fatigue Cracking

2.4.2.1 Cut-Short Transverse Stiffeners

An example of the phenomenon is illustrated in Figure 12. Standard practice for many years was to cut transverse stiffeners short of the bottom (tension) flange so as to prohibit welds on the flange transverse to the direction of stress. Experience gained over the past 20 years has shown that, in fact, the fatigue life of the detail is independent of whether the stiffener terminates in the web or is extended down to the flange. This is reflected in the current AASHTO LRFD Specifications. The only difference, then, is the effect of the difference of the stress in the web and the top of the flange. There are also practical reasons for cutting the stiffener short: the stiffener will have to be made to a precise length if it is to extend from flange to flange although some fabricators prefer fitting or welding the stiffener to the flange to help keep the flange perpendicular to the web. The height of the gap between the end of the stiffener and the girder flange is usually quite small. If lateral movement of the top flange relative to the bottom flange takes place, large strains are imposed in the gap region because of the significant change in stiffness between regions of the web. Typically, the strains that are produced are so large (9) that it may take relatively few cycles for a crack to propagate. The flange movement could be the consequence of transverse forces in a skew bridge, but it could even be due to shipping and handling.

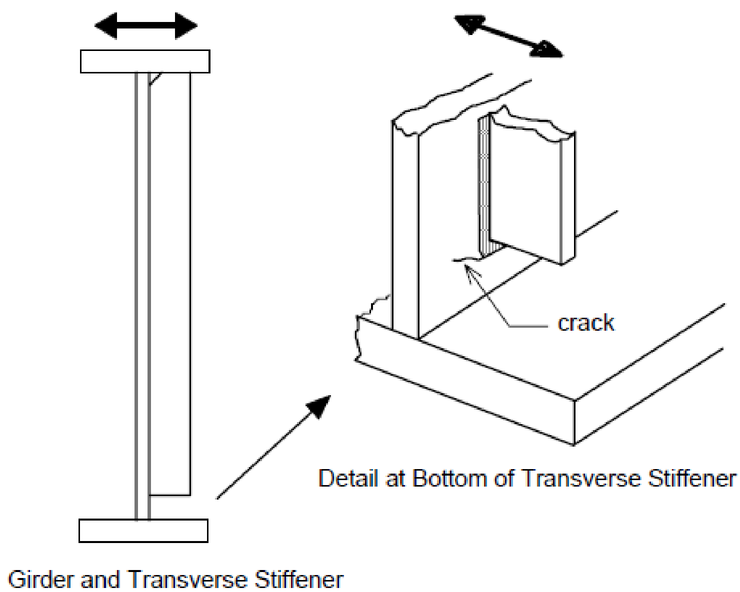


Figure 12 Fatigue Cracking from Out-of-Plane Movement

The detail in Figure 12 shows the crack emanating from the weld toe at the bottom of the stiffener. Often, the crack will also extend across the toe of the fillet weld at the underside of the stiffener and for some distance into the web. Up to this stage, the crack is more or less parallel to the direction of the main stress field that the girder will experience in service. Thus, if the source of the displacement-induced fatigue can be identified and eliminated, then further growth of the crack is unlikely. However, if crack growth has gone on for some time, the crack may have turned upwards or downwards in the web and thus be aligned in the most unfavorable orientation with respect to primary stresses.

The detail just described (Figure 12) has been the source of many fatigue cracks in the past. New designs accommodate the situation by providing rigid attachment of the stiffener to the flange where out-of-plane movement is anticipated, reducing the possibility of fatigue crack growth that is induced in this manner.

2.4.2.2 Floor Beam-to-Girder Connection

Another illustration of a case in which out-of-plane movement can produce fatigue cracks is shown in Figure 13, where a floor beam is attached to a vertical connection plate that is welded to the web of a girder. Under the passage of traffic, the floor beam will rotate as shown. As this rotation occurs, the bottom flange of the floor beam lengthens and the top flange shortens. Lengthening of the bottom flange will not be restrained because it is pushing into the web of the girder, which is flexible in this out-of-plane direction. However, because the top flange of the girder is restrained by the deck slab, shortening of the top flange of the floor beam can only be accommodated by deformation within the gap at the top of the connection plate. This type of deformation is shown (exaggerated) in the detail in Figure 13.

