Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges

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FOREWORD

This manual was the result of a consensus workshop held in August 2002 with world leaders in bridge fatigue and fracture. The purpose of the manual was to serve as a repository of common fatigue situations encountered with steel bridges and document the possibilities of repair and retrofitting fatigue cracks. The manual development was funded through a pooled fund mechanism under Solicitation 102. The pooled fund participants were the states of IA, ID, IL, IN, LA, MN, NY, PA, VA , WI, and the Transportation Association of Canada.

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The development of this manual was the vision of the late Professor Robert Dexter. The manual was based on a review of the published literature as well as the experience of participants of a "Fatigue Workshop" held at Lehigh University August 14-15, 2002. The authors would like to personally thank the participants of the Fatigue Workshop for all the information they donated, as this manual would not be possible without it. Thanks must also be given to Bill Wright of the Federal Highway Administration for the funding of the project, technical review by Hussam Mahmoud, Rob Connor, John Fisher, Mike Koob, Joey Hartmann, Brian Kozy, and Tom Everett as well as Moira Coughlin for her CAD figures. Unfortunately, Professor Dexter pasted away on November 16, 2004 from acute leukemia, prior to completion of this manual. He will always be remembered as a great contributor to fatigue and fracture, not only in the bridge industry, but shipbuilding, railroad, sign and signal, and pressure vessels. He will never be forgotten and will always missed.

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Not Pictured Petro Albrecht, Robert Connor, and Don Flemming

CHAPTER 1. INTRODUCTION

This manual is a synthesis of published literature as well as the collective experience gathered from participants of a workshop held at Lehigh University August 14-15, 2002 [herein referred to as the Fatigue Workshop]. It presents what are the consensus best practices for repair and retrofit of fatigue cracks in steel bridges as of 2010. The manual is also a guide for owners and consulting engineers to use for design and detailing of repairs and retrofits for fatigue cracks. It should be used in conjunction with existing specifications, codes, and engineering judgment.(1-4) In addition to designing and executing the repairs and/or retrofits, there are numerous other steps involved in the inspection and assessment of a cracked bridge member. Detailed discussion of inspection and fatigue and fracture assessment are beyond the scope of this manual and, therefore, are only briefly discussed below with reference to other documents.

FATIGUE

When cracks are discovered in bridge elements (members or connections) in service, fatigue is usually the cause.(5) Fatigue is the formation of a crack due to cyclic service loads.(6,7) Figure 1 shows the surface of a common fatigue crack. This crack originated at the toe of a repair butt weld in the flange of a welded built-up I-girder (hence the access hole cut into the web). The appearance of a fatigue crack is typically smooth and silky, as shown in Figure 1. Fatigue surfaces usually show distinct striations outlining the shape of the crack as it grew (as seen in Figure 1) and these point back to the origination point of the crack

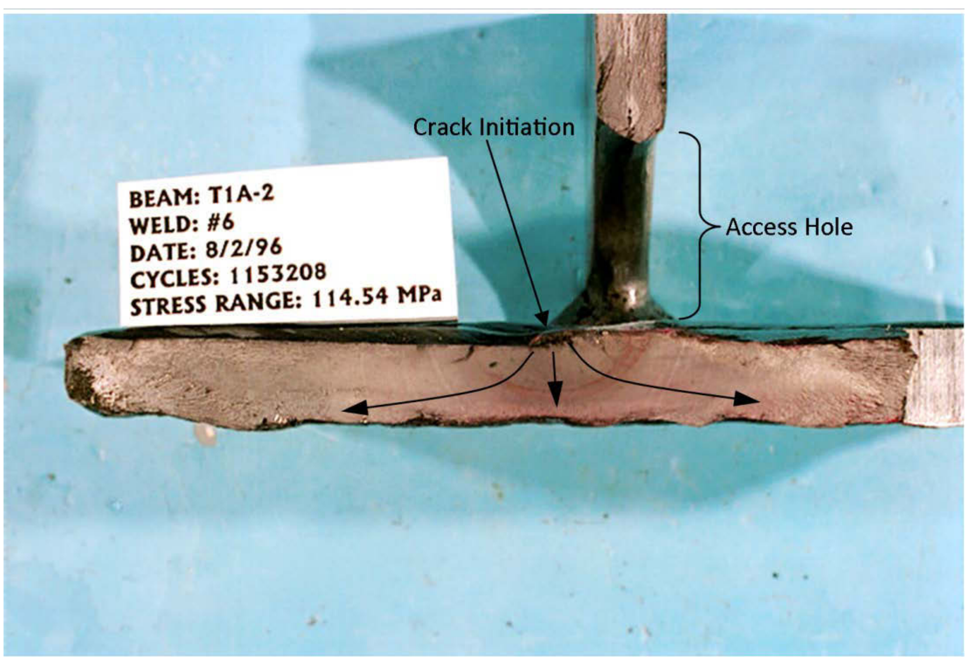


Figure 1. Photo. Typical fatigue crack across a repaired flange butt weld in a welded I-girder test specimen. The arrows show the direction crack growth and the fatigue striations show how the crack grew elliptically through the flange. Photo courtesy of US Coast Guard.(8)

