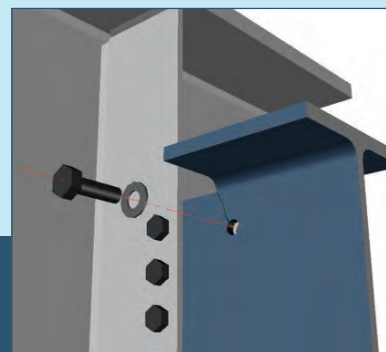


MAINTENANCE GUIDELINES FOR STEEL BRIDGES

Addressing Fatigue Cracking and Details
at Risk of Constraint-Induced Fracture

G14.1-2021



AMERICAN ASSOCIATION
OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS
AASHTO



AASHTO/NSBA STEEL BRIDGE COLLABORATION

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Executive Summary

Nearly thirty percent of the U.S. national bridge inventory (highway bridges) is made up of bridges having steel superstructures with an average age of about 48 years at the time of this writing. Fortunately, steel bridge superstructures lend themselves well to repair and retrofit methods that can improve and even eliminate fatigue and constraint-induced fracture (CIF) concerns, extending the service life for many years or even decades. Unfortunately, fatigue and fracture tend to be the least understood by engineers of the limit states affecting steel bridges. In some cases, this has led to repair or retrofit strategies that rendered a worse condition than existed prior to the attempted fix.

This document provides simple-to-follow guidelines for the maintenance actions to address fatigue cracking as well as details at risk of constraint-induced fracture (CIF) in steel bridges. It is a synthesis of best practices from published literature, project reports, and past and ongoing research projects, as well as input from industry professionals. Intended to be a very practical reference text, it is written with everyone in mind, from a maintenance contractor to an asset manager or design engineer, providing detailed descriptions of the driving causes of fatigue cracking and CIF in steel bridges and accepted methods for repair or retrofit. A number of case studies are discussed, giving context for the different detail susceptibilities and utilizing a mixture of real-world and rendered images to illustrate the problems and solutions. For each case, a suggested sequence of steps is also provided as a “how-to.”

Appendix A contains quick reference tables with Harvey Ball ideograms that help users qualitatively identify appropriate repair or retrofit approaches for a type of detail. Some steel bridge details have multiple strategies that can be implemented and the tables are intended to give the reader a snapshot view of the benefits of each one, the degree of success it has had historically, and the level of ease (which translates to cost) with which the strategies can be implemented. Chapter 1 explains the ideogram tables in more detail.

Chapter 2 reviews several important general topics such as a brief history of steel bridge issues, the basics of fracture mechanics, and the basics of fatigue and fatigue evaluation. In addition, Chapter 2 also includes a section on the urgency of repairs, helping to ensure the reader considers factors that contribute to how urgently a repair or retrofit should be treated. Chapter 3 delves into some fundamental repair and retrofit techniques, such as grinding and hole drilling. These techniques are referred to many times throughout the rest of the guidelines. Chapter 4 introduces details commonly susceptible to load-induced fatigue, giving repair and retrofit strategies specific to those details. Load-induced fatigue is that caused by primary stresses and includes many of the details contained in the fatigue detail tables in the *AASHTO LRFD Bridge Design Specifications* (referred to as LRFD Design hereafter) (AASHTO, 2017). Chapter 5 transitions into distortion-induced fatigue caused by secondary stresses in steel structures. These are the most common types of fatigue cracks found in the steel bridge inventory. Chapter 6 covers the mechanics of constraint-induced fracture and details a number of effective retrofits that can be implemented to reduce or eliminate risk of fracture. And finally, Chapter 7 discusses several retrofit or repair concepts that may be in use, or being considered for use, that need additional research and development before being recommended.

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CHAPTER 1

INTRODUCTION

The following is a synthesis of best practices from published literature, ongoing research activities, and input from industry professionals with the objective to provide guidelines for maintenance actions to address fatigue cracking in steel bridges. These Guidelines cover repair procedures, detailing techniques, maintenance recommendations, inspection recommendations, and preservation actions to repair and retrofit steel bridges in order to mitigate initiation of fatigue cracks on details known to have low fatigue resistance, control further growth of existing fatigue cracks, and reduce or eliminate the risk of CIF in steel bridges. The findings are primarily intended for highway bridges but are conceptually equally applicable to railroad bridges. In addition to fatigue, preemptive maintenance actions related to constraint-induced fracture (CIF) are also presented.

Fatigue and fracture tend to be the two limit states of steel bridges least understood when it comes to design, inspection, and especially for repair and retrofit. Although a wealth of research and case studies of fatigue damage and other failures related to steel bridge cracking exist in the literature, few university civil engineering programs offer courses on these topics. Only a limited number of professional short courses are offered which specialize in these topics, and even fewer reference manuals are available to practitioners. As a result, bridge owners and their consultants are often left to develop their own strategies. Unfortunately, experience has shown that some implemented repairs or retrofits have actually made the conditions worse due to the lack of understanding of what drives the development of fatigue cracks and how best to address it.

Detailed discussion is included regarding the cause or driving force behind various fatigue cracks observed in the field. As expected, the mitigation approaches have widely varied throughout the inventory, with some being more effective than others. During the literature review, which included a survey and conversations with industry leaders, the most effective retrofit strategies were identified for a given type of cracking. While the reader is encouraged to study these Guidelines to become fully familiar with the associated recommended retrofit strategy, ideograms were developed to assist the user in quickly selecting the most effective retrofit(s) for a given type of cracking. The approach follows that which is often used in publications that are used for rating and comparing cars or appliances. The concept is illustrated and explained in Table 1-1.

It is noted that there are often several “acceptable” retrofit strategies for a given type of cracking. Thus, assuming multiple approaches are known to be effective, other criteria, such as the required skills of the workforce or ease of installation, which generally translate into cost, should also be considered. Hence, the ideogram shown in Table 1-2 was prepared to provide a simple summary of these other factors which should be considered with each approach. While these tables explain the meaning of the ideograms, Appendix A contains the quick reference tables that help users qualitatively identify appropriate repair or retrofit approaches for a type of detail. Some steel bridge details have multiple strategies that can be implemented, and the tables are intended to provide a snapshot view of the benefits of each one, the degree of success it has had historically, and the level of ease with which the strategies can be implemented.

While the quick reference tables provided in Appendix A should not be used without reviewing the content of these Guidelines, they do provide a useful quick reference guide. Further, it is noted that some retrofits are inherently more challenging to install than others. For example, simple grinding is useful in removing a shallow nick or gouge, while retrofit and repair of a detail susceptible to CIF will require considerably more effort and possibly engineering analysis.

Table 1-1. Description of Ideograms Used in Repair and Retrofit Tables for Success of Performance

Success of Repair	
●	Well-documented successful performance in the laboratory and in the field. Significant increase in fatigue resistance or significant reduction of risk of fracture.
◐	Documented successful performance in the laboratory or in the field showing moderate fatigue resistance enhancement or reduction of risk of fracture.
○	Unknown or unproven long-term success or documented poor performance

Table 1-2. Description of Ideograms Used in Repair and Retrofit Tables for Ease of Installation

Ease of Installation	
	Relatively easy to install with common hand tools (e.g., grinder, mag-drill) and minimal experience with iron work required.
	Decreased ease of installation, but still manageable with most common hand tools and beginner skill level in iron work.
	Some ease, requiring average working knowledge of repairing steel and/or specialized tools or training (e.g., ultrasonic impact machine, turn-of-nut wrench).
	Moderate effort required. Specialized training and tools required. Sound engineering judgement needed.
	Significant effort required. Difficult to install, generally requiring expert knowledge. May also require engineering analysis.

CHAPTER 2

FATIGUE AND FRACTURE FUNDAMENTALS

2.1—HISTORICAL STEEL BRIDGE ISSUES

Early steel bridges were fabricated using mechanical fasteners. The first fastener, of course, was the rivet, which was later followed by the high-strength bolt in the 1950s. This type of fabrication resulted in internally redundant members that have been shown to be quite reliable in service. This, combined with smaller loads and lesser loading frequencies by today's standards, meant that these early structures less commonly experienced fatigue cracking. Welding became more popular in the 1950s, where it quickly rose to the most common method for steel bridge fabrication. This had two principal effects relative to fatigue, (1) welding introduced high residual stresses and more severe and common initial crack-like defects than did riveting or bolting, and (2) the continuity between elements of welded construction meant that a crack could propagate across element boundaries (Fisher, Kulak, & Smith, 1997).

Prior to 1965, the AASHTO (now known as AASHTO) *Standard Specifications for Highway Bridges* contained no guidance for fatigue design. For the following decade, only limited provisions were included, until in 1974, the modern fatigue design provisions that use the nominal stress approach developed by John Fisher were incorporated (Connor et al., 2005). The current *AASHTO LRFD Bridge Design Specifications* utilize essentially the same fatigue design approach established in 1974, along with some improvements over the years (AASHTO, 2017).

One of the most effective improvements to the design specifications was that which addressed distortion-induced fatigue. Although the provisions introduced in 1974 effectively corrected fatigue issues resulting from primary stresses, they were silent regarding the potential for cracking caused by secondary stress ranges. Secondary stress ranges are those unanticipated stress ranges that are not accounted for in the design process but occur in bridges due to the interconnectivity of structural elements into a structural system. The most common is out-of-plane distortion cracking observed within a web gap where cross-frames or floorbeams connect to the primary structural girders at transverse connection plates. In 1985, provisions were added to the *AASHTO LRFD Bridge Design Specifications* requiring improved detailing techniques to avoid this phenomenon. This was a significant advancement for the industry because of the proliferation of this type of cracking, which was estimated to be about 90 percent of all fatigue cracks (Connor & Fisher, 2006).

The 1970s also brought the topic of redundancy to the forefront. This was primarily triggered by the collapse of the Silver Bridge near Point Pleasant, West Virginia, in 1967. Years following the Silver Bridge collapse, a few other fractures in non-load-path-redundant bridges, not resulting in collapse, also heightened the interest of industry leaders, resulting in more stringent requirements for bridges containing nonredundant “fracture critical” members (FCM). The 1978 AASHTO Fracture Control Plan (FCP) included reduced allowable fatigue stress ranges, more rigorous Charpy V-Notch (CVN) testing requirements, more thorough shop inspection, and the federal requirement for arms-length visual inspection of FCMs every 24 months (AASHTO, 1978; Fish et al., 2015).

Prior to 1970, there were no minimum toughness requirements for bridge steels, leaving the fracture toughness of bridge steel basically unknown. It was not until the 1974 AASHTO *Standard Specifications for Highway Bridges* (later refined in 1978) that bridge steels were required to meet certain minimum CVN values in the *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members* (AASHTO, 1978). Interestingly, as the steel bridge industry progressed over the following years, higher strength steels such as A514 “T-1” steel became more common, which were designed for higher service stresses. However, unfortunately the higher strength steels often had lower fracture toughness. Higher service stresses combined with lower fracture toughness meant that smaller flaws and cracks were more likely to lead to brittle fracture. Further, welding of such steels was also rather challenging. It was recognized that the critical flaw sizes for these structural members could be very difficult to locate during an in-service inspection, even an arms-length inspection. The AASHTO Fracture Control Plan attempted to address this, as well as reduce risk of fracture in non-redundant steel members. These many industry improvements over the years have combined to forge a generation of steel bridges that are perceived to have a lower risk of fracture, as compared to those bridges built prior to the 1978 FCP criteria (Fish et al., 2015).

Current fatigue provisions, the FCP, tougher steels, better-developed inspection programs, and improved welding processes have made the fatigue and fracture limit states as well-controlled as any other design limit state. This

is evidenced by the fact that brittle fractures and fatigue cracking in bridges built after 1985 are extremely rare. In addition, ongoing research in areas such as fracture mechanics, damage tolerant design (DTD) in built-up members, and fracture toughness will continue to aid the steel bridge industry moving forward to take advantage of superior steels like high-performance steel (HPS) and improved design and construction methods. Nevertheless, due to funding constraints, highway and railway bridge owners must maintain their existing inventory of steel bridges. Many of these bridges have exceeded their design fatigue life, were never even designed for the fatigue limit state, or have CIF-prone details that will need to be addressed by engineers who understand proper and effective retrofitting or repair techniques. Finally, research efforts focused on developing an integrated fracture control plan that recognizes the link between superior material properties, fabrication standards, and reliability-based inspection intervals, for example, continues to advance with the objective of reliably and economically managing new and aging bridges.