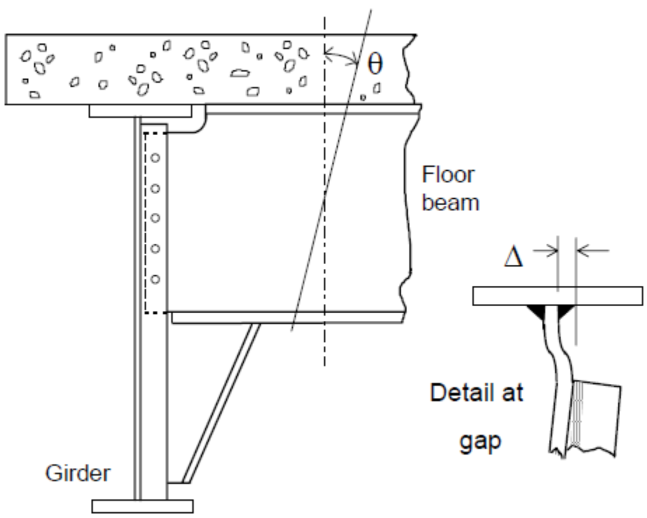


Figure 13 Floor Beam-to-Girder Connection

The behavior illustrated in Figure 13 has been confirmed by field measurements (10). Moreover, the field study showed that each passage of an axle caused a significant stress range at the top of the connection plate. In this situation, fatigue cracks could develop either at the weld at the top of the connection plate or at the web-to-flange fillet weld of the girder, or both. The residual tensile stress in this small gap will tend to be very high because of the proximity of the two welds. It can be expected that fatigue cracks could occur under relatively few cycles of load, although of course the fatigue life will depend largely upon the deformation, Δ, that actually takes place as a result of the rotation of the floorbeam. The deformation can be eliminated in new construction by welding the floor beam connection plate to the girder flange.

2.4.2.3 Web Gaps in Multiple Girder and Girder Floor Beam Bridges

Diaphragms and cross-frames in multi-beam bridges are used to provide torsional stiffness to the structure. The diaphragms or cross-frames are connected to the longitudinal members by means of transverse connection plates, and this often provides fatigue conditions similar to the floor beam-to-girder connection plate discussed above. The connection is usually made to transverse stiffeners that are welded to the girder web. In the past, it was customary that no connection be provided between the stiffener and the tension flange because this would adversely affect the fatigue detail category of the girder. Sometimes these stiffeners were not attached to either flange. Since adjacent beams deflect differing amounts under traffic, the differential vertical movement produces an out-of-plane deformation in the web gap at the stiffener ends if they are not attached to the beam flange. The magnitude of this out-of-plane movement depends on the girder spacing, amount of bridge skew, and type of diaphragm or cross-frame.

Various types of diaphragms are used, ranging from rolled sections to simple X-bracing made of angles. Figure 14 shows the underside of a multiple girder bridge that has an X-type cross-frame bracing system. In the web gap, cracking developed in the negative moment region (i.e., top flange in tension) of this continuous span structure. This cracking is illustrated in Figure 15, which is a view along the length of the bracing toward the girder and its transverse stiffener. The transverse stiffener to which the X-bracing is attached was not welded to the top (tension) flange for the reasons described earlier. This permitted out-of-plane displacement to occur in the web gap and, consequently, cracks formed along the web-to-flange fillet weld and at the top of the connection plate.

Figure 14 X-Bracing and Girders

Figure 15 Fatigue Cracks in Web Gap of Diaphragm Connection Plate

One of the earliest and most common sources of fatigue cracks in welded bridges is the cracking in the web gaps at the ends of floor beam connection plates. These cracks have occurred in the web gap near the end reactions when the floor beam connection plate was not welded to the bottom (tension) flange. However, the most extensive cracking is observed in the negative moment regions of continuous girder bridges where the connection plate is not welded to the top (tension) flange of the girder. One such case is illustrated in Figure 16 where the web gap is indicated by the arrow. The view is toward the web of the girder. Cracking is seen along the fillet weld toe at the web-to-flange weld (horizontal crack) and at the top of the stiffener (vertical crack). The illustration shows that the floor beam has been bolted to the transverse stiffener and that the top of the stiffener has a small corner clip where it reaches the top of the girder web. The clip is provided so that the stiffener clears the girder web-to-flange fillet weld. Thus, even though the stiffener extends to the underside of the girder flange, it is not fastened to the flange and the presence of the stiffener clip creates the gap in which deformation was concentrated. In new construction, the deformation can be eliminated by welding the diaphragm connection plate to the girder flanges.

