Steel Bridge Design Handbook

CHAPTER 12

Design for Fatigue

February 2022

Smarter.
Stronger.
Steel.

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Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

TECHNICAL REPORT DOCUMENTATION PAGE

1. Title and Subtitle	2. Report Date
Steel Bridge Design Handbook Chapter 12: Design for Fatigue	February 2022
3. Original Author(s)	**4. Revision Author(s)**
Dennis Mertz, Ph.D., P.E. (University of Delaware)	Michael A. Grubb, P.E (M.A. Grubb & Associates, LLC)
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7. Supplementary Notes

The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.

8. Abstract

Fatigue in metals is the process of initiation and growth of cracks under the action of repetitive tensile loads. If crack growth is permitted 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 and the crack extends in an unstable mode. The fatigue process can take place at stress levels 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. Fatigue cracking can also occur due to stresses resulting from out-of-plane distortions not typically considered by the designer when proper detailing practices are not followed. Consequently, the types of civil engineering applications that are susceptible to fatigue cracking include structures such as bridges.

Unstable crack growth or fracture occurs when the effects of total stress and flaw size exceed a critical value, commonly referred to as the fracture toughness. The designer should choose a steel with a fracture-toughness level that is sufficiently high for the intended application. The determination of the required minimum level of fracture toughness and detailing to avoid conditions susceptible to so-called “constraint-induced fracture” are discussed herein. The proper classification of Fracture-Critical Members (FCMs) and System Redundant Members (SRMs) is also discussed.

This chapter provides the practicing engineer with the background required to understand the basics of fatigue and fracture and use the design rules for fatigue resistance that are currently a part of the AASHTO LRFD BDS for fabricated steel-bridge structures.

9. Keywords	10. AISC Publication No.
Stress Range, Load-Induced Fatigue, Distortion-Induced Fatigue, Residual Stress, Fracture Toughness, Infinite Life, Finite Life, Constraint-Induced Fracture	B912-22

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 permitted 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 and 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, cantilever and overhead sign structures, lighting and traffic signal supports, 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. Steel structures are inspected for cracks, both during fabrication to limit the size of initial flaws and during service to ascertain any crack growth. However, it is almost 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 cracking and subsequent brittle fracture. The fatigue behavior of a fabricated steel structure is affected by the presence of pre-existing cracks or crack-like discontinuities, which most often occur at welded connections or other areas of stress concentration.

Bridge are probably the most common civil engineering structures that must be designed for fatigue. 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 were relatively small. In addition, the magnitude and frequency of loads 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 has two principal effects related to fatigue. First, welding often introduces more critical stress concentrations and flaws than does bolting or riveting, thus producing a more severe initial crack situation. Second, the continuity between structural elements inherent in welded construction allows for a crack in one element to propagate into an adjoining element.

As welding was being introduced, design rules were initially 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 resistance: reliance on small-scale specimens can result in overestimates of fatigue resistance.

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

This chapter of the Steel Bridge Design Handbook 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 chapter are taken with permission directly from a more extensive primer on fatigue [1], which can be reviewed for a more complete discussion on fatigue and fracture. For even greater detail, it is recommended that the reader also review the FHWA *Design and Evaluation of Steel Bridges for Fatigue and Fracture Reference Manual* [2].

This chapter presents fundamental fatigue design principles, but for specific design applications the reader is referred to the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* [3], referred to hereafter as the AASHTO LRFD BDS. For guidance on remaining fatigue life analysis of existing bridges, refer to the AASHTO *Manual for Bridge Evaluation* [4].

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 [5]. 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 [6].

Welded details were first examined in the 1930s 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 [6]. Fatigue cracks forming in steel bridges at a road test program conducted in the USA in the 1960's [7] 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 [8] and in the USA [9, 10, and 11] in the late 1960s and early 1970s. Although many other investigators have contributed to the 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. This greatly simplified fatigue design. These studies also revealed the necessity to acknowledge that fabricated steel structures virtually always contain cracks or crack-like discontinuities.

Steel structures are inspected during fabrication to identify cracks and larger discontinuities that are not permitted in bridges. However, small crack-like discontinuities, such as those occurring at the intersection of the fusion line of the weld and the plate surface at the weld toe, are typically 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.

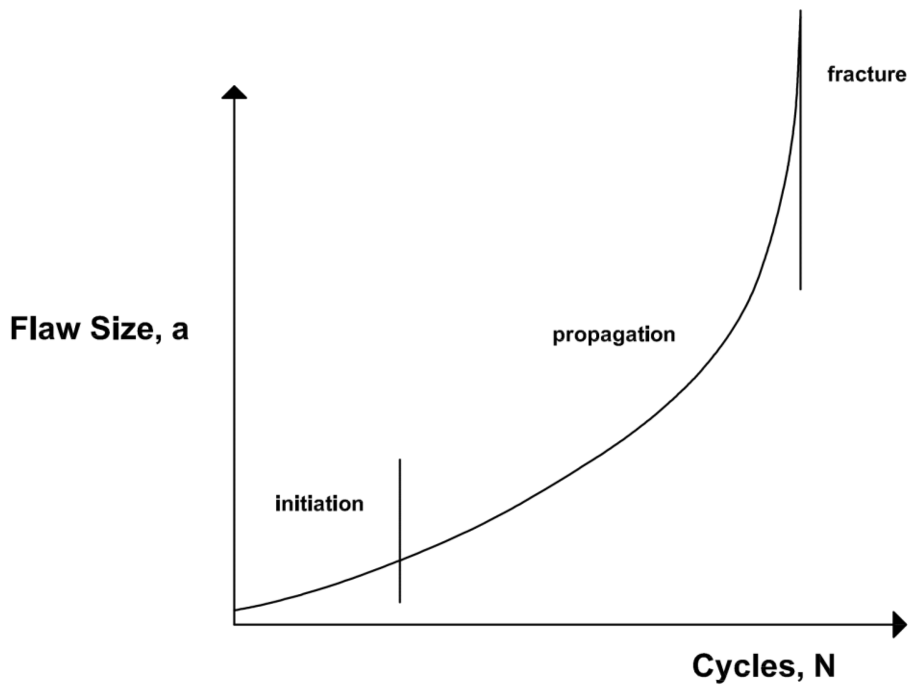


Figure 1 Regimes of Crack Growth

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

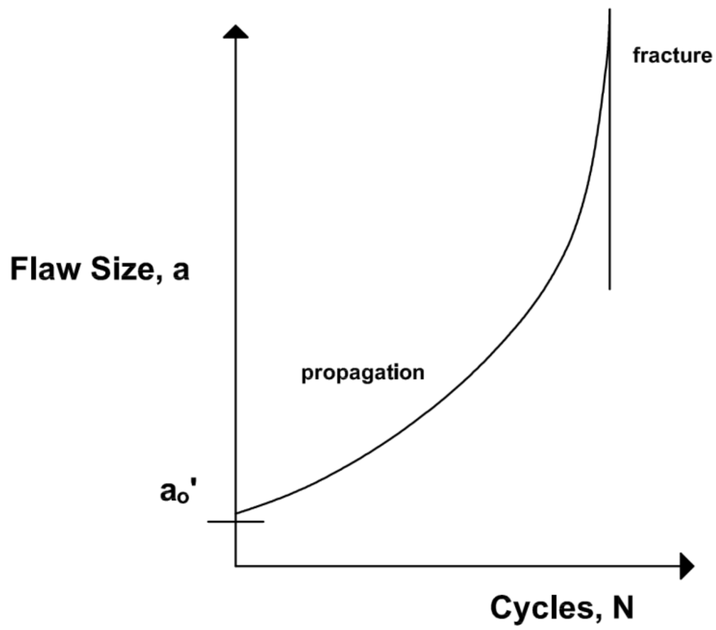


Figure 2 Regimes of Crack Growth in Bridges

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 the fracture toughness of the material, which varies with the 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 ASTM A709/A709M material specification for bridge steels (AASHTO LRFD BDS Table C6.6.2.1-1).

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 is 3/8-inches thick, is fillet-welded to 1/2-inch-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) has grown outward toward the free surface under the repeated application of stress until the crack penetrates 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.