2.2—FUNDAMENTALS OF FRACTURE

Fracture refers to an unstable and rapid extension of a crack causing a member to partially break or completely break into two parts. Fracture is only possible in the presence of tensile stress and occurs when a crack becomes unstable in a material under certain conditions, such as constraint, temperature, and stress. Fracture can be a ductile failure mode that requires slow and extensive expenditure of energy but can also happen suddenly and without any warning. The latter is referred to as “brittle” fracture and can be a dangerous mode of failure due to the lack of warning. The fracture resistance of a material is called its fracture “toughness.” Fracture toughness is a material property and is influenced by temperature, load or strain rate, and constraint (related to material thickness or equivalent material thickness). The maximum crack size that can be resisted by a structural member within those conditions is referred to as the critical crack size and can be estimated using fracture mechanics.

Fractures are generally grouped into three types: brittle, transitional, or ductile fracture.

1. Brittle Fracture: Instantaneous crack growth, characterized by little or no ductile “necking” of the material, resulting in little dissipation of energy per unit crack growth. The fracture surface will appear straight, or flat, as if cut by saw, and rough to the touch, with little or no disturbance to the coating. A video showing brittle fracture in a laboratory setting can be viewed here: <http://dx.doi.org/10.4231/R78K7710>.
2. Transitional Fracture: Generally has aspects of both brittle and ductile fractures.
3. Ductile Fracture: Occurs by formation, growth, and coalescence of voids and microcracks and results in a slow, stable crack growth. This fracture mode is relatively insensitive to temperature and loading rate (Zhu & Joyce, 2012) and results in large dissipation of energy per unit crack extension. The fracture is characterized by a very jagged surface and extensive plastic deformation, particularly at the edges of the plate where “necking” usually can be observed. Necking will precede this type of fracture, providing some initial warning prior to failure, but on very large steel bridge members it will most likely not be noticed. Some fractures will propagate in a brittle mode and then, once enough energy is dissipated, may begin to slow and transition from brittle to ductile fractures, finally arresting. This will often be distinguished by yield marks and flaking of the coating at the crack termination.

2.2.1—Introduction to Fracture Mechanics

Fracture mechanics is the field of study related to the response of solids in the presence of a defect or crack. Originally developed to explain the rupture of glass specimens, its transition into the field of structural engineering started in the 1940s to help derive the cause of catastrophic failure of welded ship hulls (Fisher, Kulak, & Smith, 1997). The method uses analytical solid mechanics to predict the driving force of crack growth as well as the resistance to crack growth by a material. This “growth” can be due to stable fatigue crack extensions or sudden brittle fracture.

Two primary approaches to fracture mechanics are linear elastic fracture mechanics (LEFM) and elastic-plastic fracture mechanics (EPFM). For ductile fracture, plastic deformations surround the crack tip and the material fracture resistance increases as the crack grows. This mode of fracture is best estimated using the J-integral or Crack Tip Opening Displacement (CTOD), which estimates the energy release rate. The linear elastic fracture mechanics (LEFM) method is most often used in fitness-for-service (FFS) evaluations of steel bridges because it conservatively simplifies the analysis for most practical applications while providing sufficient accuracy; however, it is only valid for the

brittle fracture mode. Instances of brittle fracture have elastic deformation that dominates the crack tip and initiation toughness is governed by the material's fracture resistance. The stress intensity factor, K , was proposed by Irwin in 1957 to describe the intensity of the stress state immediately ahead of a sharp crack tip as part of LEFM (Zhu & Joyce, 2012). K has the unusual units of stress $\times$ length^{1/2} (e.g., $\text{Mpa} \times \sqrt{\text{m}} = 0.9102 \text{ ksi} \times \sqrt{\text{in}}$). This should not be confused with the more familiar stress concentration factor, K_t , which is a dimensionless term that describes the local increase in stress due to changes in geometry (e.g., at a hole, K_t = local stress/nominal stress). LEFM applications are limited to nominally elastic behavior, but small amounts of crack tip plasticity are allowed (Grandt, 2004). The stress intensity factor is the LEFM parameter that relates remote stress, crack size, and structural geometry. In its general form, the stress intensity factor is defined as:

$$K = \sigma \sqrt{\pi a} \times \beta \quad (1)$$

where σ is the remotely applied stress (nominal stress), a is the crack length, and β is a dimensionless factor that depends on crack length and component geometry. Stress intensity factors are important because they provide a means for estimating when fracture will take place, which occurs when the stress intensity factor equals or exceeds the critical stress intensity factor, or fracture toughness, of a material, K_{IC} . Several factors influence the stress intensity factor and fracture toughness. These are discussed in greater detail in the following sections.

2.2.2—Constraint

Crack tip constraint caused by thickness, size, and configuration can have a large effect on fracture toughness. In general, high constraint results in higher crack tip stresses with reduced crack tip yielding, which promotes a more brittle mode of fracture. The overall effect is a lower ductile fracture resistance that pushes the mode of failure toward brittle fracture. Low constraint, on the other hand, has the opposite effect, resulting in lower crack tip stresses with more crack tip yielding and tends to reduce the possibility of brittle fracture (Zhu & Joyce, 2012). Thicker members are more constrained, behaving in a plain strain mode where deformation or yielding (Poisson effect) is limited or even prevented. This behavior can allow local stresses to increase beyond the yield strength, which decreases resistance to brittle fracture.

Brittle fracture in welded joints that are highly constrained and where tensile residual stresses are high is known as constraint-induced fracture (CIF). Such details can “lose” apparent fracture toughness and result in brittle fracture under service loads where ductile behavior would otherwise be expected. Highly constrained welded details should be avoided in new design and retrofit in existing structures in order to minimize the risk of CIF. Several techniques and effective retrofits can be utilized in these cases, such as drilled copes to eliminate intersecting welds, fracture isolation holes, minimizing weld sizes as appropriate, using bolted connections, or reconfiguring the detail. These strategies are discussed in greater detail in Chapter 6.

2.2.3—Strain Rate

The plain strain fracture toughness of some materials is affected by the strain rate, or loading rate, where a decreased fracture toughness will be observed as the strain rate increases (Zhu & Joyce, 2012). For typical bridges, the loading rate from traffic in primary elements is relatively low (0.2 to 1.0 strain cycles per sec). As a result, these structures are generally considered quasi-statically loaded. However, high strain rate events can occur on bridges, such as impact loading from an over-height vehicle. The effect of loading rate is usually considered as an adjustment to the minimum specified fracture resistance of a material instead of explicitly considering the loading rate in an analysis (Fish et al., 2015).

2.2.4—Temperature

Steel can exhibit stable, ductile fracture response at elevated temperatures, but it transitions to unstable, brittle fracture at lower temperatures. This direct relationship between toughness and temperature means that as the temperature decreases, so does the toughness. Table 6.6.2.1-2 in LRFD Design addresses this relationship in new design by dividing the United States into three zones and using the minimum service temperature expected for the area in which the bridge will be built (AASHTO, 2017).

2.2.5—Stress

Applied stress does not affect fracture toughness of the material, but as seen in Equation 1, the stress intensity factor is a direct function of stress—the higher the stress, the higher the potential for fracture. Typically, the stress referred to in terms of a fracture analysis refers to the nominal, maximum stress (measured or calculated) in a member, disregarding local stress concentrations. This is referred to as the far-field, or remote, stress. Stress can be controlled through member sizing, detailing, load distribution, or load restriction. It should be kept in mind that risk of fracture (a brittle limit) or net section yielding (a ductile limit) increases as a fatigue crack grows through a member, effectively increasing the stress by decreasing the cross-sectional area. This effect, however, is not directly calculated during fracture analysis because it is accounted for in the stress intensity factor for a given crack.

2.2.6—Discontinuity

A discontinuity refers to any interruption to uniform stress flow through a given detail. These can come in the form of drilled or punched holes, copes, thickness transitions, or attachment terminations, such as is illustrated in the red area of Figure 2-1. However, they can also be introduced into a member through material imperfections and manufacturing processes, such as weld inclusions. Abrupt changes in the stress flow result in stress concentrations, which increase the risk for fracture. Discontinuities can be controlled through appropriate material specification, fabrication processes that utilize extensive quality control measures, and detailing practices that reduce concentrations and promote uninterrupted stress flow. These concepts do not only apply to new design but also the design and installation of effective retrofits and repairs. In some cases, discontinuities have been introduced into bridges during fatigue or fracture retrofit and repair projects, not only by the design of the repair or retrofit itself but also by ignorance or negligence.

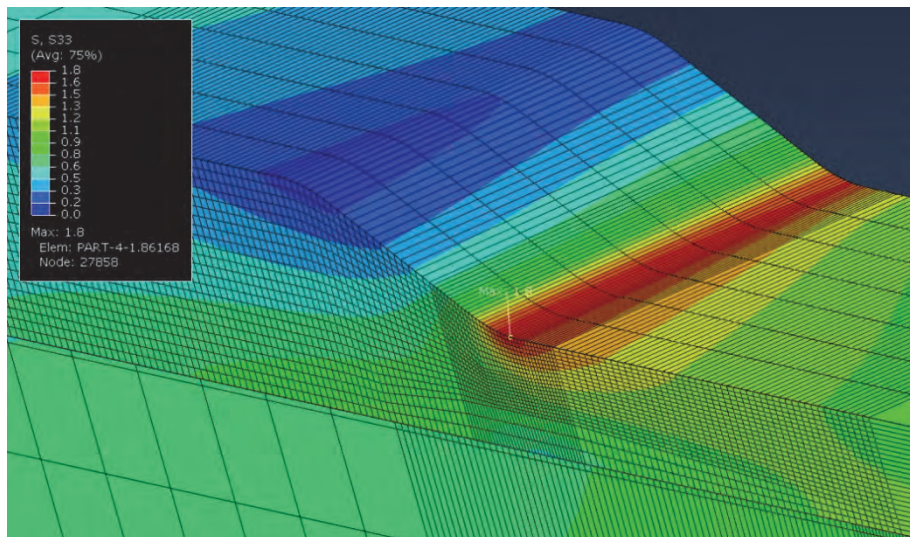


Figure 2-1. Stress Concentration at a Finite Element Model Flange Thickness Transition

2.2.7—Material Toughness

Although some material properties, such as yield strength, ductility, and corrosion damage, have some effect on the fracture resistance of a member, by far the greatest effect is due to the fracture toughness. That said, the toughest materials are usually very ductile.

In simplest terms, crack propagation is a release of stored strain energy. If the release of the energy exceeds the ability of a material to absorb energy, then the crack will continue to grow. Toughness is the material property that quantifies the ability of a material to absorb energy or resist fracture. The amount of energy that can be absorbed relates to the local plastic deformation at the crack tip prior to fracture initiation and throughout fracture propagation. This can be graphically represented by the area under the stress–strain curve. Figure 2-2 qualitatively illustrates

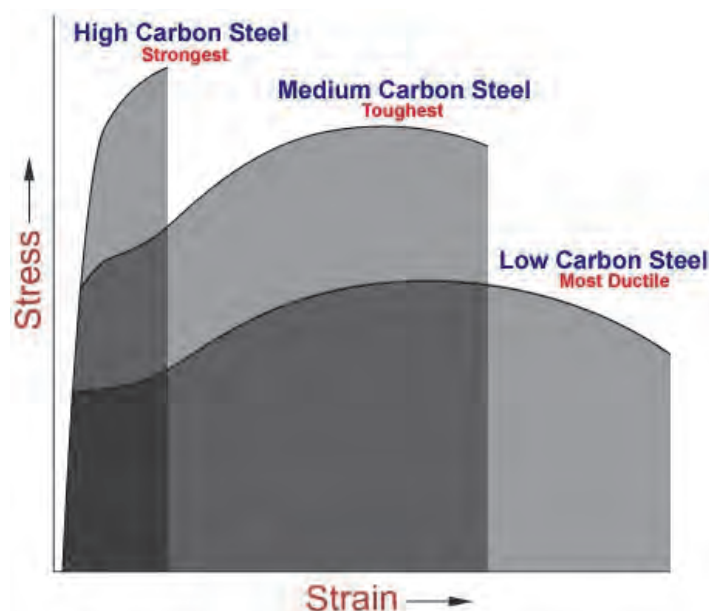


Figure 2-2. Material Toughness Curves (NDT Resource Center, 2014)

this concept with three different types of steel. The first, high-carbon steel, possesses high strength and low ductility, and the lowest toughness. The low-carbon steel possesses a lesser strength, higher ductility, and higher toughness than the high-carbon steel. The final example is the medium-carbon steel, which falls between the previous types, possessing a compromise of ductility and strength while having the highest level of toughness. In this example, the high-carbon steel would be expected to behave in a brittle manner. The medium-carbon steel better represents a typical high-performance steel (HPS) where the optimal combination of strength and toughness is retained in the same material providing the best possible structural performance for steel bridge applications.

The most widely used method of estimating the toughness of steel is the Charpy V-notch (CVN) impact test. This procedure was developed over 90 years ago, but remains a primary method of estimation (Fisher, Kulak, & Smith, 1997) due to feasibility and economics.