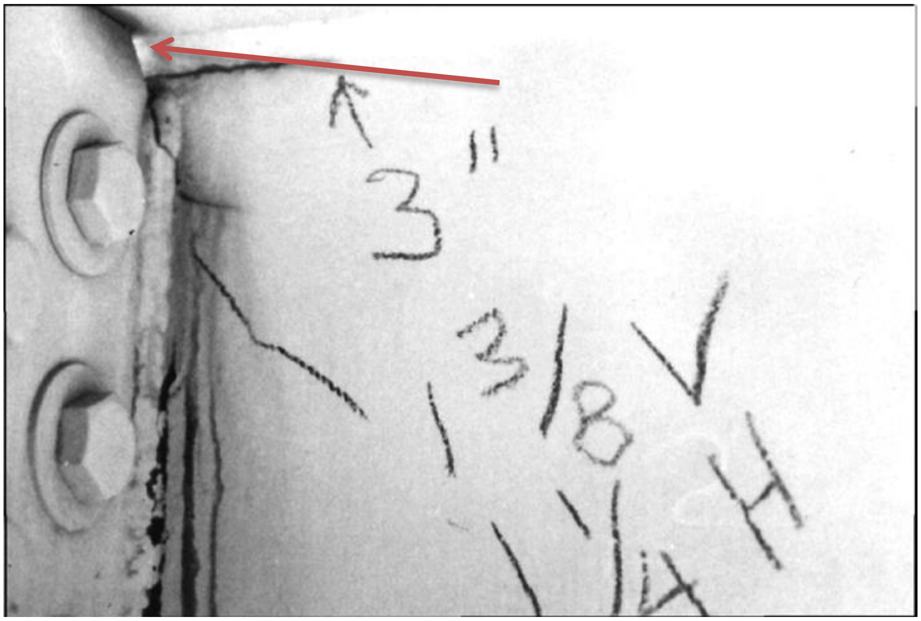


Figure 16 Fatigue Cracks at Floor Beam Connection Plate Web Gap

2.4.2.4 Web Gaps in Box Girder Bridges

Internal diaphragms in various types of box girder structures are a source of web gap cracking as a result of cross-section distortion. This type of cracking has been seen both in continuous and simple span spread box girder structures.

An example of this type of cracking is shown in Figure 17. The structure is an elevated, curved continuous span structure. Two curved steel box girders with internal diaphragms support the reinforced concrete deck. Fatigue cracking occurred in the top web gaps (negative moment region) near the piers and in the bottom web gaps (positive moment region) at the diaphragm locations. In both cases, the cracking was the result of out-of-plane movement in the web gap. This resulted from the box girder distortion and the resultant diaphragm forces.

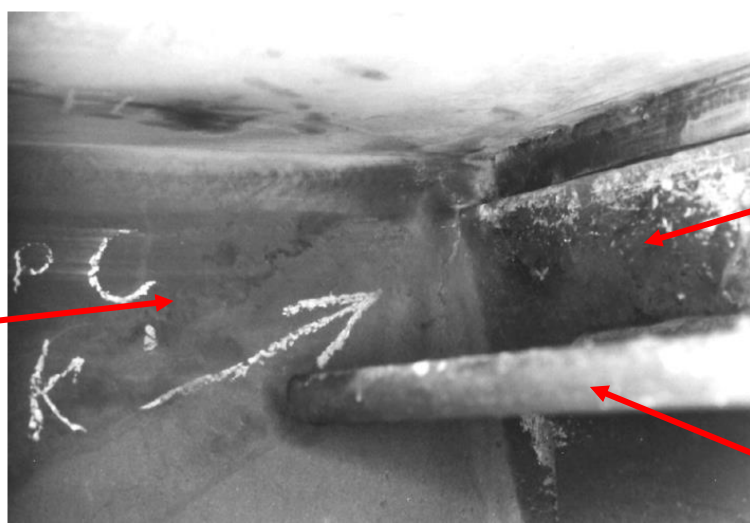


Figure 17 Web Cracking at Box Girder Diaphragm Connection Plate

The cracking that occurred at the top web gap is shown in Figure 17. The photograph, which was taken inside the girder, shows the (sloping) girder web in the left-hand portion of the figure, a transverse connection plate welded at right angles to it, and a gusset plate (horizontal) that formed part of the lateral bracing used for shipping and construction. The gusset plate is about 80 mm below the top of the connection plate and there is a small gap between the top of the transverse connection plate and the underside of the girder flange. (The photograph shows that a loose plate has been placed in this gap above the connection plate. It was later fastened in place in order to prevent further movement of the girder web in this region.)