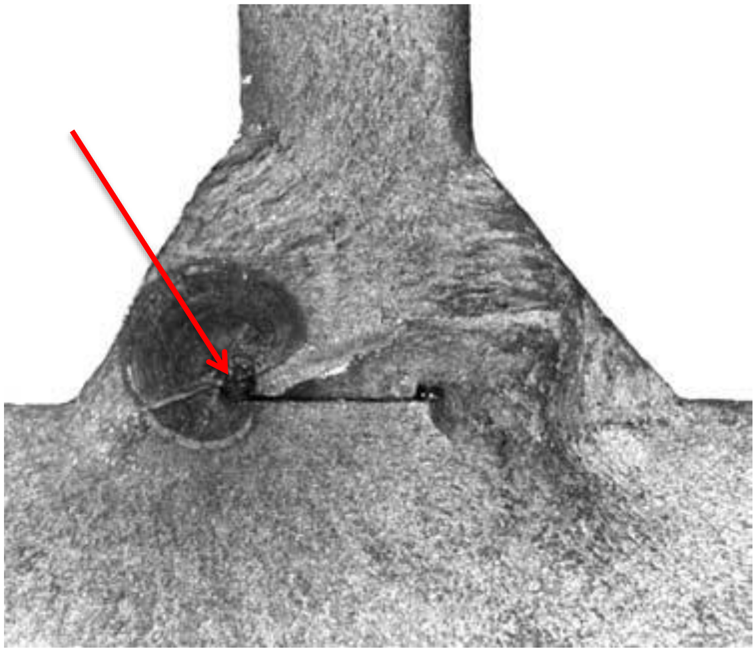


Figure 3 Fracture Surface of I-shaped Cross Section

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. In this case, 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 would continue to grow, resulting in a loss of cross-section such that the load could no longer be carried by the uncracked portion of the beam. In this case, failure would typically occur 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 could fail would be 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; 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 work to minimize these defects, it is neither practical nor economically possible to eliminate them altogether.

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 fatigue-design provisions of the AASHTO LRFD BDS were based upon tests of real welds with porosity flaws in them.

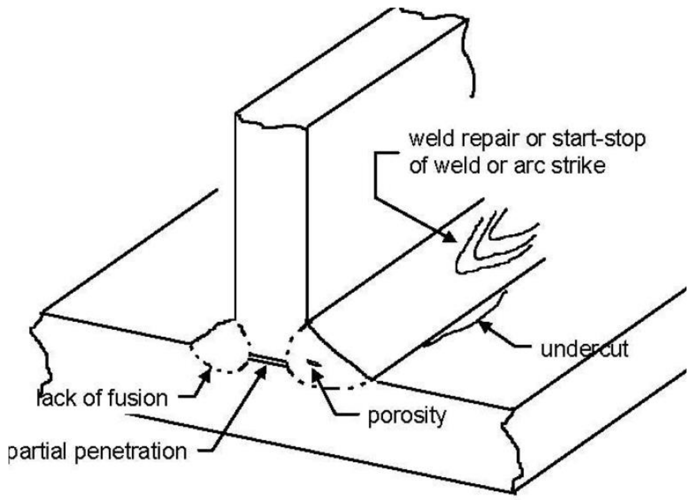


Figure 4 Discontinuities in a Fillet-Welded Detail

Test data on welded details have demonstrated that fatigue cracks typically 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 (i.e., 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. Figure 5 and Figure 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 fatigue resistance design provisions, which eliminates the need for an internal inspection of fillet welds.

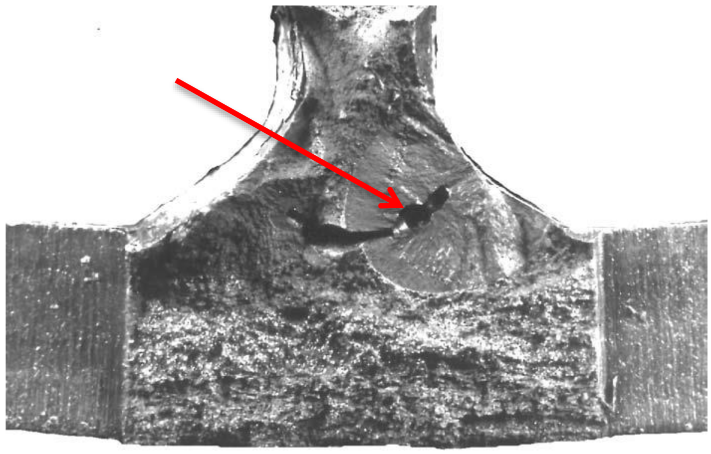


Figure 5 Fatigue Crack Forming from Internal Porosity in Web-Flange Connection

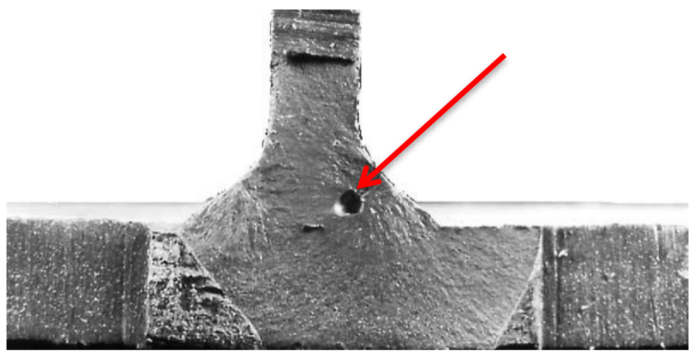


Figure 6 Fatigue Crack Enlarged to Three-Ended Crack from Internal Porosity

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. Figure 7 and Figure 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 the fracture surface suggesting more rapid propagation.

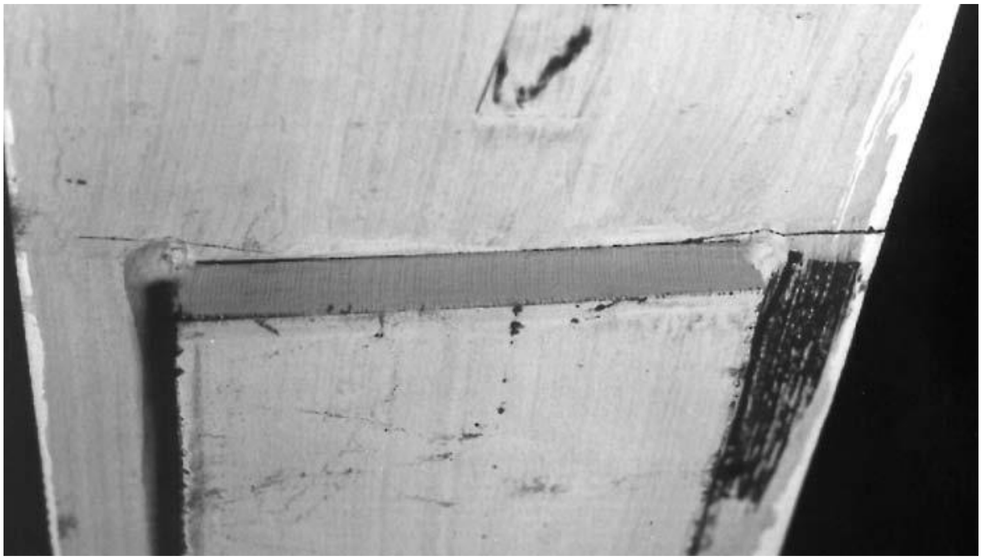


Figure 7 Fatigue Crack at End of Cover Plate Fillet Weld Toe

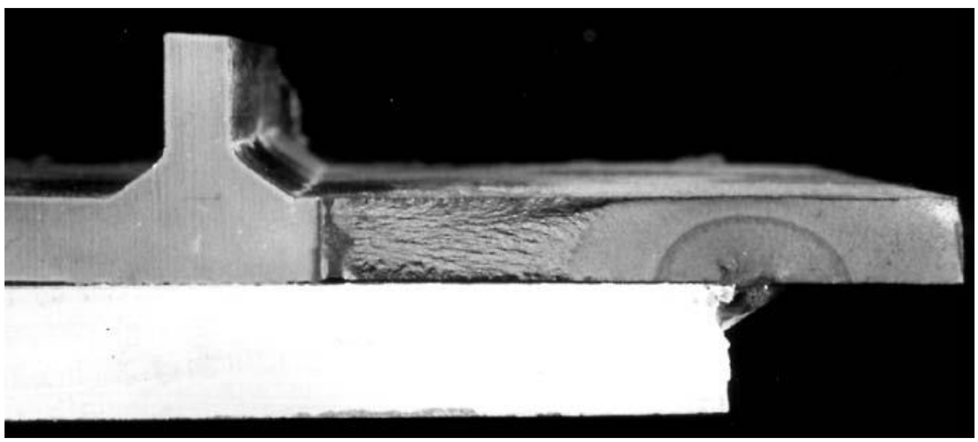


Figure 8 Crack Surface Showing Fatigue Crack Growth

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 resulting crack-like feature represented by the lack of penetration is parallel to the direction of the (bending) stress field, the crack is not expected to open under the application of expected design stresses and failure by fatigue is not anticipated to 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 resistance 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 attachment, connection, or 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 exhibit 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 pretensioning of the bolt. Punched holes exhibit 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 have already been discussed. In addition to the fact that more flaws will be present when welding is used, inspection for defects is typically more difficult than when mechanically fastened details are used. Repairing defects in welded details is also typically more difficult. Prohibiting the use of welded details in fatigue situations, however, is often not a practical or economical option.