CVN testing is a standardized, high-strain-rate impact test used to determine how much energy a small bar containing a machined notch can absorb before fracture occurs. This test is not a direct measurement of fracture toughness but rather qualitatively infers fracture toughness. By using some type of accepted relationship or correlation between measured energy absorption and toughness, the fracture toughness can be estimated. Guidance for the test is provided in ASTM E23, where requirements can be found for the test specimen dimension and orientation, and impact test machine maintenance and calibration, as well as how to perform the test itself.

During the test, a Charpy specimen is cooled to the desired temperature and maintained at that temperature for at least 5 minutes. Tongs, which are also cooled and maintained at the test temperature, are used to extract the specimen from the cooling bath and place it in the test machine within a specified time limit so as to minimize any warming of the specimen. Once the specimen is placed, a lever is pulled on the test machine and the hammer swings from a high position impacting the specimen opposite the notch, as shown in Figure 2-3. The energy absorbed by the specimen is measured by knowing the hammer weight, the height of the hammer in the start position (h_1), and the height at the end of the swing (h_2). The difference between these two potential energies is the energy absorbed by the specimen, which is indicated on the calibrated scale. Videos showing CVN impact testing can be seen here: <http://dx.doi.org/10.4231/R7H41PC8> and <http://dx.doi.org/10.4231/R74T6G98>.

The effect of temperature on the energy absorbed is often examined by repeating the test using identical specimens that have been cooled to various temperatures. A variety of tests at varying temperatures can establish the relationship between the measured absorbed energy and temperature. Figure 2-4 shows a sample of actual CVN test data from steel from a circa 1900 railroad truss. The general regions of the transition curve have been highlighted for emphasis. This figure illustrates that as the temperature is increased, the steel undergoes a brittle-to-ductile transition

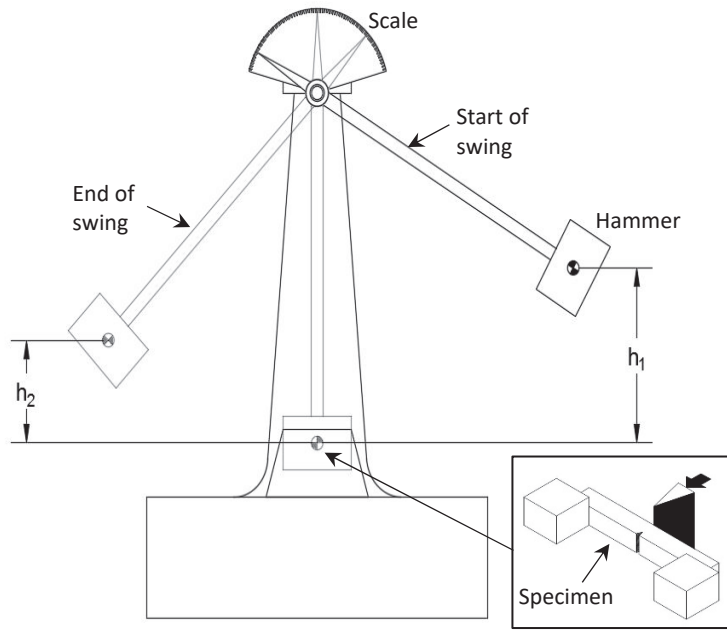


Figure 2-3. Charpy Impact Test Machine

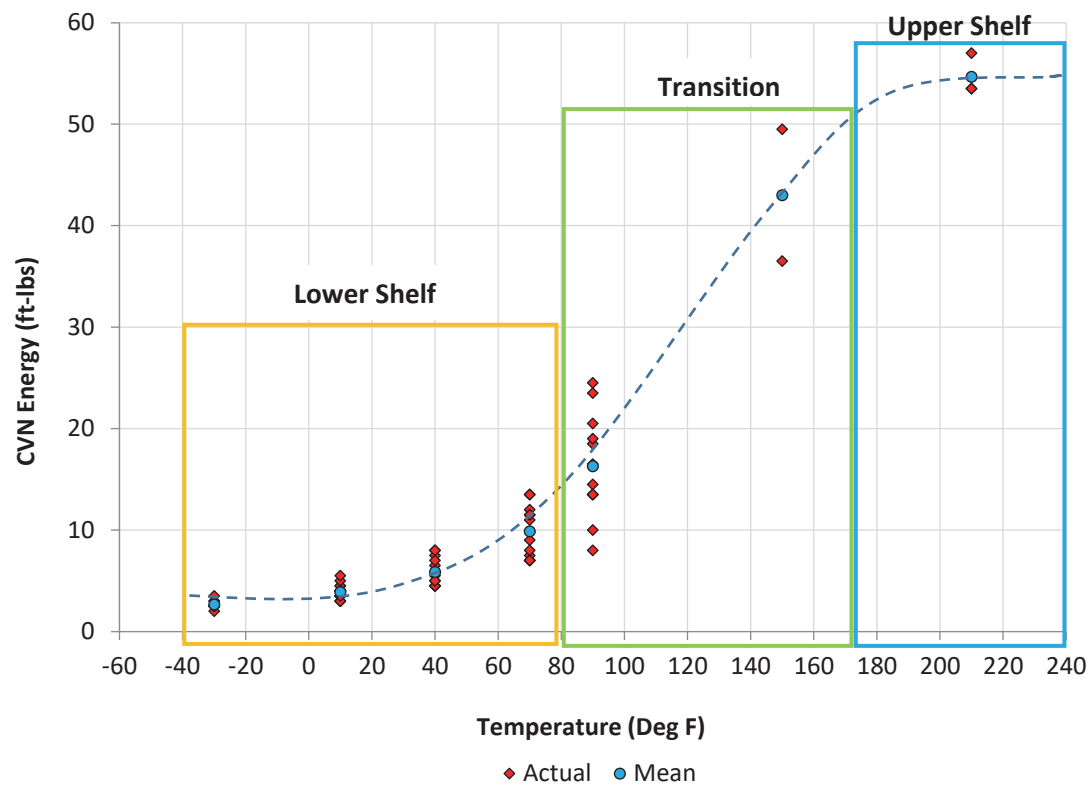


Figure 2-4. Sample of Charpy Impact Test Data Showing Transitions

in terms of fracture toughness. The three regions often referred to are lower-shelf, transition, and upper-shelf. The lower shelf is associated with brittle fracture and is the region of fracture mechanics where LEFM is valid. The upper shelf is associated with ductile fracture, or tearing, where significant inelastic deformations form shear lips on the fracture surface. Between the lower and upper shelves is called the transition region, associated with the brittle–ductile, or transitional, fracture behavior. Fractures in this region are characterized by a mix of brittle and ductile and often have larger scatter in the data. The temperature range for the transition region will differ for different steels, some being only a few degrees, while others might be spread over tens of degrees, such as is shown in Figure 2-4. It is noted that not all steels follow such an ideal curve, as shown in Figure 2-4, with some not exhibiting clear upper and lower shelves.

In present-day AASHTO specifications, the toughness requirements are enforced by requiring steels to meet minimum CVN values at specific temperatures rather than actual fracture toughness values. The temperature requirements depend on the minimum service temperature anticipated for the bridge. As stated, correlations can be used to estimate the material fracture toughness, K_{IC} , at the requisite temperature from the CVN data. The most commonly used correlation is the Master Curve method, which is valid for the lower-shelf and transition regions. The AASHTO fracture control approach is to ensure that the material possesses sufficient toughness at the minimum service temperature of a bridge in order to prevent lower shelf, brittle fracture. The CVN requirements for fracture-critical members are slightly higher in order to provide additional resistance to fracture.

2.3—FUNDAMENTALS OF FATIGUE

Fatigue is the initiation and propagation of cracks by repeated, or cyclic, stress ranges. Two different modes of fatigue damage occur in steel bridges, known as *load-induced* and *distortion-induced* fatigue. Load-induced fatigue is that due to the in-plane live-load stress ranges typically calculated by engineers during design or evaluation. Maintenance actions designed to address this type of cracking in steel bridges are provided in Chapter 4. Distortion-induced fatigue is caused by differential displacement of connected elements causing secondary stresses unaccounted for in typical design and evaluation. Details prone to distortion-induced fatigue are not categorized in the fatigue category tables in LRFD Design (AASHTO, 2017); instead, the design specification provides detailing rules that address this for new design. Maintenance actions aimed at repairing or retrofitting distortion-induced fatigue cracking in steel bridges are provided in Chapter 5.

All elements of steel bridges contain metallurgical or fabrication-related discontinuities, as well as stress concentrators (e.g., weld toes, holes, etc.). These locations are the most prone to fatigue cracking. The stresses that drive fatigue cracks can be far below the static yield strength of steel. There are three general stages of a fatigue crack: nucleation, crack growth, and final fracture. A time-lapse video that shows the accelerated timeline of a distortion-induced fatigue crack generated in a laboratory setting at Purdue University can be found here: <http://dx.doi.org/10.4231/R7W9573Z>. In the video, the first two states, nucleation and crack growth, are shown.

The nucleation phase of fatigue comprises the vast majority of the overall fatigue life of a given detail, estimated to consume as much as 90 percent, and is largely unaffected by material properties (Fish et al., 2015). However, the nucleation stage is highly dependent on pre-existing imperfections in the member. Early NCHRP laboratory testing of typical welded bridge details examined only full- or large-scale specimens. This aspect of the research was important because fatigue nucleation follows the “weakest link” concept, meaning that a larger volume of material will have a greater probability of containing imperfections that promote fatigue crack initiation. Weld inclusions, construction quality (e.g., nicks and gouges), material process inclusions, or other stress flow discontinuities will help to accelerate fatigue crack initiation.

The next stage is crack growth. The onset of this stage is not well-defined and most often will begin before visual inspection or nondestructive testing (NDT) methods will be effective in locating the crack. This stage is influenced by many factors, such as material composition and heat treatment, microstructure, type of product (e.g., plate, extrusion, forging), residual stresses, variations in material processing, and toughness. However, for steel bridge applications the primary variables are the type of detail and the stress range, S_r (Fisher, 1973). The reason for this is the local tensile residual stress at weld toes resulting from the constraint put upon the heat affected zone (HAZ) by surrounding base metal as the weld cools and attempts to shrink. The resulting tensile residual stress approaches the yield strength of the base material. Most of the fatigue life occurs within this region of high tensile residual stress. This means that under cyclic loading, the entire stress range, S_r , irrespective of being nominally compressive or

tensile, extends fatigue life. Consequently, $S_{\max}$ is usually at the yield point and S_r becomes the only stress parameter affecting fatigue life, while $S_{\min}$ essentially has no effect. This is no longer valid, however, for weld toes that have undergone post-weld impact treatment introducing plastic deformations that replace tensile residual stresses with compressive stresses. The effective stress range resulting from the superposition of the applied and residual stresses can be either tensile or compressive, depending upon their relative magnitudes. Accordingly, the propagation stage of a crack at the treated detail becomes dependent upon $S_{\min}$, as well (Roy & Fisher, 2003).

The final stage of a fatigue crack is fracture. Fracture was discussed in greater detail in Section 2.2 and the reader is referred there for more on this topic. A load-induced fatigue crack that is left to grow to a critical size will eventually result in either fracture or net section yielding of the structural member. Sometimes distortion-induced fatigue cracks will stop growing once a cracked detail becomes more flexible as a result of the cracking. Although this is possible, it is not recommended that distortion-induced fatigue cracks be left unmonitored or unrepaired. Repair and retrofit strategies for these and other details are presented below.

2.3.1—Nominal Stress Approach of Fatigue Design and Evaluation

Fatigue damage is cumulative over the life of a bridge detail. Determining the number of cycles at a given stress range is critical in evaluating or designing for fatigue. The *AASHTO LRFD Bridge Design Specifications* and the *Manual for Bridge Evaluation* use a nominal stress form of the stress-life approach in the design and evaluation of fatigue details. The stress-life approach was originally developed by German engineer August Wohler in the mid to late 1800s to analyze failures of railroad components. The key to this approach was the S - N curve, which is very similar to what is used today. An example of a modern S - N curve is shown in Figure 2-5, discussed more below, where S is the nominal stress range and N is the number of cycles until significant cracking is detected. Using the stress-life method, extensive experimental fatigue testing developed the modern fatigue categories for typical steel bridge details, which were based on nominal stresses (Fisher et al., 1974; Schilling et al., 1978; Keating & Fisher, 1986). The current fatigue details are summarized in Section 2.4.