Fatigue cracking has also been observed in the bottom web gaps at the locations of internal diaphragms in simple span steel box girders. An example of this type of cracking is shown in Figure 18. These cracks are also the result of out-of-plane distortion in the web gap. The web gap displacement results primarily from torsional distortion of the box girder when eccentric loads are applied to the structure and the diaphragm is not connected to the flange. The deformation can be eliminated in new construction by welding the diaphragm connection plate to the flanges.

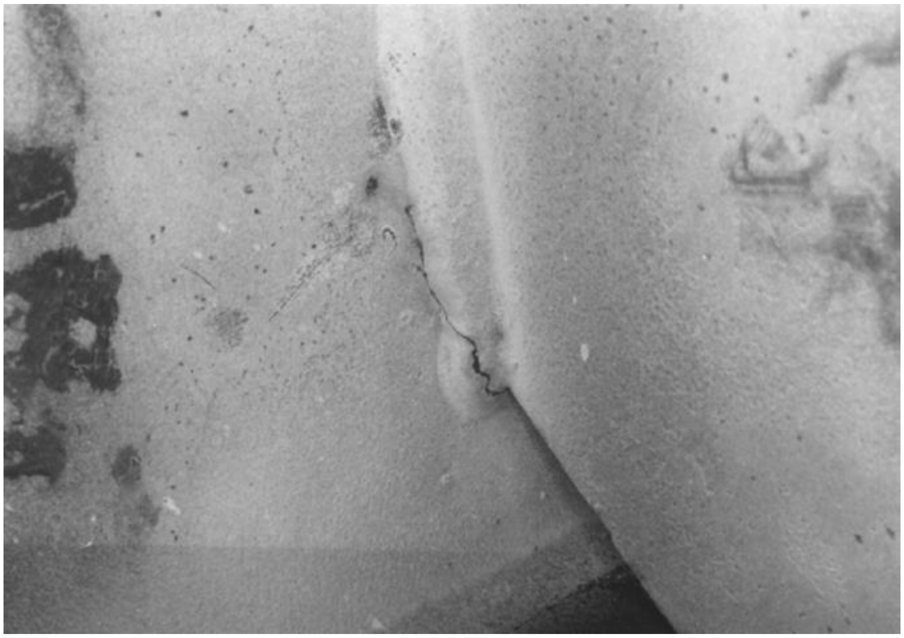


Figure 18 Web Gap Cracking at End of Transverse Connection Plate

2.4.2.5 Coped Beam Connections

In order to facilitate the easy connection of one flexural member to another, the flange of one of the members is often cut back, as is illustrated in Figure 19 (The detail shown was used extensively in the past in through-girder railway spans.) In other cases, the flanges may simply be narrowed: this is called a "blocked" beam. Fatigue cracking at coped beam locations is not so much related to distortion-induced fatigue as it is illustrative of cracking at a location where the calculated stress is zero.

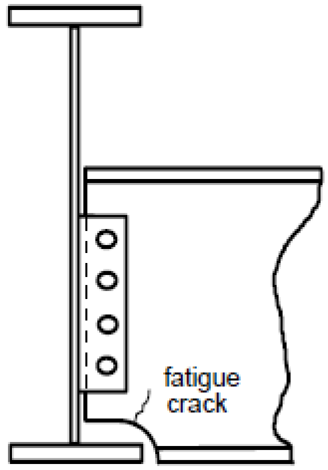


Figure 19 Bottom Flange of Floor Beam Coped at Connection to Girder

In the case of coped beams, either the top or bottom flange, or both, may be coped. The cope is generally made by flame-cutting the material, and experience shows that workmanship is often unsatisfactory. The radius of the cope may be small and the cutting done unevenly. In addition to the potential for fatigue cracking created by such workmanship, the flame-cutting process can leave a region of hardened and brittle material adjacent to the cut.

The coped end of the beam is at a location of theoretical zero stress since the connection must necessarily be one that does not transmit moment and the shear force is carried by the web. Nevertheless, the region of the cope will have stresses due to bending because of the restraint at the connection. There are many examples of fatigue cracking at cope locations (9), and the best solution in the case of new designs is to avoid copes entirely. If copes must be included, execution of the work and inspection must be of a high standard.