The task of the structural engineer is 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 an environment in which 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 with the expectation that flaws will be present initially in fabricated steel structures and that such members will contain residual stresses of relatively high magnitude. A concomitant feature is that in the design process it is possible to identify permissible flaw sizes and use that information to formulate acceptance criteria for both initial inspection of the structure as well as periodic in-service inspections. Criteria for in-service inspections are not yet well-developed in design specifications, and the usual procedure is to accept as permissible flaw sizes that are consistent with fabrication specifications, e.g., the welding specifications.

2.2 Load-Induced Fatigue

2.2.1 Stress Range as the Dominant Stress Parameter

Welded steel structures 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 [5]. Furthermore, fatigue resistance is independent of steel grade, so the use of the relatively large database of laboratory results taken from tests of different steel grades produced in different countries is justified. 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 subjects the weld and a relatively small volume of plate adjacent to the weld to tensile stress. Conversely, the main portion of the plate is compressed by the contracting weld, thereby subjecting it to compressive stress. The stresses resulting 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. These residual stresses are not insignificant; the magnitude of the tensile residual stress can reach the yield strength of the material.

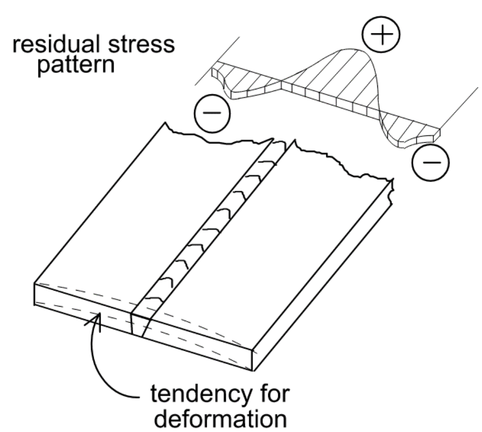


Figure 9 Residual Stresses in Groove-weld Connected Plates

As a result of analogous differential rates of cooling, both rolled shapes and built-up, welded members contain regions of high residual tensile stress. For example, 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, in which this initial stress is at or near the yield stress level, this means that the stress range is the governing condition affecting fatigue crack growth, rather than the maximum applied stress, the stress ratio (ratio of maximum stress to minimum stress), or some other parameter of applied stress.

To summarize, in larger welded structures high tensile residual stresses occur near potential fatigue crack sites and their presence significantly reduces the effects of the mean level of the applied stresses and the 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

In the AASHTO LRFD BDS fatigue design approach, standard structural details are arranged into categories relative to their expected fatigue life, based on nominal stress ranges. For example, Figure 10 illustrates the fatigue life representations for two different member 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. Using the information shown in Figure 10, it would be possible for a designer of one of those two types of member categories 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 considerations, changing to a built-up section plate girder would produce better fatigue performance than adding a cover plate to the rolled beam. At a stress range of 100 MPa (approximately 15 ksi), the average fatigue life of a cover-plated rolled beam would be 1 million cycles while the built-up girder life would be over 8 million cycles.

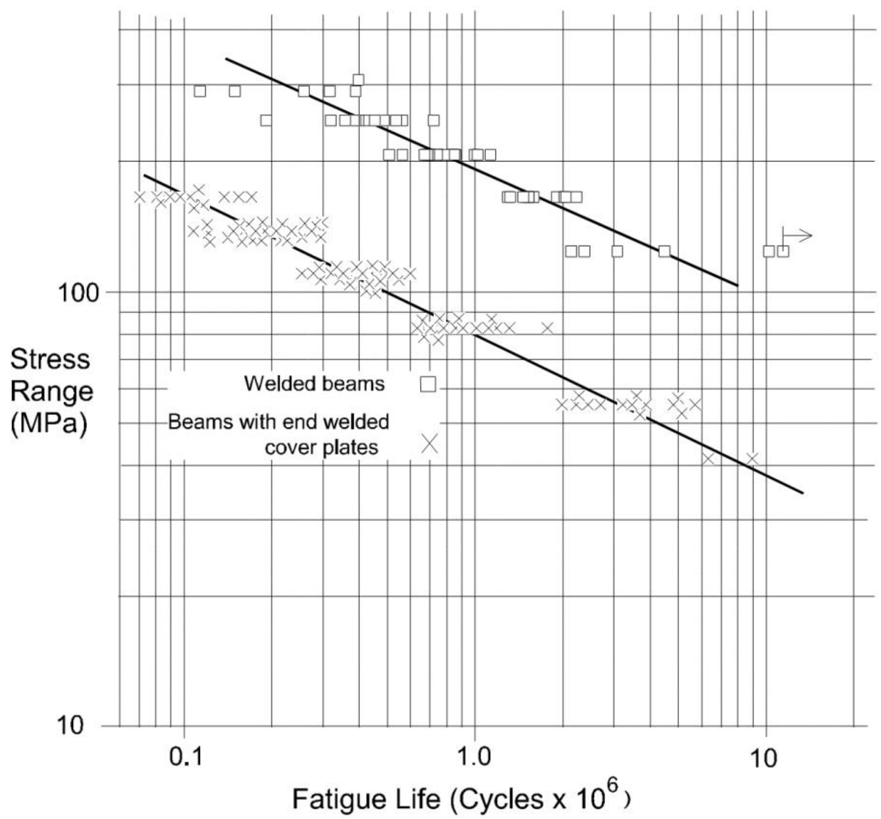


Figure 10 Fatigue Strength of Welded and Cover-plated Beams

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 anticipated design loads applied to the structure. This is indeed an important case, and for this situation the calculated stress range corresponds to the range of applied force effects 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—the secondary stresses (actually strains) produced by displacements of the structural system. Distortion-induced fatigue cracking is as important as load-induced fatigue cracking, and it will be discussed in Section 2.4.

2.3 Fracture

2.3.1 General

Unstable crack growth or fracture occurs when the effects of total stress and flaw size exceed 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 (Clause 12) [12]. 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 a temperature that is based upon the lowest service temperature (Section 3.3.1).