The three most influential factors on fatigue life are stress range, number of cycles, and the detail type (referring to the orientation and configuration relative to the applied stress range). The stress range that drives fatigue cracking

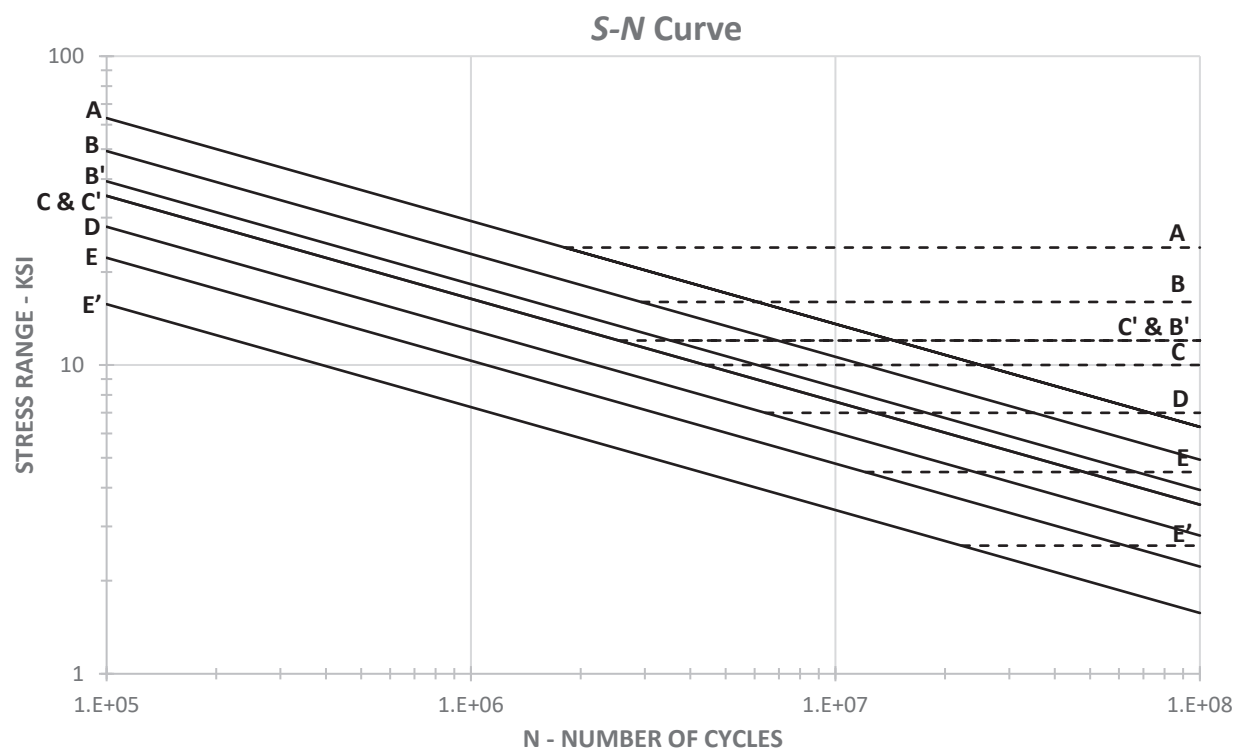


Figure 2-5. Modern S - N Curve Showing All AASHTO Fatigue Categories

is that produced by live loads. The range is the difference between the maximum and minimum stress in a given load cycle. Although other sources of live loads, such as wind and temperature change, produce stress ranges in a bridge, their contribution to the overall fatigue life is minimal in most cases when compared to traffic loads and is typically negligible. However, there have been reported cases of aeroelastic instability due to wind loading resulting in excessive vibration of truss hangers, for example, leading to fatigue cracking.

Only tensile stress ranges promote crack initiation and growth. As a result, any element of the bridge that is subjected to net tensile stress ranges after accounting for compressive and tensile dead load stresses must be designed or evaluated for fatigue. The design/evaluation does not directly take into account the tensile residual stresses from welding (hot spot stresses), though, as this is already accounted for in the fatigue category obtained from laboratory testing. This means that nominal stress ranges calculated using conventional axial and bending stress equations (i.e., P/A or Mc/I) are used to determine the stress range. The same is also true for non-welded details possessing discontinuities that generate stress concentrations (e.g., rivet holes). The stress concentration effect on the live-load stress range is already accounted for in the applicable fatigue category. This greatly simplifies the analysis while providing an accurate and consistent method for practicing engineers.

Fatigue test data used to develop modern fatigue categories were primarily obtained from constant amplitude laboratory testing. However, later testing by Fisher et al. (1983) verified the $S-N$ curve developed for details under constant amplitude loading can be used for predicting the fatigue life of details subjected to variable amplitude loading. Fluctuating traffic volumes, configurations, and axle weights provide for highly variable fatigue load spectrums on actual bridges. These are referred to as *variable amplitude spectrums*, such as that shown in Figure 2-6. Each cycle accumulates and contributes to fatigue damage; however, the larger the stress range, the greater the damage it causes. In design for fatigue, a single stress range (i.e., single truck) is used and represents the effective loading condition that accounts for the equivalent damage produced by the variable loading. Commonly known as the “fatigue truck,” the HS15 with a fixed rear axle spacing of 30 feet is the AASHTO loading model used for this purpose. The *AASHTO LRFD Bridge Design Specifications, 9th Edition* (AASHTO, 2020) actually uses the HS20 truck with a fatigue load factor of 0.8 (finite life) and 1.75 (infinite life) to effectively achieve the stress range produced by the HS15, but without adding another load design truck. When field measurements are recorded for variable amplitude load spectrums on existing bridges, a conversion to an equivalent constant amplitude fatigue loading spectrum is performed. This must be done in order to compare the fatigue demand on the detail to the constant amplitude fatigue categories in the *AASHTO LRFD Bridge Design Specifications*.

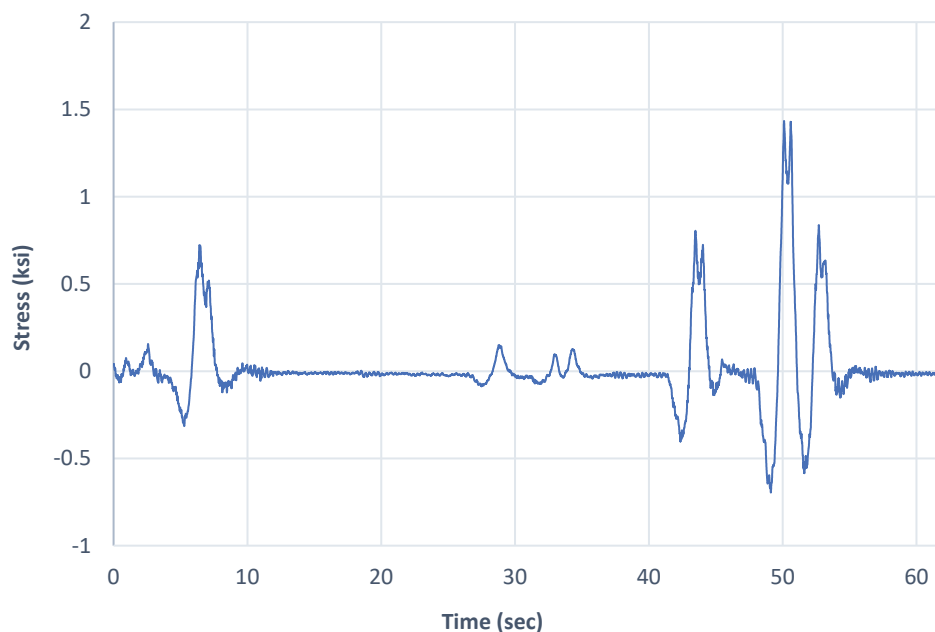


Figure 2-6. Sample Variable Amplitude Load Spectrum from Field Monitoring

A fatigue cycle constitutes a complete peak-to-peak stress-range oscillation. This should not be confused with reversals, which are only half of a complete cycle. Depending on the element of the bridge being evaluated or designed, a single truck may cause one or many fatigue load cycles. For example, the bottom chord of a simple-span through-truss will experience only one fatigue cycle for a single truck, while a hanger in that same bridge might see two or more fatigue cycles from the same truck. Secondary members, such as floorbeams, stringers, or attachments, may also be subjected to multiple cycles for a single truck. This is addressed in the *AASHTO LRFD Bridge Design Specifications*.

Steel bridge details possessing similar fatigue resistance have been grouped into categories and assigned letters A through E'. There are eight categories in total, listed here in descending order of fatigue resistance: A, B, B', C', C, D, E, and E'. An AASHTO design curve, such as that seen in Figure 2-7, represents the fatigue resistance for each respective category. The sloped portion of the curve represents the number of cycles at a given stress range that can be expected for a detail before significant cracking will occur. The region of the plot below the sloped line is referred to as the “finite life” region. The horizontal section of the curves is called the Constant Amplitude Fatigue Limits (CAFL) or Constant Amplitude Fatigue Thresholds (CAFT). These terms are synonymous and used interchangeably in the literature. If the maximum stress range is less than the CAFL for a given detail category, it can be expected that an infinite number of cycles can be resisted without incurring fatigue damage. The region of the curve below the CAFL is referred to as the “infinite life” region.

The S - N curves can be represented by a power law relationship:

$$N = \frac{A}{S^3} \quad (2)$$

where N is the number of stress cycles, S is the nominal stress range, and A is a detail constant (the y-intercept). Notice that the stress range is cubed in Equation 2, implying that small changes in the stress range can have a significant impact on the fatigue life estimation. This should be remembered for two reasons. First, from a design/evaluation perspective it means that accuracy in the estimation of the stress range is very important, and where estimated remaining fatigue life is found to be unacceptable, more sophisticated analysis or field measurement may result in a

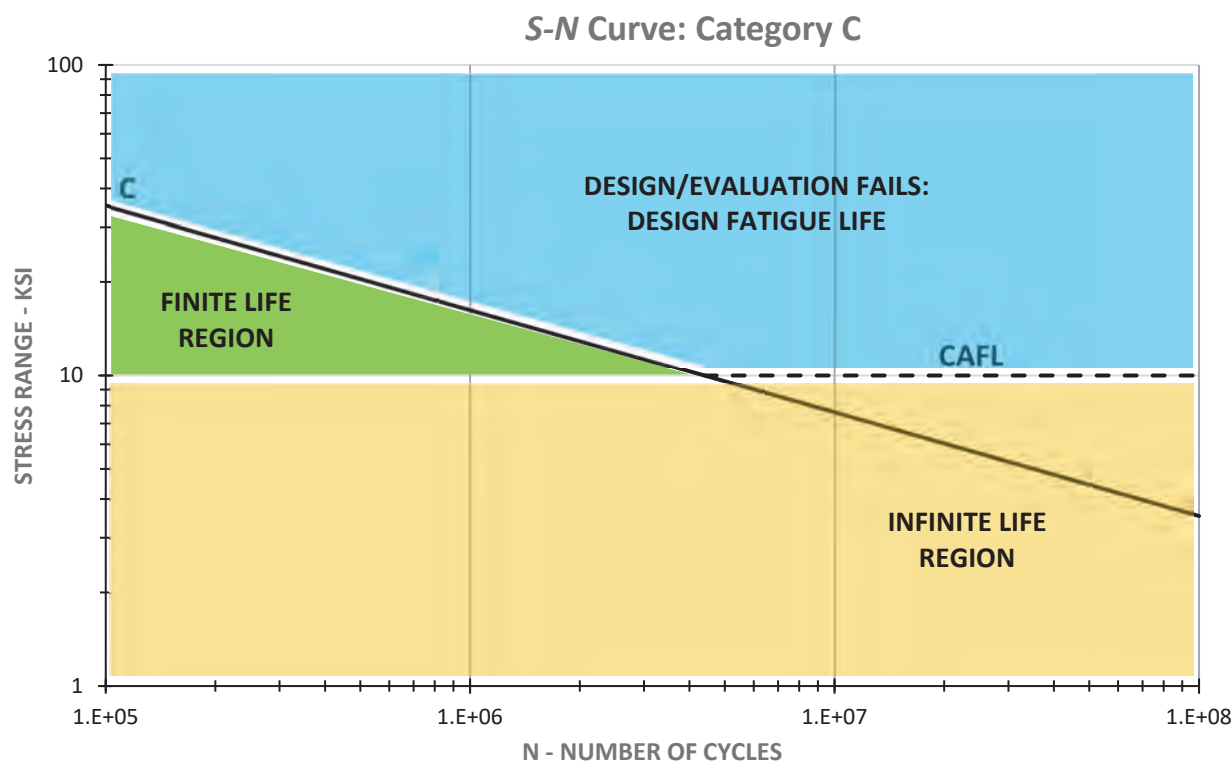


Figure 2-7. Category C S - N Curve Illustrating the Regions of the Plot

lower stress range input and therefore an increase in the calculated fatigue life. Second, from a retrofit or repair perspective, it means that implementing a modification to a detail that will reduce the stress range, stress concentration, or both will have the greatest impact on the fatigue resistance improvement.