2.4.2.6 Connections for Lateral Bracing

The connection of lateral bracing to girders should be done as close to the plane of the bottom flange as possible. (In this discussion, it is assumed that the top flange will be braced by the deck.) Sometimes, the connection between the lateral bracing system and the girder will be to a horizontal plate welded to the girder web. (In the AASHTO LRFD Specifications, this is termed a "lateral connection plate.") If transverse stiffeners are present, usually the case, the arrangement will be as shown in Figure 20.

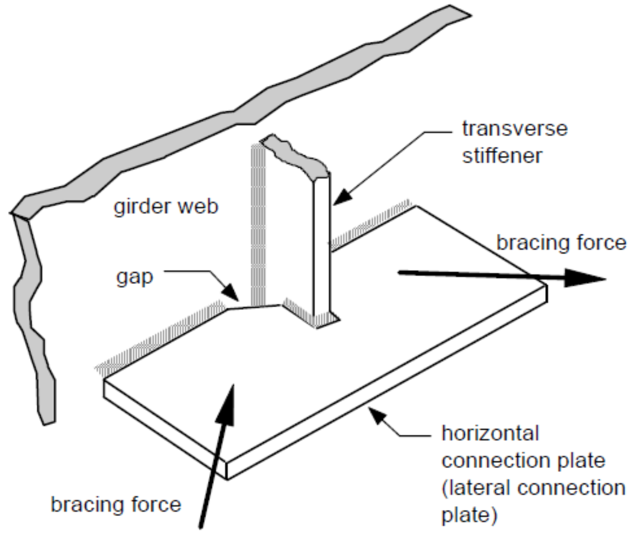


Figure 20 Connection of Lateral Connection Plate

In the illustration, the horizontal connection plate is fitted around the vertical transverse stiffener and welded to it where they are in contact. In other arrangements, the vertical stiffener passes clear through a slot in the horizontal plate without attachment at that location. In either event, it is highly likely that a gap will be left in the region shown in order to avoid intersection of the horizontal and vertical welds. Consequently, as the bracing forces push into and pull on the web at this location, the web will rotate about a vertical axis formed by the back of the vertical stiffener and the plane of the web. Because the web is very flexible in the out-of-plane direction, this causes large strains in the web in the gap region when the lateral connection plate is not attached to the transverse stiffener. The region is also a zone of high residual stresses because of the proximity of the vertical and horizontal welds. Lack of fusion in the weld or other micro-defects can also be anticipated at the weld terminations. Taken all together, these conditions mean that the type of detail shown is very susceptible to fatigue crack growth. Due to the potential for fatigue cracking, when lateral bracing is required, and when the flange width and girder design allows, the designer should consider bolting the lateral-bracing members directly to the flanges.

2.4.2.7 Summary

Out-of-plane distortion that is concentrated in small web gaps remains a large source of fatigue cracking in bridge structures. It develops in nearly every type of bridge structure including trusses, suspension bridges, plate girders, box girders and tied arches. It is fortunate that most of the cracks that develop from local distortion lie in planes parallel to the load-induced stresses. In addition, since stress intensities around distortion-induced cracks may decrease with increasing crack length, cracks can slow, and even stop, once a certain flexibility has been provided. As a result, distortion-induced cracks have not caused significant numbers of fractured flanges or hampered the load-carrying capability of the bridge member in which they form.

3.0 THE AASHTO PROVISIONS

3.1 Load-Induced Fatigue

3.1.1 The Limit-State Function

The limit-state function for load-induced fatigue design, as specified in LRFD Equation 6.6.1.2.2-1, is:

$$\gamma(\Delta f) \leq (\Delta F)_n$$

where

γ	=	appropriate fatigue limit-state load factor, either Fatigue I or Fatigue II
(Δf)	=	force effect, live load stress range due to the passage of the fatigue load as specified in LRFD Article 3.6.1.4
$(\Delta F)_n$	=	appropriate fatigue resistance

The load applied for fatigue design of LRFD Article 3.6.1.4 is not the HL-93 vehicle (design truck or design tandem) and lane superposition used for strength design, but only to the HL-93 design truck with a fixed rear-axle spacing of 30 feet. The 30-foot rear axle spacing represents an average axle spacing as opposed to the variable spacing used for strength. Fatigue is not based upon a single one-off load as strength may be, but the vast majority of average trucks crossing the bridge. Further, a dynamic load allowance (IM) of 15% is applied to the truck, representing a reduction of the 33% used for strength design, again representing average conditions not the extreme IM used in strength design.