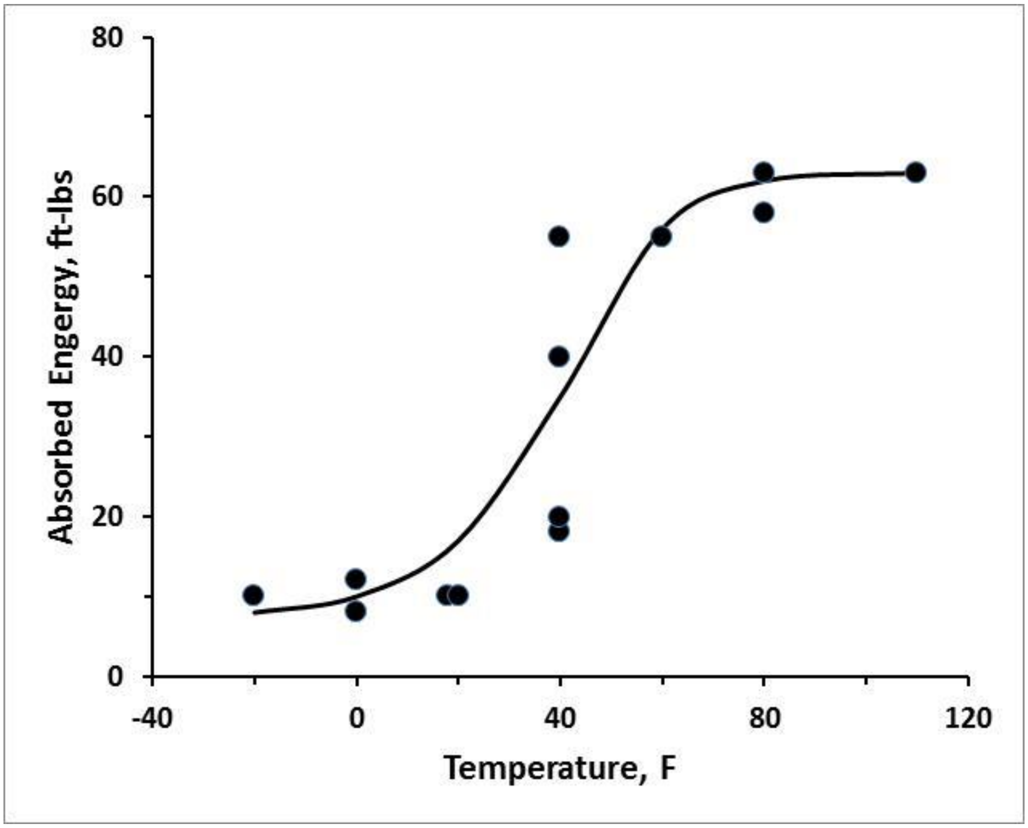


Figure 11 General Charpy V-Notch Absorbed Energy as a Function of Temperature

2.3.2 Constraint-Induced Fracture

Details in steel bridges are often welded, and by their nature frequently involve the intersection of multiple elements. In certain situations, these details can be subject to a high degree of constraint at the intersection and can potentially be at elevated risk of so-called "constraint-induced fracture"; a sudden and brittle form of failure that can occur without any perceptible form of crack growth. Identifying and avoiding or mitigating details at risk of constraint-induced fracture is an important aspect of bridge design, fabrication, and inspection.

In general, three conditions acting together contribute to an elevated risk of constraint-induced fracture [13]: 1) a high level of net tensile stress including consideration of residual stresses; 2) a high degree of constraint in which the joint is subject to a three-dimensional state of stress that prevents local yielding (sometimes referred to as "a high degree of triaxial constraint"), and; 3) a crack-like or notch-like plane of discontinuity *approximately perpendicular* to the primary flow of tensile stress, which produces a stress concentration and provides a potential fracture initiation point. Note that the presence of intersecting welds does not necessarily equate to an elevated susceptibility to constraint-induced fracture, particularly when the detail features fewer than three intersecting welded steel elements; for example, the intersection of flange-to-web welds with welded flange or web shop splices.

Constraint-induced fracture was documented during investigations of failures of the Hoan Bridge and SR 422 over the Schuylkill River [14,15]. Criteria have been developed to identify bridges and details susceptible to this failure mode [16]. Conditions susceptible to constraint-induced fracture can be avoided by following good detailing practices. These detailing practices are simple and well-defined in the literature, including in the FHWA *Design and Evaluation of Steel Bridges for Fatigue and Fracture Reference Manual* [2] and in the AASHTO LRFD BDS [3] (Section 3.3.3).

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; this type of phenomenon is called "distortion-induced fatigue." Although it is possible in some of these cases to calculate a stress range, this is usually performed after the fact and requires sophisticated refined analysis models and/or field measurements. Designers do not typically identify the need for such calculations in the course of their regular design 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 difficult to estimate in the design process. The main defense against this source of cracking is proper detailing.

2.4.2 Examples of Distortion-Induced Fatigue Cracking

2.4.2.1 Cut-Short Transverse Stiffeners

An example of distortion-induced fatigue is illustrated in Figure 12. In years past, it was standard practice to cut transverse stiffeners short of the bottom (tension) flange to prohibit welds on the flange transverse to the direction of stress. Experience gained over a number of 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 BDS. 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, as might commonly be the case if these cut-short stiffeners are used to connect cross-frames and diaphragms (see Section 2.4.2.3), large strains can occur in the region of the gap between the end of the stiffener and the top of the flange because of the significant change in stiffness between regions of the web. The resulting out-of-plane bending strains in the web can become so large [11] that it may take only a relatively small number of cycles for a crack to propagate. The flange movement could be the consequence of transverse forces in a skewed bridge, but it could even be due to shipping and handling.

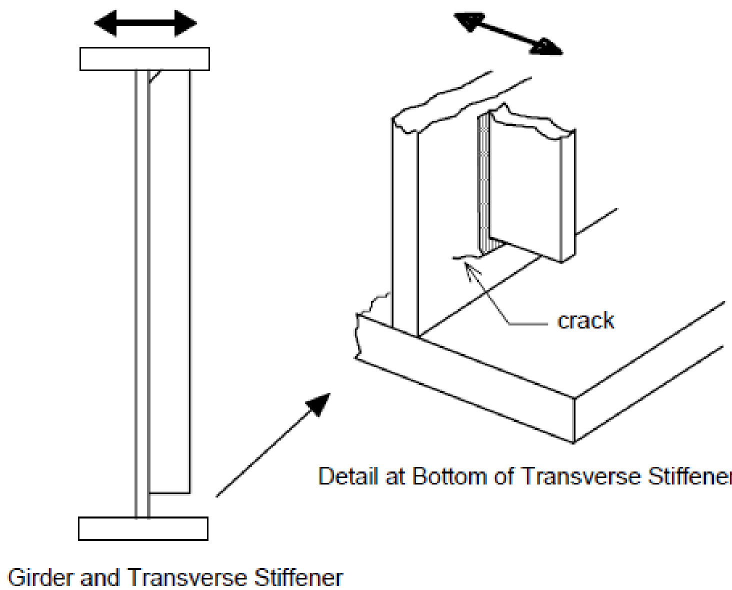


Figure 12 Fatigue Cracking from Out-of-Plane Movement

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 distortion-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, becoming oriented perpendicular to the direction of tensile stress associated with primary bending in the girder.

The detail shown in Figure 12 has been the source of many fatigue cracks in the past. New designs accommodate the situation by requiring 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 out-of-plane deformation of the girder web within the gap at the top of the connection plate. This type of deformation is shown (exaggerated) in the detail in Figure 13.

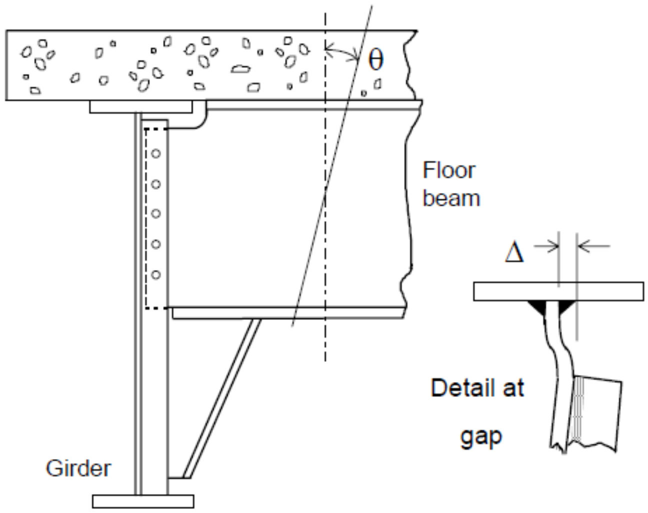


Figure 13 Floor Beam-to-Girder Connection

The behavior illustrated in Figure 13 has been confirmed by field measurements [17]. Moreover, the field study showed that each passage of an axle caused a significant out-of-plane bending stress range in the girder web 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 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 floor beam. The deformation can be eliminated in new construction by attaching 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 flange. 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 girder 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- or K-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 diagonal bracing member looking 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 girder web. 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 positively attaching the diaphragm connection plate to the girder flanges. AASHTO LRFD BDS Article 6.6.1.3.1 does permit less than full-depth end angles or connection plates to be bolted or welded to the beam web to connect intermediate diaphragms in straight rolled-beam bridges with limited skew under certain conditions (Section 3.2).