2.4—CONSIDERATIONS FOR FATIGUE DETAIL CLASSIFICATION

As stated, LRFD Design divides steel bridge details having similar fatigue resistance into groups, or categories (AASHTO, 2017). The development of these categories has changed very little since early development and subsequent research has supported early findings (Fisher et al., 1970; Fisher et al., 1974; Keating & Fisher, 1986). Experience suggests that Categories A, B, B', C', and C will rarely have problems in the field due to their higher CAFs, as they generally exceed real live-load stresses in most highway bridges. Many of these details continue to be implemented in new designs. The majority of cracking will focus around Categories D, E and E'. These details have lower CAFs of 7 ksi, 4.5 ksi, and 2.6 ksi, respectively, making them more susceptible to typical live-load stresses. Due to their low fatigue resistance, it is also recommended (and reasonable) that Category E and E' details be given special attention during inspections.

LRFD Design (AASHTO, 2017) provides illustrations of typical fatigue details comprising each category in Table 6.6.1.2.3-1. The illustrations are not intended to be exclusive; rather, they are examples to be used as guides in determining category classifications of other details found on existing bridges. Engineering judgement can and should be used, giving consideration to how an existing detail being evaluated is configured and how stresses flow through it in order to classify the detail. Additionally, Fish et al. (2015) provides guidance on several nonstandard details often found in steel structures.

Stress flow through the detail being evaluated is a very important consideration knowing that fatigue resistance is significantly affected by stress concentration. Take, for example, Figure 2-8, where results from a finite element model (FEM) have been rendered with primary longitudinal stress plots. The three welded attachments represent generic welded plates attached to a web plate. The shortest (top) plate is 2 inches long and $\frac{3}{8}$ inch thick and would be considered a Category D detail at the ends. The middle plate is 20 inches long and also $\frac{3}{8}$ inch thick and would be considered a Category E detail at the ends. The bottom plate is also 20 inches long, but is 1 inch thick and would be considered a Category E' detail at the ends. Notice how the stress concentration changes from the top plate to the bottom plate, as seen by the increasing accrual of orange and red contours near the ends of the attachments.

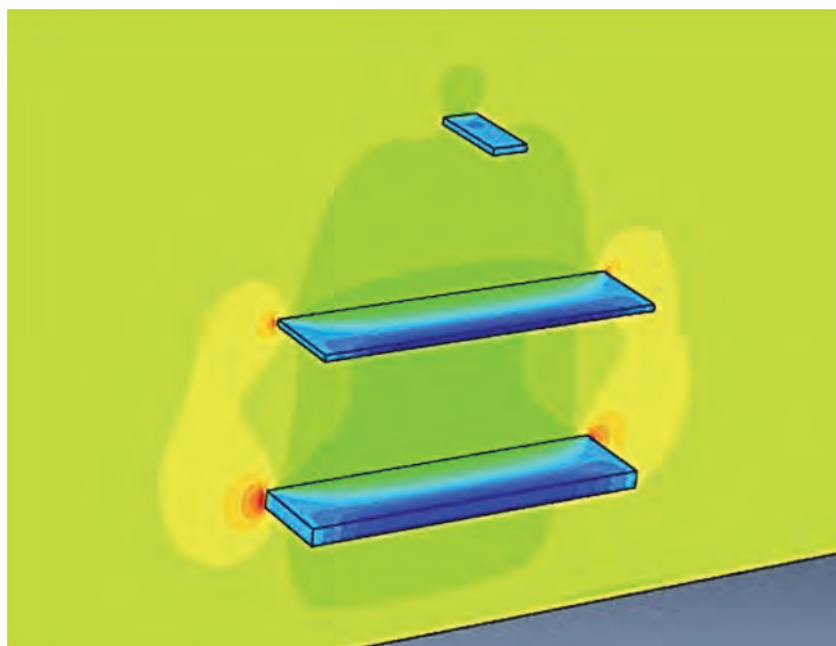


Figure 2-8. FEM Illustrating Differing Stress Concentrations for Length and Thickness Effects

This FEM nominally illustrates how a longer attachment “collects” more stress than a shorter one of equivalent thickness and how similarly a thicker attachment “collects” more stress than a thinner one of equivalent length. Stresses flow from the web plate into each of the attachments and back out again, concentrating at the ends of each. A similar comparison has been shown where with increasing radii at the ends of the attachments the stress concentration is reduced, as indicated in LRFD Design, Table 6.6.1.2.3-1, Description 4.3.

There are many details and conditions that remain unclassified, such as pins, forged welds, rolling flaws, extraneous welds, and very importantly, details which are cracked. The AASHTO fatigue categories only apply to uncracked details. Once a detail has cracked, a fracture mechanics analysis must be employed to determine remaining life, fitness-for-service, or both. Finally, details susceptible to distortion-induced fatigue are also not classified by the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017). However, Connor & Fisher (2006) reported that several tests have been conducted supporting classification as Category C, as long as the stress range is calculated in a manner consistent with how it was measured during the testing. Distortion-induced fatigue is discussed in greater detail in Chapter 5.

2.5—CONSIDERATIONS FOR EVALUATING EXISTING STEEL BRIDGES

When evaluating an existing steel bridge, the year it was built will provide some indication of whether fatigue and fracture would be expected to be a concern. Bridges built before 1974 will most likely have fatigue prone details because they would not have benefited from implementation of the 1974 fatigue design specifications. Bridges built before 1978 would not have benefited from the detailing rules and CVN requirements implemented in the Fracture Control Plan. Bridges built prior to the mid-1980s would not have benefited from research into distortion-induced fatigue and may likely be prone to web gap fatigue cracking. Additionally, bridges built prior to 2009 may not have benefited from modern understanding of the web gap between perpendicular attachment welds that will help to resist onset of constraint-induced fracture. Although these dates help provide a general guide, it should also be considered that some owners are slower to adopt new specifications and so bridges built some years after these dates could also possess critical details.

2.5.1—Fatigue Evaluation of Existing Steel Bridges

As stated, there are two types of fatigue cracks that can occur in steel bridges. They are distinguished by the fundamental differences of what drives the nucleation and propagation. Load-induced fatigue is driven by live-load stress ranges in primary members and causes cracks more prone to fracture of primary load-carrying members. Distortion-induced fatigue is driven by unanticipated relative movement between connected elements. Typically, this is caused by relative displacement of the adjoining members (e.g., adjacent primary girders connected by cross bracing) due to disproportionate traffic loads. Fisher (1984) also documented a few rare cases where distortion-induced fatigue occurred during shipping and handling of girders prior to being put into service. These instances are extremely rare, however, and typically this type of cracking occurs under in-service live loading and takes many years, even decades, to develop detectable cracks.

Due to a random distribution of defects in steel bridge members, there is considerable scatter in the experimentally derived fatigue life estimations. For this reason, the *AASHTO Manual for Bridge Evaluation* (MBE) (AASHTO, 2016) establishes three levels of finite fatigue life estimation, each progressively less conservative (increasing risk):

- Minimum expected fatigue life (equivalent to the LRFD Design design life—two standard deviations below mean life)
- Evaluation fatigue life (considered a conservative fatigue life for evaluation purposes—one standard deviation below mean life)
- Mean fatigue life (considered the least conservative, but statistically the most likely, fatigue life)

The difference between the three levels of finite fatigue life is defined by the resistance factor, R_R , which was updated in research conducted by Bowman et al. (2012). Using the resistance factors, it is apparent that mean life can be as much as 1.6 to 2.1 times larger than minimum life for typical steel bridge details prone to fatigue. The MBE (AASHTO, 2016) offers two strategies when estimated remaining fatigue life is deemed unacceptable, recognizing

that limiting the useful fatigue life of bridges to the Minimum (design) life may be overly conservative and uneconomical. The first strategy is the recalculation of fatigue life using the Evaluation life or Mean life. The Mean life simply has a higher probability of cracking. The MBE clearly states, however, that this is only done if the evaluator is willing to accept greater risk of fatigue cracking, suggesting the following possible reasons to justify acceptance of higher risk:

- Long satisfactory fatigue life of the detail to date
- A high degree of redundancy (considering load path, structural, and/or member-level redundancies)
- Increased inspection effort (shorter inspection interval)
- Or some combination of the above

The second strategy suggested is usually preferred, which is recalculating fatigue life using more accurate stress range data. Generally, a more accurate model or data obtained from field measurements reveals actual in-service stresses or numbers of cycles to be less than using simplified approaches. Some sources of data improvement, which generally result in a lower stress range at the detail of interest, include:

- Using field instrumentation to obtain actual effective stress ranges and numbers of cycles.
- Using weigh-in-motion (WIM) data to determine the effective truck weight, which can improve accuracy.
- Improved average daily truck traffic (ADTT) counts, which would improve the accuracy of the number of cycles a given fatigue detail is submitted to each day. (These data can also be collected while using field instrumentation.)
- Better count of the number of cycles per truck passage on a given detail. (These data can also be collected while using field instrumentation.)

Equation 1 shows that stress range is a cubed variable in the fatigue life calculation. Hence, obtaining more accurate stress range data will have the largest impact on the estimated fatigue life. Field testing and monitoring is a very effective way to obtain more accurate live-load stress range data at a detail in question. Normally, a measured stress range will be less than a theoretical one, improving the calculated fatigue life. In addition, the data related to the number of cycles per day is also measured through the same instrumentation. However, to ensure an accurate assessment, it is critical that monitoring include uncontrolled, or random, traffic for a sufficient period of time. Connor & Fisher (2006) suggest that a monitoring period between two and four weeks is generally sufficient to establish the stress range histograms used in fatigue evaluations. However, any unique or seasonally-driven fluctuations in truck (axle) weights should also be considered so that an accurate sampling of extreme overloads or permit loads are captured.

2.5.2—Fracture Evaluation of Existing Steel Bridges

Evaluation for fracture is more commonly focused around bridge members identified as fracture critical. Fracture critical members are subjected to more rigorous inspection; however, inspection of FCMs only serves to detect fatigue cracks or identify CIF-prone details that contribute to the risk of fracture. Due to the rapid propagation of fracture, it is not something that can be inspected for in the same way that fatigue cracks are inspected and monitored. Additionally, the age of the bridge provides no indication of the risk of fracture where fracture-prone details exist. Even though a bridge with CIF details may be decades old and never experienced a fracture, this does not mean that it is at lower risk for constraint-induced fracture in the future. It is very important to recognize that leaving a CIF detail unretrofitted is accepting the risk of potential fracture no matter the age of the bridge.

Fitness-for-service (FFS) evaluations can be used to determine acceptability of existing fatigue cracks in steel bridges. FFS is based in fracture mechanics and can be quite complex requiring accurate knowledge of the material fracture toughness, flaw size, live load, dead load, and residual stresses. Due to the complexity of an FFS, it is generally reserved for bridges of higher importance. Where bridges are known to have CIF-prone details, FFS is not

applicable since there is no known initial flaw which can be input into the fracture mechanics calculations (Connor et al., 2007). Rather, preemptive retrofit is recommended using one of the strategies presented in these Guidelines.

2.6—CONSIDERATIONS FOR URGENCY OF REPAIR AND RETROFIT

The urgency for retrofit of a fatigue-prone or fracture-prone detail, or repair of a cracked detail is ultimately determined by the bridge owner. Each situation will be unique, whether it is the bridge design, the feature carried, the feature intersected, and the material from which the bridge is built, or some combination of all of these and other factors. The risk associated with each case needs to be considered carefully. The following is not meant to be a comprehensive list of considerations for this topic, nor is it intended to provide a prescribed method for determining urgency. These cannot practically be accomplished this way. Determining the urgency of repair or retrofit of a steel bridge is founded upon structural analysis; personal, agency, and industry experience (documented and otherwise); research; and engineering judgement. The topic of urgency involves the balance of economics and public safety. Typically, this is referred to as a risk analysis which accounts for both likelihood of the event happening and consequence of the event, presuming it occurs. Obviously, public perception, government policy, and political sway are also often a large part of the decision making, but these factors will not be discussed here.