3.1.2 Detail Categories

The AASHTO LRFD Specifications defines eight Detail Categories for fatigue: A, B, B′, C, C′, D, E and E′. Figure 21 shows the fatigue-resistance curves given in the LRFD Specifications. The plot shows stress range on the vertical axis and number of cycles on the horizontal axis for the various Detail Categories. Both axes are logarithmic representations. Over some portion of the range, each Detail Category is a sloping straight line with a constant slope equal to 3. Beyond a certain point, which depends on the Detail Category, the fatigue-resistance line is horizontal. This feature will be discussed subsequently.

Equations are presented in the LRFD Specifications for the various Detail Categories. The plot of the Detail Categories appears in the Commentary of the LRFD Specifications. The equations for the design line are given in the body of the specification. Detail Categories are defined through a table of verbal descriptions and sketches, a portion of which is illustrated in Figure 22. The ranking of the Detail Categories is such that Category A has the best fatigue resistance and Category E′ the worst. The numerical entries in the table will be discussed subsequently.

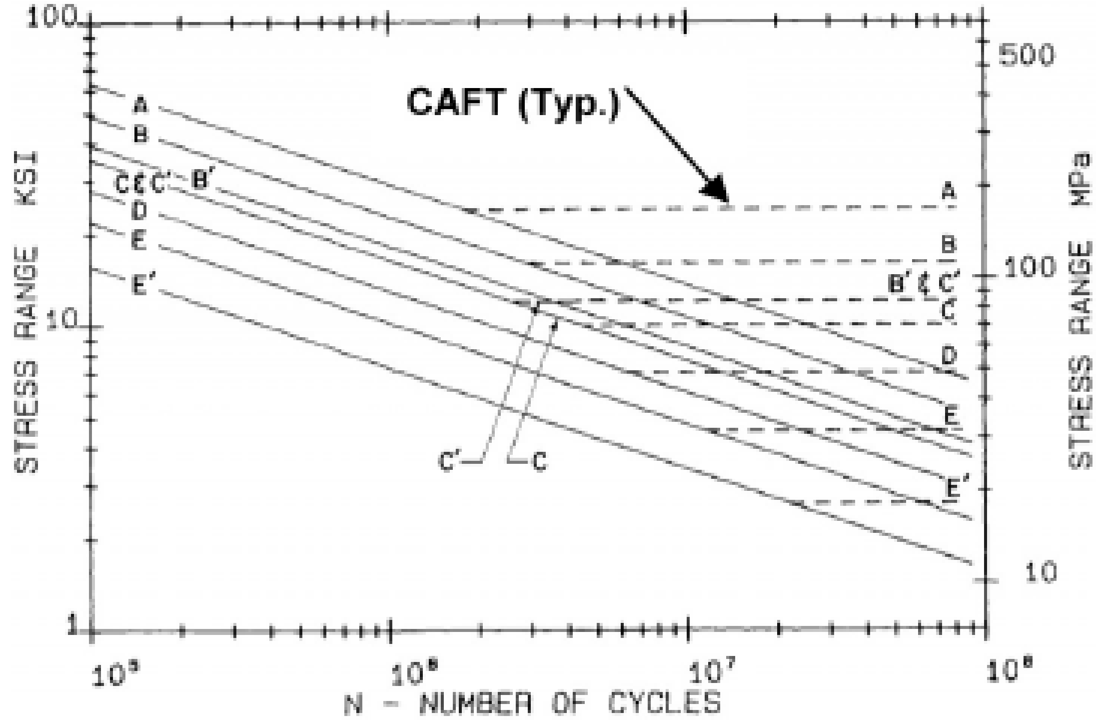


Figure 21 Fatigue Resistance of the AASHTO Detail Categories as Represented in the Commentary to the LRFD Specifications