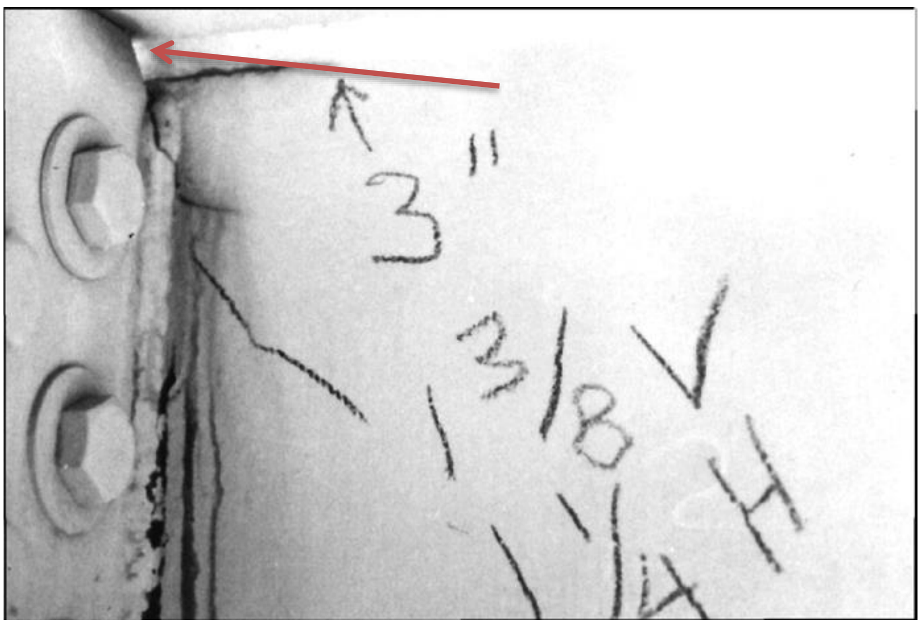


Figure 16 Fatigue Cracks at Floor Beam Connection Plate Web Gap

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 resulting from 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.

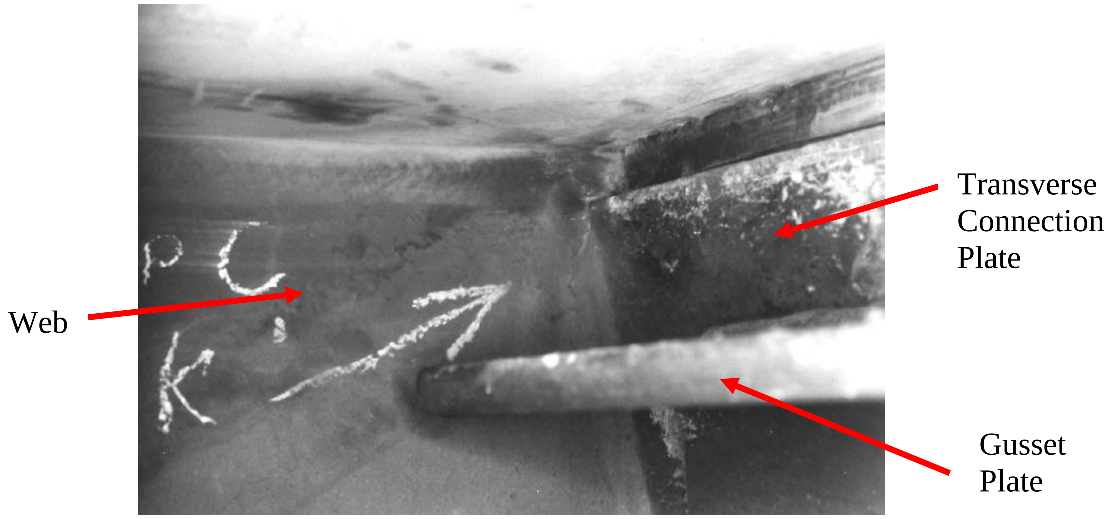


Figure 17 Web Cracking at Box Girder Diaphragm Connection Plate

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 3-1/8 inches 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 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.

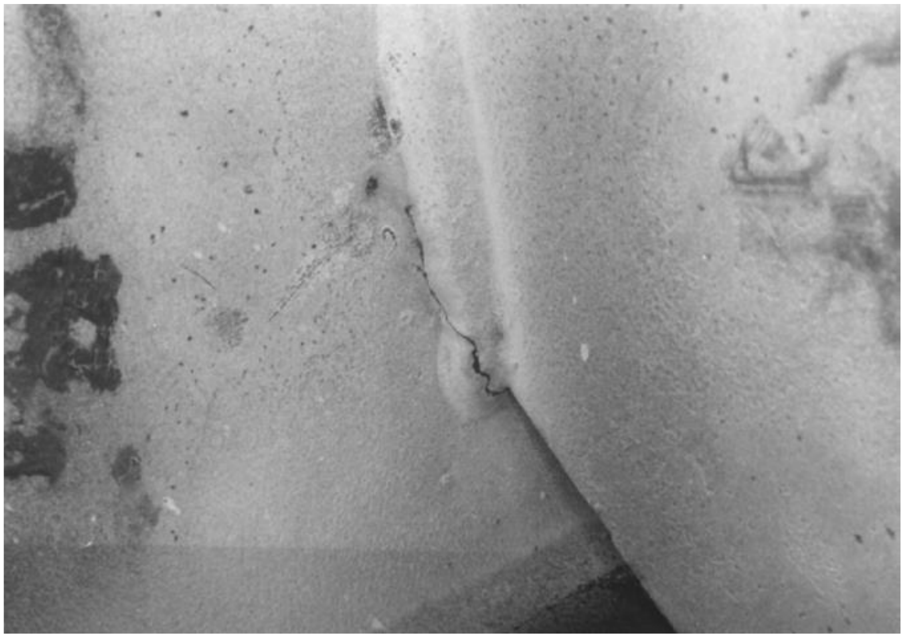


Figure 18 Web Gap Cracking at End of Transverse Connection Plate

2.4.2.5 Coped Beam Connections

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.

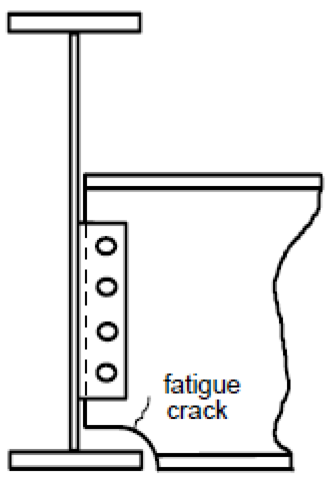


Figure 19 Bottom Flange of Floor Beam Coped at Connection to Girder

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 is assumed to 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 [11], and the preferred solution in the case of new designs is to avoid copes entirely. If copes must be included, execution of the work and inspection must satisfy the applicable requirements of the AASHTO/AWS-D1.5 Specifications [12].

2.4.2.6 Connections for Lateral Bracing

The connection of lateral bracing members to girders should be done directly to the adjacent flange if possible and preferably by bolting. Sometimes, the connection between the lateral bracing system and the girder will be to a horizontal plate bolted or welded to the girder flange or web. In the AASHTO LRFD BDS, this is termed a "lateral connection plate."

When it is not practical to attach the lateral connection plate (or lateral bracing members) directly to the flange, details at the intersection of lateral connection plates and transverse elements welded to the web to avoid conditions susceptible to constraint-induced fracture (Sections 2.3.2) are provided in AASHTO LRFD BDS Table 6.6.1.2.4-2 (Section 3.3.3). Note however that the base metal at the termination of the horizontal weld of the lateral connection plate to the web is classified as a Category E fatigue detail (AASHTO LRFD BDS Table 6.6.1.2.3-1) with a low nominal fatigue resistance, and so bolting of the lateral connection plate to the web may be required if the fatigue resistance is not satisfactory. When welding or bolting a lateral connection plate to the web, the detailing provisions of AASHTO LRFD BDS Article 6.6.1.3.2 also apply to prevent conditions susceptible to distortion-induced fatigue.

2.4.2.7 Summary

Out-of-plane distortion that is concentrated in small web gaps remains a potential source of fatigue cracking in bridge structures if proper detailing practices are not followed (Section 3.2). Distortion-induced fatigue cracking can potentially develop 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 have developed 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 LRFD 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 AASHTO LRFD BDS Equation 6.6.1.2.2-1, is:

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

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 AASHTO LRFD BDS Article 3.6.1.4
$(\Delta F)_n$	=	appropriate nominal fatigue resistance

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

AASHTO LRFD BDS Article 6.6.1.2.1 specifies that the preceding equation need only be checked for details subject to a net applied tensile stress. That is, in regions where the unfactored permanent loads produce compression, fatigue need only be considered at a particular detail if the compressive stress at that detail is less than the maximum tensile live load stress caused by the Fatigue I load combination. The Fatigue I load combination represents the heaviest truck expected to cross the bridge over its 75-year fatigue design life. When determining the permanent compressive stress for these checks, the effect of any future wearing surface may be ignored. If the tensile component of the Fatigue I stress range does not exceed the compressive stress due to the unfactored permanent loads, there is no net tensile stress. As a result, the stress cycle is compression/compression and a fatigue crack will not propagate beyond a heat-affected zone.