Topics to consider when determining urgency of repair or retrofit:

- *Is the structure redundant?* A nonredundant structure likely has greater consequence of unstable crack propagation than a redundant one. Hence, a nonredundant bridge should be given higher urgency than a redundant one. There are three types of redundancy, namely load path redundancy, structural redundancy, and internal member redundancy. Load path redundancy refers to the number of primary load-carrying members between points of support. Typically, a state will consider a two-girder bridge to be non-load path redundant, whereas a three- or four-girder bridge (or more) would be considered load path redundant. Structural redundancy refers to continuity of the primary members over supports, or other three-dimensional aspects such as transverse member spacing. Internal member redundancy refers to built-up members where mechanical connections isolate member components against fracture propagation through the entire cross section. This type of redundancy is also known as a “fail-safe design” and is often utilized by the aviation industry. The assumption is that the surrounding components are designed to pick up the load of the fractured component and that inspection will identify the fractured component in time to have it repaired without another component also failing.
- *Is there cracking already present? If so, does it appear to be rapid growth or slow growth? And what is the orientation of the crack relative to primary stresses?* As discussed previously, the presence of cracks at a detail means that the fatigue life has already been consumed and the detail is no longer categorized by AASHTO. The follow-up questions help to identify the level of urgency that could be necessary for known cracks on a steel bridge. Rapid growth cracks typically are given a higher repair urgency because they indicate that a fracture has occurred (typically 4 or more inches growth between routine inspections, as a rule of thumb). Whereas, fatigue cracks are considered to have slow growth (approximately 3 or less inches growth between typical two-year routine inspections, as a rule of thumb). Cracks oriented perpendicular to primary stresses are more prone to fracture, and propagation is driven by the primary stresses in the member. Higher urgency should be given to cracks that are oriented this way. Distortion-induced cracks that start out parallel to primary stresses, can turn under the increasing influence of primary stresses when left to propagate unchecked.
- *Is the live-load stress range known? If so, is it greater than the CAFL?* Some owners reported to the authors that their agency has a policy to preemptively retrofit Category E and E' fatigue details. For these agencies, the urgency is already decided. However, knowing that a poor fatigue detail, such as E or E', exists on a steel bridge may not always be enough to determine urgency for repair where policy has not already determined that. Lower urgency can be given to poor fatigue details where no cracking is present and so long as the stress range can be confidently determined to be less than the CAFL. This could be determined through field monitoring projects that typically would cost far less than a bridge-wide retrofit project.
- *Is the detail of concern in a location subject to net tension or net compression?* Cracks can initiate at welded details in compression members due to the high residual tensile stresses. However, once the crack grows out

of the localized residual stress zone it will no longer propagate under net compressive stress cycles. Higher urgency should be given to members in net tensile zones, particularly if they are known to have cracks.

- *Are the material properties known?* Where the material properties are not known, conservative assumptions would need to be made in order to ensure reliable operation. At the same time, higher urgency should be given to details where material properties are known to have low fracture toughness.
- *What are the consequences of potential failure? What is the feature carried and what is the feature intersected? What is the importance of the structure?* Clearly an interstate, either carried or intersected, where high volumes of traffic are traveling at high speeds, would be at much greater risk in the event of failure than would a bridge intersecting an unnavigable waterway. Bridges that provide exclusive access to hospitals, or where if closed would cause large detours with safety and economic consequences should be given higher urgency. All of these consequences of the worst-case scenario should be considered when determining the urgency of repair or retrofit for a bridge.

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CHAPTER 3

FUNDAMENTAL TECHNIQUES FOR FATIGUE REPAIR AND RETROFIT OF STEEL BRIDGES

A number of general repair and retrofit techniques are presented in this chapter, which will be referenced in subsequent chapters. These effective techniques are fundamental to many of the repair and retrofit strategies proven through research and in practice. General recommendations are made regarding the type of tools to use, how to position them, and so forth, based on the combined experience of research and practicing engineering professionals across the industry.

3.1—BASIC INSPECTION TECHNIQUES

Inspection for fatigue should focus around elements in tension, particularly those known to have poor fatigue resistance, particularly when designed prior to the AASHTO fatigue provisions. Fatigue detail Categories E' and E, for example, should receive the most attention and resources because they are the most likely to experience cracking if not designed properly. Due to the repetition of details in a typical steel bridge, when one crack is found, it is likely that another similar detail will also have a crack at the same location in regions of similar stress range. Thus, repair and retrofit planning should always include thorough inspection of all fatigue-prone details. The same is true when a fracture is discovered. Specifically, other similar details on the same bridge should be inspected thoroughly to ensure all fracture locations or susceptible details are documented and repaired.

Several methods of nondestructive testing (NDT) are recommended when inspecting for cracks before, during, and after retrofit or repair of steel bridges. The most common methods of NDT include visual testing (VT), liquid penetrant testing (PT), magnetic particle testing (MT), electromagnetic testing (eddy current) (ET), ultrasonic testing (UT), and phased array ultrasonic testing (PAUT). Surface cracks are detected using VT, PT, MT, or ET. Internal cracks and defects can be found using UT or PAUT. When using VT, it is recommended that a good quality light source and minimum 10× magnification be used to enhance contrast, making it easier to identify cracks. PT, MT, and ET further enhance detectability and are recommended in the following repair and retrofit guidelines. If the reader wishes to learn more about any of the NDT methods above, more information can be found in Washer (2014b) or ASNT (2016).

3.2—PAINT COATING TECHNIQUES FOR LOCAL REPAIRS

All of the repair and retrofit techniques discussed in these Guidelines result in exposure of bare steel surfaces through drilling, cutting, or grinding. If left uncoated following the repair or retrofit work, these surfaces may quickly corrode. The following provides some basic guidelines for local spot repair to coatings that will help ensure the implemented retrofit or repair will be preserved for many years of service. Proper surface preparation is key to a successful coating system and may require the use of cleaning media and profiling tools. Satisfactory surface profiles can also be obtained by use of rotating sanders or grinders (rather than profiling tools); however, be careful to not over-polish, or burnish, the surface as this will lead to premature adhesion failure of the paint.

Refer to the respective bridge owner provisions for any paint coating requirements, as these take precedence over suggested coating methods provided in these Guidelines. Many bridge owners have vetted and preferred coating systems for application over tool-cleaned (non-blasted) steel surfaces. This typically will also include surface preparation requirements either specified by the owner or by the manufacturer of the owner-preferred coating systems. In the absence of such a preferred coating system, the following may be used as a general recommendation for surface preparation and coating application.

***WARNING!** If you scrape, sand, or remove old paint from any surface coated with lead-based paint, you may release lead paint dust. LEAD IS TOXIC. EXPOSURE TO LEAD DUST CAN CAUSE SERIOUS ILLNESS. Wear a NIOSH-approved respirator to control lead exposure when working with lead-based paint. Carefully clean up with a wet mop or HEPA vacuum.*

Surface profile:

During retrofit and repair procedures outlined in these Guidelines, bare steel surfaces will be exposed and smoothed. Suggested grits for sanding wheels, etc., are recommended to obtain a surface free of stress risers that helps ensure the highest level of fatigue resistance. However, in order for a quality paint coating system to adhere to the repaired area, a proper surface profile must also be made. The Society for Protective Coatings (2015b) recommends a minimum 25 micrometer (1.0 mil) surface profile for bare steel surface preparation. This standard is suitable where roughened, clean bare metal is required but where abrasive blasting is not feasible, which will be the case for many of the localized repair and retrofit procedures outlined herein. However, if abrasive blast cleaning is required by the bridge owner, this can also be used following completion of the repair or retrofit without compromising the fatigue resistance of the repaired area. The following steps and recommendations are provided for cases where blast cleaning is not feasible or economical, and hand tool cleaning and surface profiling methods are needed.

1. With the bare steel surface exposed, feather the remaining, intact surrounding coating with light sanding so as to remove any “step” from bare steel to remaining paint at the edge of the repair area. This will help ensure a more uniform appearance once completed.
2. Use a profile-producing power tool to roughen the surface slightly, such as a roto-peening tool or a bristle blaster tool. Be sure that the roughening tool does not produce burrs and gouges.

Note that many profiling tools may not be able to access the interior of drilled holes and copes, etc., created as part of many of the retrofit and repair procedures. Sanding grits of 80 to 100 grit flap wheels may provide a properly roughened surface for painting. However, ensure the flap wheels used for the final surface roughening are not worn and that finish sanding does not burnish (mirror finish) the surface.

Surface cleaning:

3. Remove foreign material such as rust, drilling shards, and sanding dust, using a brush, scraper, cloth, dry compressed air, or vacuum.
4. Perform solvent cleaning to remove grease, oil, dust, and debris, etc., from the repair area. Ensure solvent is compatible with the repair paint. Direct spray and scrub, wetted rag scrubbing, steam cleaning, or vapor degreasing methods can be used. Use clean solvent and clean rags or brushes for the final wiping (SSPC, 2015a).

Painting:

5. Apply one of the following coating systems to the profiled and cleaned surface:
 - a. Organic zinc rich primer followed by compatible top coat. These paints can typically be sprayed, rolled, or brushed on. Follow manufacturer recommendations.
 - b. Aluminum pigmented epoxy. These paints are typically two-part mixes that are mixed one-to-one, which can be sprayed, rolled, or brushed on. Follow manufacturer recommendations.

3.3—HOLE DRILLING TECHNIQUES FOR CRACK ARREST

Stop-holes, or crack arrest holes, are holes drilled at the tip of a crack intended to blunt the tip and slow or completely prevent growth. Note that the stress concentration effect for a notch is found to be inversely proportional to the radius of the notch (Anderson, 2005). Mathematically this means that as the radius of the notch approaches zero (infinitely sharp crack), the stress concentration at the tip of the crack approaches infinity. Obviously, no material is capable of resisting infinite stress. The infinitely sharp crack, in reality, is a purely theoretical concept. Plastic deformation at the crack tip in ductile materials such as steel helps to somewhat blunt the tip and prevent an actual infinite stress concentration. For most steels, however, relying on the crack tip plasticity to sufficiently blunt the tip to prevent growth is not enough, especially as temperatures decrease. Drilled holes have proven to be an economical

and highly effective method for repair and retrofit of steel bridge fatigue. Drilling a large radius hole at the tip of the crack increases the blunting effect of the crack tip significantly, dropping the stress concentration to approximately 3 (depending on the hole radius)—far from the theoretically calculated infinite stress. The effectiveness of drilled holes has been proven in the field over many decades of use. In addition to blunting existing crack tips, drilled holes have also been effectively used to prevent fatigue cracking by isolating details, as well as for providing an effective means to arrest a running fracture. For example, the large-hole retrofit discussed in Section 5.1.2.4 uses drilled holes to increase detail flexibility, thereby reducing the driving stress condition of distortion-induced fatigue while also isolating it from crack propagation beyond the immediate weld toe.

Drilled holes have been shown to be effective for load-induced fatigue applications as well as for distortion-induced fatigue. This repair can be a very effective strategy for long-term repair or retrofit when properly sized and installed. The aspects of a proper installation have been detailed below, which includes tactics for hole placement, hole diameter, and hole edge condition. Another benefit of hole drilling is that it is relatively inexpensive to execute, requiring lower-skilled labor and portable and affordable equipment. Often the drilled hole should be the first measure taken upon discovery of a crack in a steel bridge and generally is recommended as part of any repair project where known cracks exist.

The equipment recommended for hole drilling includes a magnetic-based drill and annular cutter, such as that seen in Figure 3-1. Although hand drills have been shown to be effective, even for larger diameter holes, when operated by trained and experienced crews, magnetic-based drills offer superb control and precision for a wide range of hole sizes. The greatest challenge can be positioning the drill in tight spaces where intersecting elements of a detail leave little room for the drill to seat properly.

LRFD Design (AASHTO, 2017) conservatively categorizes an open hole in a member as a Category D fatigue detail. Fisher et al. (1987) found that plates with open holes yielded fatigue strengths that exceeded the Category C resistance curve. This conclusion was also supported by research carried out by Brown et al. (2007). Hence, drilled holes installed according to the recommendations provided in these Guidelines can be expected to have a fatigue resistance of Category C. Any deviation from the recommended procedures for drilled holes may provide a fatigue resistance of less than Category C, but not less than Category D. In most cases Category C resistance would satisfy fatigue life requirements, particularly when considering that the day the hole is drilled is day “one” for the fatigue life of the hole, even if the bridge itself is decades old, and may well provide sufficient fatigue resistance for the remaining service life of the bridge. This depends on the future stress ranges and number of cycles. By installing a fully-tensioned high-strength bolt (F3125 Grade A325 or A490) with F436 hardened washers on both sides of the plate, however, the fatigue category can be improved further to Category B. This approach is recommended for load-induced fatigue cases. In circumstances where the additional flexibility offered by the drilled hole improves the performance of the cracked detail, such as for distortion-induced fatigue cracks (web gap fatigue cracks), it is not desirable to add the pretensioned bolt because of the added stiffness.



Figure 3-1. (Left) Example of Magnetic-Based Drill, (Right) Annular Cutter Used for Hole Drilling

3.3.1—General Considerations for Hole Drilling to Arrest Cracks

Although it is not difficult to perform hole drilling to arrest cracks, if certain important steps are neglected or carelessly performed, it could lead to an ineffective repair or retrofit. These types of mistakes are common and often attributed to a lack of training. A survey of state owners reported that over 52 percent of the respondents at some point had a fatigue repair implemented in their inventory that had failed to repair a fatigue crack. Approximately 67 percent of those failed fatigue repairs were directly the result of missed crack tips during hole drilling repairs. The importance of catching the crack tip within the drilled hole cannot be overemphasized. If the tip is missed, the repair attempt will have no effect, and the crack will most likely continue to grow. At a minimum, this results in the cost and possible service disruption of repeated repair attempts in the future, as well as additional material removed from the steel member when another drilled hole has to be made to arrest the crack. It could also leave the member vulnerable to fracture if the crack is left to grow to a critical length.