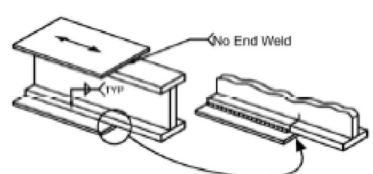
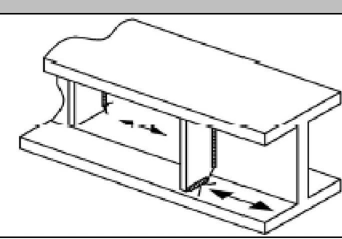
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
3.7 Base metal at the termination of partial length welded cover plates that are wider than the flange and without welds across the ends.	E′	3.9×10^8	2.6	In the edge of the flange at the end of the cover plate weld	
Section 4—Welded Stiffener Connections					
4.1 Base metal at the toe of transverse stiffener-to-flange fillet welds and transverse stiffener-to-web fillet welds. (Note: includes similar welds on bearing stiffeners and connection plates).	C′	44×10^8	12	Initiating from the geometrical discontinuity at the toe of the fillet weld extending into the base metal	

Figure 22 Tabularized Detail Category Descriptions as Given in the LRFD Specifications

3.1.3 Infinite –Life Design

The Fatigue I limit-state should be checked first. Satisfaction of the Fatigue I limit state should theoretically provide an infinite fatigue life for the detail in question. This limit state does not require the average daily truck traffic (ADTT) which should be averaged over the life of the bridge, to date and into the future. If this limit state is not satisfied, the Fatigue II limit-state can be investigated with an estimated ADTT averaged over the life of the bridge.

3.1.3.1 Load-Side of the Function

Many bridge details exhibit a fatigue threshold such that if all applied stress ranges are kept below this threshold value of stress range, the detail will not crack during its design life but will theoretically exhibit infinite fatigue life. The Fatigue I limit-state load combination is intended to represent infinite-life fatigue design. The Fatigue I load factor on live load of 1.5 represents the stress range due to the heaviest truck that needs to be considered for fatigue. It is not the absolute heaviest truck. The live-load load factor of 1.5 was derived as the 1-in-10,000 greatest stress range experienced by a bridge detail.

3.1.3.2 Resistance-Side of the Function

The fatigue resistance for the Fatigue I limit-state consists of the straight, horizontal lines in Figure 21. These horizontal lines constitute the constant-amplitude fatigue threshold (CAFT) as depicted in the figure. The threshold values for the various Detail Categories are tabularized in the LRFD Specifications and given in Table 1.

Table 1 Constant-Amplitude Fatigue Thresholds

Detail Category	Constant-Amplitude Fatigue Threshold (ksi)
A	24
B	16
B′	12
C	10
C′	12
D	7
E	4.5
E′	2.6

3.1.4 Finite –Life Design

If the Fatigue I limit state cannot be satisfied, the Fatigue II limit-state should be checked. LRFD Table 6.6.1.2.3-2 presents the 75-year ADTT in a single lane for each Detail Category below which the Fatigue II limit state for finite-life design governs. Note that these values of ADTT are relatively low (less than 2000 trucks per day) for most categories.

3.1.4.1 Load-Side of the Function

Fatigue damage does not accumulate significantly due to the relatively small number of heavy trucks but more so due to the vast number of trucks of more typical gross-vehicle weight. Thus, the Fatigue II limit-state load factor on live load included in the LRFD Specifications is less than one, specifically 0.75. Further, this load factor is not applied to the HL-93 vehicle and lane superposition, but only to the design truck with a fixed rear-axle spacing of 30 feet. The factored stress range from this load factor and truck represents the most typical truck. This factored stress range is used to design bridge details to exhibit a finite fatigue life based upon the average daily truck traffic (ADTT). The live-load load factor of 0.75 was derived as the root-mean-cube of the stress ranges experienced by a bridge detail.

3.1.4.2 Resistance-Side of the Function

The sloping portion of the fatigue-resistance curves of Figure 21 represent the fatigue resistance for the Fatigue II limit state for finite-life fatigue design. For finite-life fatigue design, the fatigue resistance is a function of the number of cycles as specified in the LRFD Equation 6.6.1.2.5-2 as:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}}$$

in which:

$N = (365)\,(75)\,n\,(ADTT)_{SL}$

where:

A = a constant given in Table 2

n = number of stress range cycles per truck passage, as tabulated in the LRFD Specifications

$(ADTT)_{SL}$ = single-lane ADTT as specified in the LRFD Specifications

The significance of the exponential relationship of this equation is that a small change in stress range produces a big change in fatigue life.