Live load almost always produces negative moments in girders in adjacent spans of continuous span bridges and possible in the same span when supports are sharply skewed. Live load also produces positive moment in girders in regions defined as negative moment regions between points of dead load contraflexure and supports. Thus, it is necessary to test each point in a span, and both flanges, to determine if net tension might exist for the fatigue loading.

Under certain conditions, AASHTO LRFD BDS Article 6.6.1.2.1 permits dead load stresses, live load stresses, and live load stress ranges for the fatigue limit state checks due to loads applied to the composite section to be computed using the appropriate corresponding composite section assuming the concrete deck is effective (i.e., uncracked) for both positive and negative flexure. The long-term ($3n$) composite section is to be used for the dead loads, and the short-term (n) composite section is to be used for the live loads. AASHTO LRFD BDS Article 6.10.1.7 specifies those conditions as the following: 1) shear connectors must be provided along the entire length of the girder; and 2) the minimum 1 percent longitudinal deck reinforcement must be placed wherever the tensile stress in the concrete deck due to either the factored construction loads (Article 3.4.2.1) or load combination Service II (Table 3.4.1-1) exceeds the factored lower-bound modulus of rupture of the concrete. The degree of deck cracking is felt to be controlled to such a degree under these conditions that the full stiffness of the deck over the length of the bridge is a reasonable assumption. Where cracks do occur, the stress in the longitudinal reinforcement will increase until the crack is arrested, and the cracked concrete and reinforcement reach equilibrium. With a small number of staggered cracks that do not coalesce at any given section, the concrete can provide significant resistance to tensile stress at service load levels. If one or both of the preceding conditions is not satisfied, then the concrete deck is not to be considered effective for computing stresses on the composite section for negative flexure for fatigue.

Using the short-term n-composite section to compute the factored fatigue load stresses due to both positive and negative flexure results in a significant reduction in the computed stress range at fatigue details at and near the top flange. The stress range at fatigue details at or near the bottom flange is largely unaffected because the increase in stiffness for negative flexure is essentially offset by the increase in the distance from the n-composite neutral axis to the bottom flange.

The lateral bending stress range in the bottom flange due to the factored fatigue live load plus impact must be included in (Δf) for curved bridges, where applicable (AASHTO LRFD BDS Article 6.10.5.1). Bottom flange-to-web welds need only be checked for the average stress range (due to major-axis bending) since the welds are near the mid-width of the flange. However, at points where attachments are welded to the girder bottom flanges, such as at cross-frame connection plates, the flange should be checked for the average stress range plus the lateral flange bending stress range at the critical transverse location on the flange. Bottom flange butt welds should also be checked for the lateral bending stress range in addition to the average stress range. Consideration should also be given to including the bottom-flange lateral bending stress range in straight skewed bridges in regions where cross-frames are discontinuous. The stress range due to flange lateral bending is generally not a consideration for details on the top flange because the flange is continuously braced by the concrete deck. A challenge in these curved and skewed analysis lies in the calculation of the stress ranges. Analysis of bridge sof these types is usually performed by software and the question of whether the vertical and lateral bending moments (stress) are envelope values or concurrent forces is an important one to understand. The use of envelope values is surely conservative but may also be punitive for certain designs or evaluations.

3.1.2 Detail Categories

The AASHTO LRFD BDS defines eight Detail Categories for fatigue: A, B, B′, C, C′, D, E and E′. Figure 20 shows the fatigue-resistance curves given in the AASHTO LRFD BDS. 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. Beyond a certain point, which depends on the Detail Category, the fatigue-resistance line is horizontal. This feature will be discussed subsequently. The design curves represent permissible stress range values that are based on a 98 percent confidence limit, or lower bound of fatigue resistance. Thus, for a particular detail type, most of the fatigue data falls above the design curve and the test data should not deviate significantly from the curve. The slope of all the design curves was determined to be very close to a constant value of -3.0. Thus, a constant slope of -3.0 was imposed on the equations.

Equations are presented in the AASHTO LRFD BDS for the various Detail Categories. The plot of the Detail Categories appears in Article C6.6.1.2.5 of the AASHTO LRFD BDS. 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 (AASHTO LRFD BDS Table 6.6.1.2.3-1), a portion of which is illustrated in Figure 21. The ranking of the Detail Categories is such that Category A has the highest fatigue resistance and Category E′ the lowest. The numerical entries in the table will be discussed subsequently.

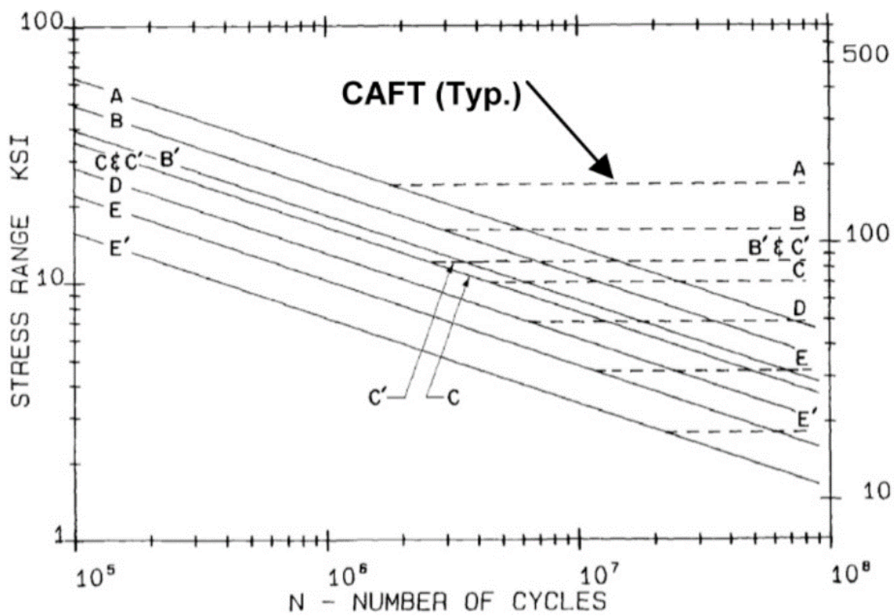


Figure 20 Fatigue Resistance of the AASHTO Detail Categories as Represented in Article C6.6.1.2.5 of the AASHTO LRFD BDS

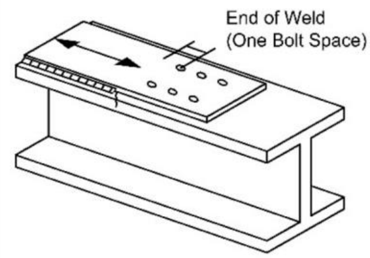
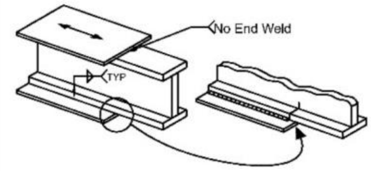
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-Up Members (continued)					
3.6 Base metal at the termination of partial length welded cover plates with slip-critical bolted end connections satisfying the requirements of Article 6.10.12.2.3.	B	120×10^8	16	In the flange at the termination of the longitudinal weld	
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	

Figure 21 Sample Tabularized Detail Category Descriptions as Given in Table 6.6.1.2.3-1 of the AASHTO LRFD BDS

3.1.3 Infinite–Life Design

Satisfaction of the Fatigue I limit state should theoretically provide an infinite fatigue life for the detail in question. Table 1 (AASHTO LRFD BDS Table 6.6.1.2.3-2) shows for each detail category the values of the projected average daily truck traffic in a single lane in one direction, $(ADTT)_{SL}$, above which the infinite life check governs, assuming a 75-year fatigue design life and one stress range cycle per truck (i.e., n = 1.0). Note that these values of ADTT are relatively low (less than 2,000 trucks per day) for most categories. Therefore, except for Detail Categories E and E′, the design will most often be governed by the infinite life check.