Crack tunneling can also contribute to missed crack tips if the engineer or technician is unaware of this phenomenon. Crack tunneling is the term used to describe the preferential advance of the crack tip at the center of a plate with the edges lagging behind. Additional triaxial restraint through the thickness of the plate can promote slightly quicker growth rates with a hidden leading edge, particularly in thicker plates. An extreme example of crack tunneling is shown in Figure 3-2, where a drilled hole was installed to attempt to arrest a crack caused during an impact event. The corroded surface indicates the extent of the previous crack, and the bright silver surface indicates the new fracture that propagated instantaneously during another impact event some months later. Notice that the previous crack tunneled ahead of where it breaks the surface of the plate. Furthermore, surface inspection methods such as MT or PT could not have detected the leading crack tip toward the middle of the plate thickness; UT could have been used to detect it, however.

A similar effect has been observed for penny-shaped fatigue cracks that will be one length on a given side of a plate and appear to be shorter on another as the crack transitions to a through-thickness crack. For this reason, NDT should be performed on both sides of the cracked plate to ensure the longest dimension is characterized and the leading edge is arrested with the drilled hole. A sketch illustrating an extreme example of this concept is provided in Figure 3-3, which shows the cross section view of a typical interior plate girder with welded gusset plates. The hatched section indicates the uncracked section. Notice that if inspection only occurred on the left side of the web



Figure 3-2. Crack Tip Tunneling Contributing to a Missed Crack Tip That Later Fractured

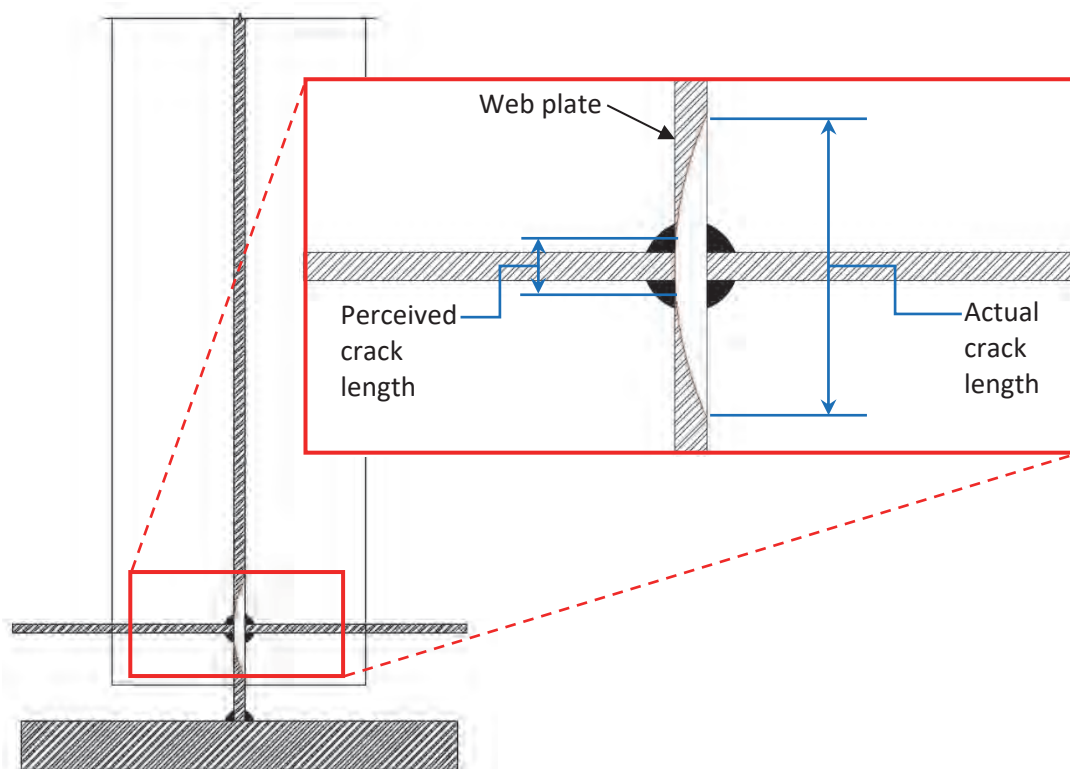


Figure 3-3. Crack Length Appears Different on Each Side of a Plate as it Transitions to Through-Thickness

plate, the crack length would be severely mischaracterized and it would be possible that the crack tip would not be removed during hole drilling.

Correct hole placement, relative to the crack tip, is one of the most important considerations for effectively drilled holes. Consider the examples of hole placement illustrated in Figure 3-4, as first proposed by Fisher et al. (1980). The top example, A, shows a case where the crack tip is entirely missed by the drilled hole and the crack tip is allowed to continue growing. The second, B, depicts an acceptably placed hole because it adequately captures the crack tip. The third scenario, C, is the recommended placement. The hole is drilled such that the edge of the annular cutter just intercepts the crack tip. This approach is recommended because it better safeguards against the possibility of missing the tip. Even if the hole is inadvertently drilled slightly ahead of the crack tip, the crack will only be able to grow until it intercepts the hole and then will become arrested. However, it is not necessarily recommended that the hole be purposefully drilled away from the crack tip as if to anticipate arrest. All efforts should be made to locate the crack tip and place the cutter such that it intercepts the tip. Once the hole has been drilled, a couple of simple checks can be done to determine the hole placement, as follows:

- If the removed core splits in half, then the crack tip has been missed and the hole placement corresponds with scenario A in Figure 3-4. The crack tip should be relocated using MT or PT and then another hole drilled, or grinding should be performed to remove it.
- If the crack can be seen on one edge of the hole in the drilled plate, but not on the opposite side, then the tip has been adequately removed (see Figure 3-9). The crack may also appear on one side of the removed core, in which case the hole placement corresponds with scenario B in Figure 3-4.
- If the crack can be observed on one edge of the drilled plate, but not on the opposite side, and also cannot be observed anywhere in the removed core, then the hole placement corresponds with scenario C in Figure 3-4.

The diameter of the drilled hole can also impact the effectiveness of the repair or retrofit. Larger diameter holes will further reduce stress concentrations while also lowering the odds of missing the crack tip should it be tunneling.

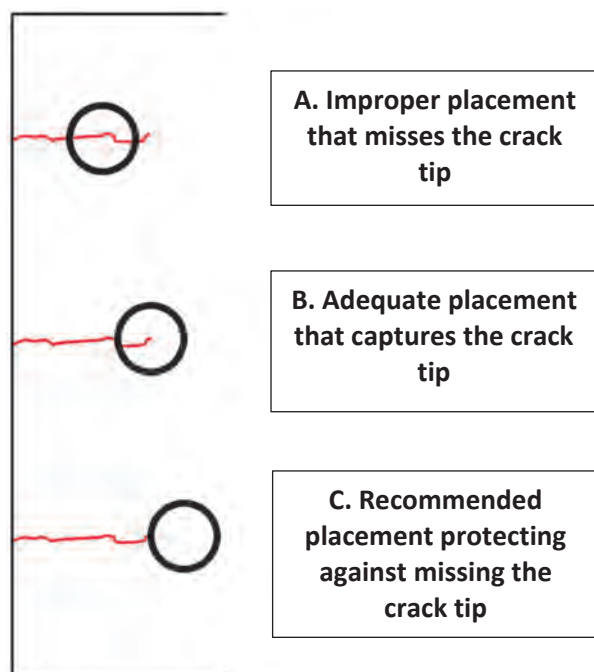


Figure 3-4. Improper and Proper Drilled Hole Placements for Crack Arrest

A review of literature and case studies shows that different diameter drilled holes have been implemented with varying levels of success. Figure 3-5 shows an example of the “Swiss cheese” effect that can occur when an insufficient hole size is used (often combined with insufficient edge conditions as discussed below) or when the crack tip is missed. Fisher et al. (1980) suggested an engineering calculation that can be used to estimate the minimum hole size, which may estimate a hole size that is less than 1 inch for typical highway bridges. However, experience has shown that enforcing a minimum hole diameter of at least 1 inch will help ensure sound performance (Dexter & Ocel, 2013), while increasing the size to between 2 and 4 inches, when possible, may provide even better results (Fish et al., 2015), in particular when trying to mitigate cracking due to out-of-plane distortion. Net section yielding could become a concern and should be considered, particularly when the crack is located in a tension flange of a long-span bridge versus a short-span bridge web plate, for example. In all cases, however, a drilled hole at the end of the crack tip will always be better than leaving a crack tip “as is.” Typically, when a drilled hole is used to repair load-induced fatigue cracks, the hole will also be filled with a fully-tensioned high-strength bolt. This practice further improves the fatigue resistance of the hole by instating a compressive stress field around the hole’s perimeter. Larger holes, such as 2- to 4-inch holes, are normally used in distortion-induced fatigue repairs and retrofits where increased connection flexibility improves the fatigue resistance.

Another important consideration is the edge condition of the hole. The edge condition of the hole may very well impact the fatigue life of the repair or retrofit more than the diameter of the hole. Even the quality of the drill bit used can have an effect on the fatigue life. Brown et al. (2007) found that holes drilled with dull bits often resulted in a fatigue life similar to punched holes, or about Category C. They reference other research which reportedly has shown that these differences can drop the fatigue resistance of the hole to as low as Category D. Figure 3-6 shows an example of the edge conditions following drilling and before sanding. The cutting process naturally leaves rough edges and burrs that can generate stress concentrations that over time can become initiation sites for fatigue cracks. A recommended sanding procedure includes using an 80 to 100 grit flap wheel with an angle grinder around the outer edges of the hole on both sides of the plate. Then the inside of the hole should be sanded using an 80 to 100 grit flap wheel with a die grinder. The result is a smooth-to-the-touch surface free of burrs and rough and gouged edges while being careful not to burnish the surface, as discussed for painting in Section 3.2. Also, sometimes when drilling begins, the magnetic base may shift as the annular cutter digs into the plate. This can happen when the magnetic connection to the plate is insufficient due to a number of possible reasons, such as excessive moisture, oil, or corrosion; plate curvature; or on very thin plates. When this happens, gouges can result that are capable of initiating

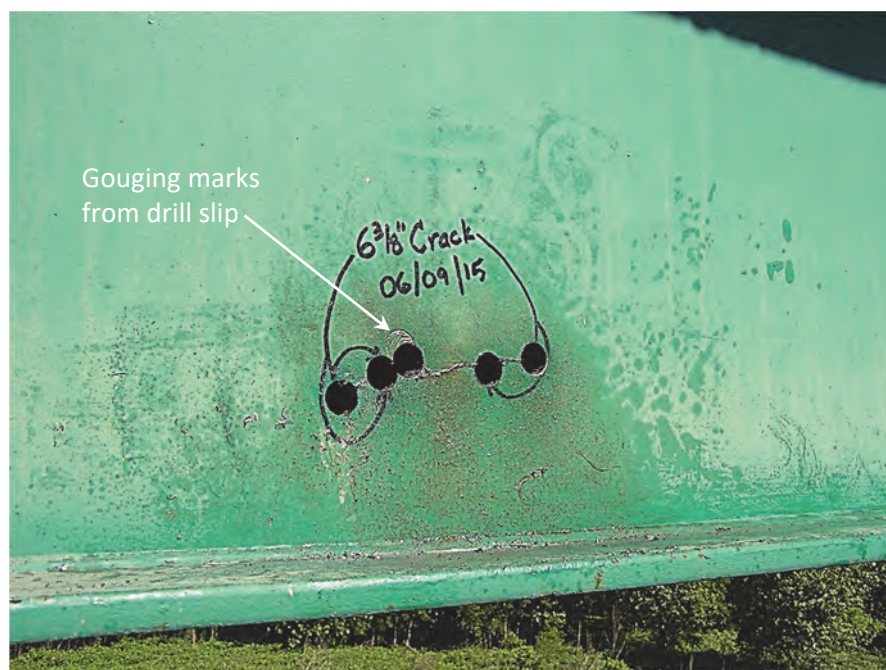


Figure 3-5. Example of Chasing the Crack Tip with Repeated Drilling

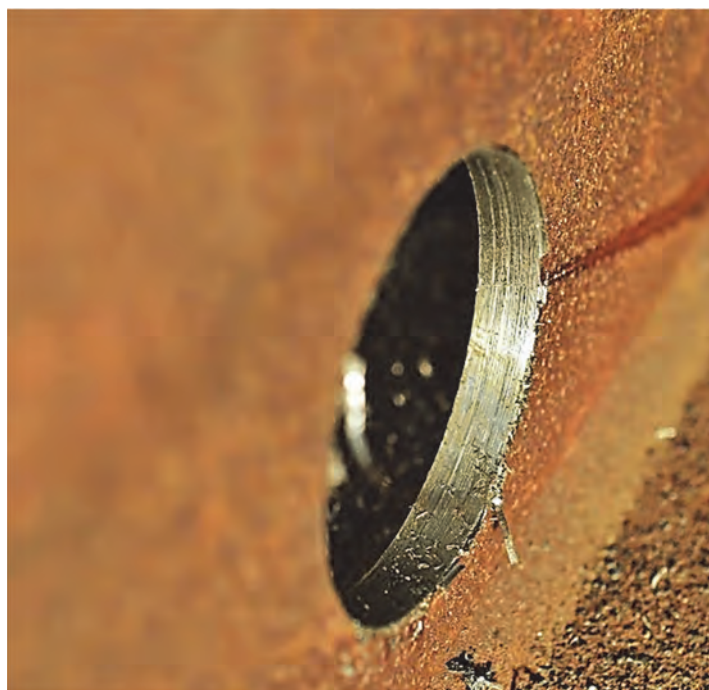


Figure 3-6. Drilled Hole Showing Rough Edges and Burrs

cracks over time. This condition can be observed in Figure 3-5 above the third drilled hole from the left. The gouges should be removed using the surface grinding techniques discussed in Section 3.4.