Table 2 Detail Category Constant, A

Detail Category	Constant A
A	250×10^8
B	120×10^8
B′	61×10^8
C	44×10^8
C′	44×10^8
D	22×10^8
E	11×10^8
E′	3.9×10^8

3.2 Distortion-Induced Fatigue

Based on the discussion above on distortion-induced fatigue, it can easily be appreciated that it is difficult for a specification or design standard to provide very much in the way of explicit rules relating to distortion-induced fatigue. The AASHTO LRFD Specifications provides a separate section on distortion-induced fatigue (LRFD Article 6.6.1.3). It contains a general statement that stresses the importance of proper connection of transverse components to longitudinal (i.e., main) components. Sub-sections then offer specific information relating to transverse connection plates (LRFD Article 6.6.1.3.1), lateral connection plates (LRFD Article 6.6.1.3.2), and orthotropic decks (LRFD Article 6.6.1.3.3).

LRFD Article 6.6.1.3.3 simply directs the reader to LRFD Article 9.8.3.6, which gives detailing requirements for orthotropic decks. Those requirements are mainly a reflection of good practice and experience derived from this type of deck construction. The articles relating to connection plates provide rules for this important topic of girder detailing. Cracking resulting from improper detailing of connection plates is a significant source of fatigue cracking in bridges. LRFD Article 6.6.1.3 also alerts the designer to the possibility of fatigue cracking as a result of excessive out-of-plane flexing of a girder web.

3.3 Fracture Control

Fracture of steel bridge members is typically precluded through material selection by specifying adequate minimum levels of toughness for the various grades of bridge steels based upon minimum expected temperatures and material thickness. Also, a Fracture Control Plan, per AASHTO/AWS-D1.5 Specifications, can provide a higher level of safety for fracture critical bridges. A Fracture Control Plan is a holistic approach that includes everything that affects the potential for fracture, including aspects of design, detailing, materials, fabrication, and inspection.

The designer is responsible for determining which, if any, component is a fracture-critical member (FCM). The LRFD Specifications defines a fracture-critical member as a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function. The location of all FCMs shall be clearly indicated on the contract plans.

Three temperature zones specified in the LRFD Specifications are based upon minimum service temperature. Higher CVN impact energy requirements are specified for higher grades of steel, thicker material and lower service temperature. Requirements for fracture-critical members are more severe than for non-fracture-critical members. These CVN impact energy requirements are tabulated in LRFD Table 6.6.2-2.

CVN impact energy testing requirements vary along a continuous girder. In the positive-moment region (in other words in the spans), the bottom flange is in tension and requires testing, while in the negative-moment region (in other words over the piers), the top flange is in tension and requires testing.

4.0 REFERENCES

1. Fisher, J.W., Kulak, G.L. and Smith, I.F.C., A Fatigue Primer for Structural Engineers, National Steel Bridge Alliance, 1998.

2. American Association of State Highway and Transportation Officials, "AASHTO LRFD Bridge Design Specifications," Seventh Edition, Washington, D.C., 2014.

3. Wöhler, A., Über die Festigkeitsversuche mit Eisen and Stahl, Zeitschrift für Bauwesen Volume 20, 1870.

4. Munse, W.H. and Grover, L.M., Fatigue of Welded Steel Structures, Welding Researcj Council, New York, 1964.

5. Fisher, J.W. and Viest, I.M., "Fatigue Life of Bridge Beams Subjected to Controlled Truck Traffic," Preliminary Publication, 7th Congress, IABSE, 1964.

6. Gurney, T.R., Fatigue of Welded Structures, 2nd Edition, Cambridge University Press, 1979.

7. Fisher, J.W., Frank, K.H., Hirt, M.A., and McNamee, B.M., "Effect of Weldments on the Fatigue Strength of Steel Beams," National Cooperative Highway Research Program Report 102, Highway Research Board, Washington, D.C., 1970.

8. Fisher, J.W., Albrecht, P.A., Yen, B.T., Klingerman, D.J., and McNamee, B.M., "Fatigue Strength of Steel Beams With Transverse Stiffeners and Attachments," National Cooperative Highway Research Program, Report 147, Highway Research Board, Washington, D.C., 1974.

9. Fisher, J.W., Fatigue and Fracture in Steel Bridges: Case Studies, Wiley Interscience, 1984.

10. Fisher, J.W., "Fatigue Cracking in Bridges from Out-of-Plane Displacements," Canadian Journal of Civil Engineering, Vol. 5, No. 4, December 1978.