Table 1 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	690
B	1120
B′	1350
C	1680
C′	975
D	2450
E	4615
E′	8485

The indicated values in Table 1 were determined by equating the infinite life and finite life resistances (see Sections 3.1.3.2 and 3.1.4.2) with due regard to the difference in load factors used with the Fatigue I and Fatigue II load combinations (see AASHTO LRFD BDS Equation C6.6.1.2.3-1). For other values of n, the values in Table 1 should be modified by dividing by the appropriate value of n taken from Table 2 (AASHTO LRFD BDS Table 6.6.1.2.5-2). For values of the fatigue design life other than 75 years, the values in Table 1 should be modified by multiplying the values by the ratio of 75 divided by the fatigue life sought in years. The determination of the projected $(ADTT)_{SL}$ is discussed further in AASHTO LRFD BDS Article 3.6.1.4.2.

Table 2 Cycles per Truck Passage

Longitudinal Members	
Simple Span Girders	1.0
Continuous Girders: 1) near interior support 2) elsewhere	 1.5 1.0
Cantilever Girders	5.0
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling	5.0
Trusses	1.0
Transverse Members	
Spacing > 20.0 ft	1.0
Spacing ≤ 20.0 ft	2.0

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 1.75 represents the stress range due to the heaviest truck that needs to be considered for fatigue. It is not the absolute heaviest truck. The derivation of the fatigue limit state load factors and the calibration of the fatigue limit state in the AASHTO LRFD BDS is described further in [18].

3.1.3.2 Resistance-Side of the Function

The nominal fatigue resistance for the Fatigue I limit-state consists of the straight, horizontal lines in Figure 20. 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 AASHTO LRFD BDS (Table 6.6.1.2.5-3) and given in Table 3.

Table 3 Constant-Amplitude Fatigue Thresholds

Detail Category	Threshold (ksi)
A	24.0
B	16.0
B'	12.0
C	10.0
C'	12.0
D	7.0
E	4.5
E'	2.6

3.1.4 Finite–Life Design

Except as discussed below for components and details on Fracture-Critical Members, where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is less than or equal to the applicable value specified in Table 1 (AASHTO LRFD BDS Table 6.6.1.2.3-2) for the Detail Category under consideration, the Fatigue II load combination may be used in combination with the nominal fatigue resistance for finite life.

For details on members in tension, or with a tension element, that are classified as Fracture-Critical Members (Section 3.3.2), the Fatigue I load combination must be used in combination with the nominal fatigue resistance for infinite life.

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 AASHTO LRFD BDS is less than one, specifically 0.80. Further, this load factor is not applied to the HL-93 vehicle and lane superposition, but only to the fatigue 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 derivation of the Fatigue II load factor is described further in [18].

3.1.4.2 Resistance-Side of the Function

The sloping portion of the fatigue-resistance curves of Figure 20 represent the fatigue resistance for the Fatigue II limit state for finite-life fatigue design. For finite-life fatigue design, the nominal fatigue resistance is calculated as a function of the number of cycles in AASHTO LRFD BDS Equation 6.6.1.2.5-2 as follows:

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

in which:

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

where:

A = detail category constant given in Table 4 (AASHTO LRFD BDS Table 6.6.1.2.5-1)

n = number of stress range cycles per truck passage given in Table 2 (AASHTO LRFD BDS Table 6.6.1.2.5-2)

$(ADTT)_{SL}$ = single-lane ADTT (AASHTO LRFD BDS Article 3.6.1.4.2)

The detail category constant, A, represents the y-intercept of the sloping portion of the fatigue-resistance curves for each detail category.

The significance of the exponential relationship in this equation is that a small change in stress range produces a significant change in the nominal fatigue resistance.

Table 4 Detail Category Constant, A

Detail Category	Constant, *A* (ksi^3)
A	250.0×10^8
B	120.0×10^8
B'	61.0×10^8
C	44.0×10^8
C'	44.0×10^8
D	22.0×10^8
E	11.0×10^8
E'	3.9×10^8

3.2 Distortion-Induced Fatigue

Based on the previous discussion of 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 quantitative rules relating to distortion-induced fatigue. The AASHTO LRFD BDS provides a separate article on distortion-induced fatigue (Article 6.6.1.3). This article contains a general statement that stresses the importance of proper connection of transverse components to longitudinal (i.e., main) components. Sub-articles then offer specific information relating to transverse connection plates (Article 6.6.1.3.1), lateral connection plates (Article 6.6.1.3.2), and orthotropic decks (Article 6.6.1.3.3).

Article 6.6.1.3.3 simply directs the reader to 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 to control distortion-induced fatigue of deck details subject to local secondary stresses due to out-of-plane bending.

The articles relating to connection plates provide rules for this important topic of girder detailing. Improper detailing of connection plates has been a significant source of fatigue cracking in older steel bridges.

Article 6.6.1.3 alerts the designer to the possibility of fatigue cracking resulting from excessive out-of-plane flexing of a girder web where a rigid load path has not been provided to adequately transmit the force in the transverse member from the web to the flange. To verify that the connection of transverse connection plates to the flanges is not undersized, particularly at locations where larger out-of-plane forces may develop, Article 6.6.1.3.1 recommends that in the absence of better information, the welded or bolted connection in straight, non-skewed bridges should be designed for a minimum of a factored 20-kip lateral force. Engineers will commonly check the connection between the stiffener and flange for this force but there is no guidance given on what portions of the stiffener to girder connection should be included in this strength check. For straight, skewed bridges and horizontally curved bridges with or without skew, this force can be determined by analysis. The attachment of internal cross-frame connection plates to bottom flanges of tub girders is discussed in Article C6.6.1.3.1.

Article 6.6.1.3.1 provides an exception where intermediate connecting diaphragms are used on rolled beams in straight bridges with composite reinforced decks whose supports are normal or skewed not more than 10 degrees from normal, and with the intermediate diaphragms placed in contiguous lines parallel to the supports. Under such conditions, live load forces in the intermediate diaphragms are typically relatively small. In such cases, Article 6.6.1.3.1 permits less than full-depth end angles or connection plates to be bolted or welded to the beam web to connect the diaphragms. This provision reflects the fact that less than full-depth end angles or connection plates have been bolted or welded to the webs of rolled beams to connect intermediate diaphragms for a number of years without noted issues. Rolled beams typically have thicker webs resulting in larger resistance to out-of-plane distortion and larger lateral-torsional buckling resistance. The end angles or plates must be at least two-thirds the depth of the web to provide some additional torsional resistance to the beam. For bolted angles, a minimum gap of 3.0 inches must be provided between the top and bottom bolt holes and each flange to preclude potential problems with distortion-induced fatigue. All bolt spacing requirements specified in Article 6.13.2.6 are to be satisfied. For welded angles or plates, a minimum gap of 3.0 inches is to be provided between the top and bottom of the end-angle or plate welds and each flange, and the heel and toe of the end angles or both sides of the connection plate, as applicable, is to be welded to the beam web. Welds are not to be placed along the top and bottom of the end angles or connection plates.

Article 6.6.1.3.2 specifies minimum distance requirements from the flange for lateral connection plates in cases where it is not practical to attach the lateral connection plate (or the lateral bracing members) directly to the flange to reduce the concentration of out-of-plane distortion in the web between the lateral connection plate and the flange to a tolerable magnitude. The specified minimum distance also provides adequate electrode access and moves the connection plate closer to the neutral axis of the girder to reduce the impact of the weld termination on fatigue strength. Lateral connection plates are to be centered on transverse stiffeners, whether or not the plate is on the same side of the web as the stiffener, and the detailing of welded lateral connection plates is also to satisfy the provisions of Article 6.6.1.2.4 to avoid conditions susceptible to constraint-induced fracture at the intersection of lateral connection plates and transverse/bearing stiffeners or connection plates welded to the web (see Table 6.6.1.2.4-2 and Section 3.3.3). Article C6.6.1.3.2 provides specific recommendations for such details when there is interference from a cross-frame or diaphragm attached to a bearing stiffener, or from a transverse floor beam or internal plate diaphragm.

3.3 Fracture Control

3.3.1 General

Fracture of steel bridge members is typically precluded through material selection by specifying adequate minimum levels of fracture toughness for the various grades of bridge steels based upon minimum expected temperatures and material thickness. As discussed previously, the Charpy V-Notch (CVN) impact test is used to determine the fracture toughness for various bridge steels. Also, a Fracture Control Plan, per AASHTO/AWS-D1.5 Specifications (Clause 12) [12], can provide a higher level of safety for fracture-critical members (Section 3.3.2). 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.