A final general consideration for box girder bridges is to plug the drilled hole with a bolt, wood, metal screen, or rubber stopper. This will help to prevent birds or other animals from nesting inside the structure, which could lead to accelerated corrosion pockets where fecal matter or nesting materials accumulate (Dexter & Ocel, 2013).

3.3.2—Procedural Guidelines for Hole Drilling to Arrest Cracks

The following guideline outlines proper procedure for repairs and retrofits utilizing drilled holes for crack arrest.

1. Locate the crack tip using enhanced inspection techniques, such as MT or PT. Figure 3-7 shows an MT inspection being performed where the crack is highlighted by the red magnetic particle dust. Inspect both sides of the plate to ensure the tip is not missed.
2. Position the magnetic-based drill as seen in scenario C of Figure 3-4. *Caution: Tie off the mag-based drill to the girder using sturdy clamps and chain/rope. In the event of power loss, this will help prevent injury or damage to equipment.*
3. Drill the hole. *Note: Carbide tip annular cutters should be used when the cut penetrates welds or heat-affected zones. This is because high-speed steel (HSS) cutters will quickly become dulled by the hardened material. HSS annular cutters can be used successfully in base metal applications away from welds.*
4. Check the drilled plate and removed core for evidence of the crack tip, as discussed in Section 3.3.1. Figure 3-8 shows the back side of a plate as the annular cutter breaks the surface. Proper placement of the drilled hole relative to the crack tip is evident.
5. Following removal of the core, reinspect the edge of the drilled hole using MT or PT to ensure the crack tip does not remain.
6. Sand the edges of the drilled hole using an 80 to 100 grit flap wheel with an angle grinder on the exterior edges and an 80 to 100 grit flap wheel with a die grinder on the interior edges. An example of a polished drilled hole is shown in Figure 3-9.
7. Paint the repair area using the techniques provided in Section 3.2.



Figure 3-7. Magnetic Particle Test Performed Before Drilling Crack Arrest Hole in Order to Locate the Crack Tip

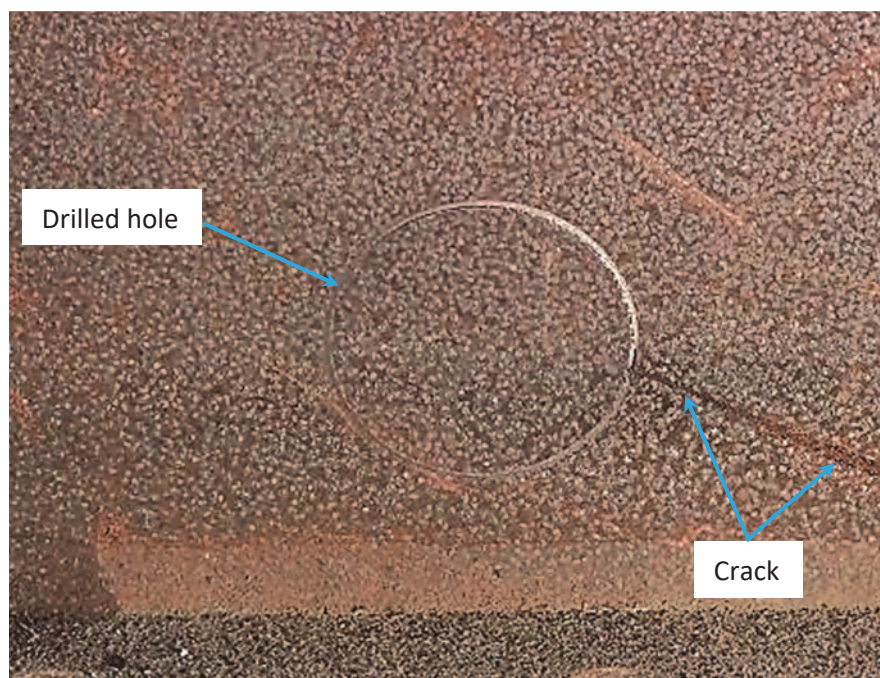


Figure 3-8. Shows an Annular Cutter Breaking the Surface on the Opposite Side of the Plate, Positioned Correctly to Intercept the Crack Tip

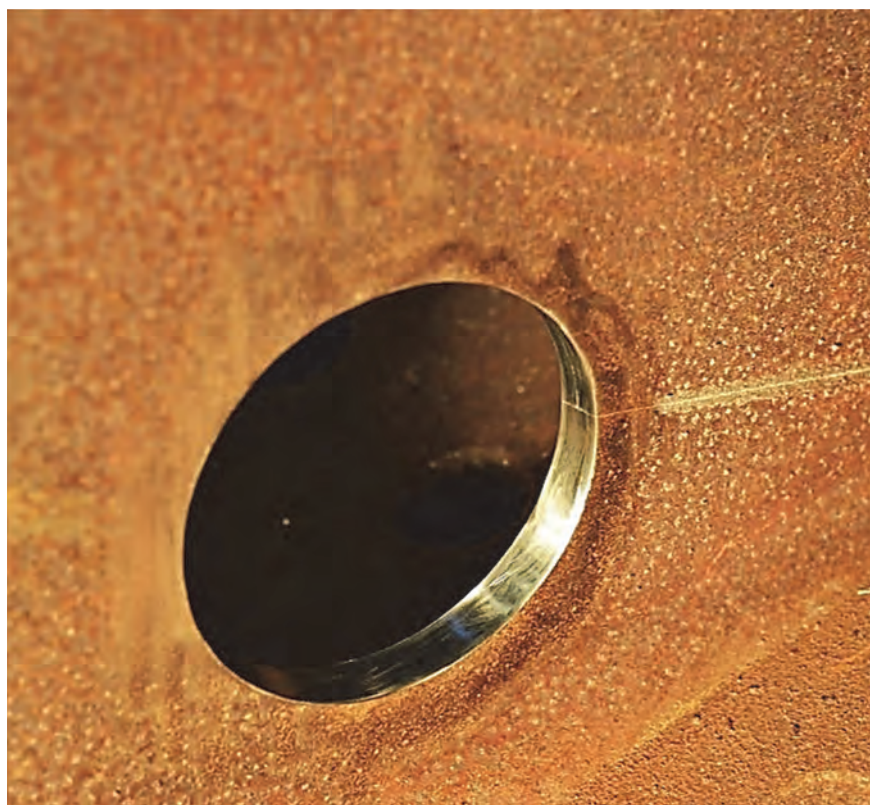


Figure 3-9. Example of Finished Crack Arrest Hole Prior to Painting

3.4—SURFACE GRINDING TECHNIQUES

Surface grinding techniques can be used for removing shallow defects, improving geometry to provide smooth stress flow thereby reducing stress concentrations, and repairing nicks and gouges. These types of defects, if left unrepaired, can become initiation sites for fatigue cracks, effectively reducing the fatigue resistance of the member or detail. Typical causes of these surface discontinuities include vehicle impacts, flame cutting, grinders, pneumatic hammers (e.g., peening or rivet busters), weld spatter, arc strikes, lifting chains, and other maintenance operations (Fish et al., 2015). Surface grinding is only recommended for the following types of repairs:

- Notches, nicks, or gouges
- Shallow surface crack removal (generally $\leq 3/16$ inch deep)
- Removal of tack welds and other extraneous welds
- Finish work on drilled holes

There are two main types of handheld grinders, angle and die. Grinding wheels for the angle grinder typically come in either 4.5- or 9-inch diameters and a wide range of types and grits. Also called a disc grinder, the angle grinder can also be used to quickly cut steel when using a cutting wheel. An example of an angle grinder is shown in Figure 3-10 paired with a flap wheel for sanding. A typical die grinder is shown in Figure 3-11 fitted with a rotary burr bit for rough grinding. The die grinder typically uses much smaller abrasive wheels and cutting wheels than the angle grinder, as well as carbide rotary burrs, allowing for access to smaller spaces. The angle grinder, although more difficult to get into tight spaces, has the advantage of faster material removal.



Figure 3-10. Typical Angle Grinder with Flap Wheel Figure 3-11. Typical Die Grinder with Burr Bit

3.4.1—General Considerations for Surface Grinding

Although grinding is effective and requires little training or skill, careless handling can cause new problems or leave the original problem unresolved. Discontinuities must be completely removed in order to be effectively repaired, but without removing too much material or losing control of tools and leaving new nicks and gouges. The following provides some general considerations for surface grinding:

- Although visual inspection can be very effective for locating and evaluating surface defects such as notches or gouges, enhanced NDT, such as PT or MT, is highly recommended to help identify the limits of a crack before beginning grinding.

- Once grinding has been completed, NDT should again be used to ensure the crack has been completely removed. Grinding can “smear” a crack, or partially fill the visual opening with metal shavings, effectively hiding the crack from view. Note that if the crack is smeared enough, PT may also be made ineffective.
- Surface discontinuities are generally simple to repair and should not require engineering analysis. An exception would be where after removing $1/8$ to $3/16$ inch of material, the defect has not yet been completely removed. In this case, it may be prudent to perform engineering analysis before removing additional material. Ensure a gradual stress flow is left by the grinding operation; see Section 4.3 for additional guidance.
- If a crack cannot be completely removed by shallow grinding, another repair strategy should be considered, such as a bolted splice (in combination with grinding or hole drilling). See Section 4.6.2.3 for guidance on bolted splice repairs.

3.4.2—Procedural Guidelines for Surface Grinding

The following guideline delineates proper procedure for repairs and retrofits utilizing surface grinding.

1. Locate the discontinuity and evaluate with enhanced inspection techniques, such as PT or MT, as needed.
2. Perform coarse grit grinding for bulk of material removal.
 - a. If a lot of material needs to be removed, such as the remainder of a truncated stiffener plate, consider using a 24 grit grinding wheel with an angle grinder for fastest material removal rates.
 - b. 40 to 60 grit flap wheels with the angle grinder will provide excellent material removal rates that can be well controlled for applications needing less material removal.
 - c. A die grinder with carbide rotary burr is recommended for areas of restricted access.
3. Complete fine grit grinding to create a smooth and polished finish.
 - a. Grind in an orientation that is parallel with primary stresses in the member. Typically, grinding wheels will rotate clockwise. Another way to tell is to note the direction of the sparks coming off the wheel; make sure they are parallel with primary stress flow. This will help reduce the risk of any grinding striations becoming future initiation sites for fatigue cracks.
 - b. 80 to 100 grit flap wheels on angle and die grinders are recommended for this step.
4. Paint the repair area using the techniques provided in Section 3.2.

3.5—WELD TOE IMPROVEMENT TECHNIQUES

Several weld toe improvement techniques exist that can be used as standalone repair or retrofit strategies, or can be combined with other methods for a broader approach. This section presents considerations and procedures for weld toe grinding and impact treatments.

3.5.1—Weld Toe Grinding

There are several reasons that fatigue cracks initiate at weld toes, including slag intrusions that create initiation points; high residual tensile stresses; and the geometric discontinuity they cause for stresses flowing through the detail, causing increased stress concentration. The basic concept behind grinding to reshape the weld toe is to possibly remove some slag intrusions, but primarily to change the weld toe geometry, thereby reducing stress concentration. There are mixed results, however, reported in literature for using this technique to improve weld toe fatigue resistance (Fisher et al., 1979; Gregory et al., 1989; Rutherford & Polezhayeva, 2006; Dexter & Ocel, 2013). Due to the inconsistency of performance and the extensive labor required to implement this retrofit on steel bridges, it is not recommended as a primary strategy for weld toe fatigue resistance improvement. However, if elected as a supplementary method for weld toe improvement, the following techniques are recommended.