Three temperature zones specified in the AASHTO LRFD BDS (Table 6.6.2.1-2) are based upon minimum service temperature. Higher minimum 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 minimum CVN impact energy requirements are specified in the ASTM A709/A709M specification for bridge steels and are tabulated in LRFD Table C6.6.2.1-1.

Unless otherwise indicated on the contract plans, CVN testing is required for primary members or components that are subject to a net tensile stress, or for portions thereof located in designated tension zones, under Strength Load Combination I, except for diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally curved bridges (AASHTO LRFD BDS Article 6.6.2.1). Although cross-frames and diaphragms and their components in curved bridges are classified as primary members, in many years of practice, no specific concerns related to the fatigue and fracture performance of these members have been reported. Specifying that CVN testing be performed for members, components, or portions thereof for which such testing is not required, e.g., secondary members, adds additional complexity and cost without providing any significant additional value. Designation of entire members or components as subject to a net tensile stress, rather than merely their tension zones where applicable, also adds unnecessary cost and is inconsistent with the objectives of the fracture control plan.

AASHTO LRFD BDS Article 6.6.2.1 also requires that primary members or components, or portions thereof, subject to a net tensile stress under Strength Load Combination I be designated on the contract plans. Member or component designations (i.e., primary vs. secondary) are provided in Table 6.6.2.1-1. Proper designation of members or components is essential because this information affects various aspects of fabrication such as material purchasing, welding, and inspection. Designation of secondary members or components, as listed in Table 6.6.2.1-1, as primary members or components should be scrutinized as this will invoke more costly and complex fabrication and testing requirements that do not add significant value and are not necessary. Secondary members or components that are specifically designated as primary members or components by the Owner should be indicated as such on the contract plans, and whether those members or components, or portions thereof, are subject to a net tensile stress under Strength Load Combination I.

3.3.2 Fracture-Critical Members

The AASHTO LRFD BDS defines a FCM as a steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse. The designer is responsible for determining which, if any, member or component is without load-path redundancy and should therefore be classified as a fracture-critical member (FCM). The location of all FCMs is to be clearly indicated on the contract plans. Primary steel members or elements that are not subject to a net tensile stress under Strength Load Combination I are not to be classified as a FCM. Secondary members, and diaphragm or cross-frame members in horizontally curved bridges, should also not be designated as FCMs. FCMs are to be fabricated in accordance with Clause 12 of the AASHTO/AWS-D1.5 Specifications [12] and are to have routine inspections performed, along with hands-on in-service inspections as described in 23 CFR 650.

A System Redundant Member, or SRM, is defined as a primary steel member in tension, or with a tension element, that is without load path redundancy but has redundancy in the bridge system, such that fracture of the cross-section at one location of the member will not cause the entire bridge, or a portion thereof, to collapse. An SRM classification is to be supported through Owner-approved calculation, analysis, or other criteria that are supported by experimental verification. SRMs are to be identified on the contract plans. An acceptable detailed finite element analysis and evaluation procedure for classification of SRMs is provided in the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* [19]. An alternative simplified approach for classifying SRMs in continuous composite twin tub-girder bridges is discussed in [20]. SRMs are to be fabricated in accordance with Clause 12 of the AASHTO/AWS-D1.5 Specifications and are to have routine inspections performed but need not be subject to the hands-on in-service inspection requirements described in 23 CFR 650 according to an FHWA Memorandum dated June 20, 2012 [21]. SRMs should satisfy the Charpy V-notch impact energy requirements specified for FCMs and fatigue details on SRMs should also be designed for infinite life.

3.3.3 Constraint-Induced Fracture

Constraint-induced fracture was described previously in Section 2.3.2. Conditions susceptible to constraint-induced fracture can be avoided by following good detailing practices.

Attached welded elements parallel to the primary stress sometimes intersect a full-depth transverse/bearing stiffener or connection plate. In regions subject to a net tensile stress under

Strength Load Combination I, these elements are less susceptible to fracture and fatigue if the attachment parallel to the primary stress is detailed to be continuous while the transverse attachment is detailed to be discontinuous. AASHTO LRFD BDS Table 6.6.1.2.4-1 provides a detail for the intersection of a longitudinal stiffener and a transverse element welded to the web. The longitudinal stiffener, which is parallel to the primary stress, is less susceptible to fatigue and fracture if it is continuous and the transverse web element is discontinuous and attached to the longitudinal stiffener with fillet welds. This avoids the creation of a crack-like plane of discontinuity perpendicular to the primary flow of tensile stress. The fillet welds must meet minimum size requirements; groove welds are not necessary and should not be used to attach the stiffeners.

Approximately a ¾ inch minimum gap between the weld toes is recommended in all cases where a gap is specified, but not less than ½ inch; larger gaps are also acceptable. If a gap is not specified, since the continuous longitudinal stiffener is typically welded to the web before the discontinuous transverse web element, the cope or snipe in the transverse web element should be reduced so that it just clears the longitudinal weld. The welds may either be stopped short of the free edges or wrapped around the ends of the transverse web element for sealing.

The longitudinal stiffener may be discontinuous at the intersection, but only if the intersection is subject to a net compressive stress under Strength I, and the longitudinal stiffener is attached to the continuous transverse web element, again with fillet welds as shown in Table 6.6.1.2.4-1. Such a detail is recommended at intersections with bearing stiffeners. Longitudinal stiffeners should only be interrupted and left unattached to transverse web elements if it is desired to terminate the longitudinal stiffener at that location, and either the location is subject to a net compressive stress, or the nominal fatigue resistance of the longitudinal stiffener end detail is shown to be satisfactory in regions subject to net tension.

Table 6.6.1.2.4-2 provides similar recommended details to avoid conditions susceptible to constraint-induced fracture at the intersections of lateral connection plates and transverse/bearing stiffeners or connection plates welded to the web when it is not practical to attach a lateral connection plate (or the lateral bracing members) directly to a flange and when the lateral connection plate must be placed on the same side of the web as the transverse web element.

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. FHWA. *Design and Evaluation of Steel Bridges for Fatigue and Fracture.* Federal Highway Administration, U.S. Department of Transportation, Washington, DC, December 2016.

3. AASHTO. *LRFD Bridge Design Specifications*, Ninth Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2020.

4. AASHTO. *The Manual for Bridge Evaluation*, Third Edition with 2019 and 2020 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, DC, 2018.

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

6. Munse, W. H. and Grover, L. M., *Fatigue of Welded Steel Structures*. Welding Research Council, New York, 1964.

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

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

9. 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, DC, 1970.

10. 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, DC, 1974.

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

12. AASHTO/AWS. *Bridge Welding Code*, BWC-8 (AASHTO/AWS D1.5M/D1.5:2020). American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC, 2020.

13. Connor, R. J. and Lloyd, J. B. *Maintenance Actions to Address Fatigue Cracking in Steel Bridge Structures, Proposed Guidelines and Commentary,* Purdue University, West Lafayette, IN, March 2017.

14. Wright, W. J., Fisher, J. W., and Kaufmann, E. J. “Failure Analysis of the Hoan Bridge Fractures,” *Recent Developments in Bridge Engineering*, Mahmoud ed. Swets & Zeitlinger, Lisse, 2003

15. Kaufmann, E. J., Connor, R.J., and Fisher, J. W. *Failure Investigation of the SR 422 over the Schuylkill River Girder Fracture-Draft Final Report*. ATLSS Engineering Research Center, Lehigh University, October, Bethlehem, PA, 2004.

16. Mahmoud, H. N., Connor, R. J., and Fisher, J. W. “Finite Element Investigation of the Fracture Potential of Highly Constrained Details,” *Journal of Computer-aided Civil and Infrastructure Engineering*. Blackwell Publishing, Inc., Malden, MA, Vol. 20, 2005.

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

18. Kulicki, J. M., Wassef, W. G., Mertz, D. R., Nowak, A. S., and Samtani, N. C. *Bridges for Service Life Beyond 100 Years: Service Limit State Design*, SHRP2, Transportation Research Board of the National Academies, Washington, DC, 2015.

19. AASHTO. *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*, 1st Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2018.

20. Connor, R. J., Korkmaz, C., Campbell, L. E., Bonachera Martin, F. J., and Lloyd, J. B. “A Simplified Approach to Design Composite Continuous Twin Tub-Girder Bridges as Redundant Structures”, Purdue University, West Lafayette, IN, 2020.

21. FHWA. “Clarification of Requirements for Fracture Critical Members”, Memorandum from M. M. Lwin, P.E., S.E., Federal Highway Administration, Washington, DC, June 20, 2012.

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