FRACTURE

Fracture may be defined as rupture in tension or rapid extension of a crack, leading to gross deformation, loss of function or serviceability, or complete separation of the component.[9,10,11] Fatigue and fracture are different, albeit related, limit states. Fatigue may or may not lead to fracture. In addition, fracture may or may not be the result of fatigue. It is urgent to assess the potential for fracture whenever there is a crack in any tension element (tension member, or tension flange or portion of the web of a flexural member). However, the presence of fatigue cracks does not necessarily mean that the structure is unsafe.[5,9] In fact, in some redundant structures, a fatigue crack may stop propagating with no intervention at all due to redistribution of stresses.[12] Usually, however, a fatigue crack will propagate and eventually cause a fracture if not mitigated.[5,9,10]

If a bridge system is not redundant and cannot develop alternative load paths under loss of member continuity, then fracture of a particular member could lead to partial or total collapse of the structure. Such a member is defined as a Fracture Critical Member (FCM). The development of a fatigue crack in a FCM should warrant immediate action, such as closure of one or more lanes, posting the bridge for cars only, or complete closure of the bridge until repairs can be made.

Most modern bridges are redundant and can develop alternative load paths.[9] If a member with a crack is not fracture critical, the consequences of fracture are less severe; therefore keeping the bridge open may be justified. Although unlikely to cause collapse, any fracture is undesirable, and a fracture assessment must be performed to assure the possibility of fracture at service loads is acceptably small in any tension element. If the fracture assessment indicates that the potential for fracture is unacceptably high, it may be necessary to close lanes, post the bridge, or to completely close the bridge.

A fracture assessment is made using fracture mechanics principles. (See references 5, 10, 11, 13, and 14.) In the absence of other established procedures, it is recommended that a fracture assessment be conducted according to British Standard, BS 7910: 1999 "Guide on methods for assessing the acceptability of flaws in metallic structures."[13] There is presently no comparable United States standard; however, BS 7910 is widely used by some U.S. industries (primarily the oil and gas industry).

A fracture assessment requires knowledge of the fracture toughness of the steel and/or weld metal, which may be estimated from the Charpy energy or CVN test results.[10, 14] Unlike fatigue, the susceptibility to fracture is strongly dependent on the type of material and even the particular heat of steel or lot of weld metal. (See references 10, 11, 15, and 16.) CVN results may be obtained from the mill reports for the steel and from the certifications for the weld metal, if records were retained. Minimum Charpy requirements were first included in the ASTM A709 specification for bridge steels in 1974, therefore the CVN properties for the steel plate and weld metal may be assumed to be at least equal to the minimum specified values for bridges designed after 1974, if records are not available.[17] Care must be taken to not consider the 1974 date firm, and some work may be required to ensure that the design specifications reference the 1974 standard as there maybe instances where design started prior to 1974, but construction occurred well after 1974. For older structures, or for cases when the assumed minimum values are not sufficient, it may be necessary to core samples of the steel, then machine and test Charpy specimens. A minimum of three specimens should be tested from each plate or structural shape in accordance with the appropriate ASTM specification. Fracture assessment will not be discussed further since it is documented elsewhere and is not the emphasis of this manual.

If a detail or material are particularly susceptible, a fracture will usually occur as the loads are applied for the first time during or soon after construction. An example is the fracture that occurred when the Caltrans Workers Memorial Bridge (previously known as the Bryte Bend Bridge in Sacramento, CA) was under construction in 1970. The fracture, shown in Figure 2, was attributed to use of low-toughness A514 steel at a reentrant corner detail (between a transverse bracing member and the primary tension flange of the tub girder) in combination with a preexisting weld toe crack resulting from the fabrication and shipping processes.[18] A514 steel, sometimes marketed under the name of T1 steel, is quenched and tempered with a minimum specified yield stress (MSYS) of 100 ksi (690 MPa).

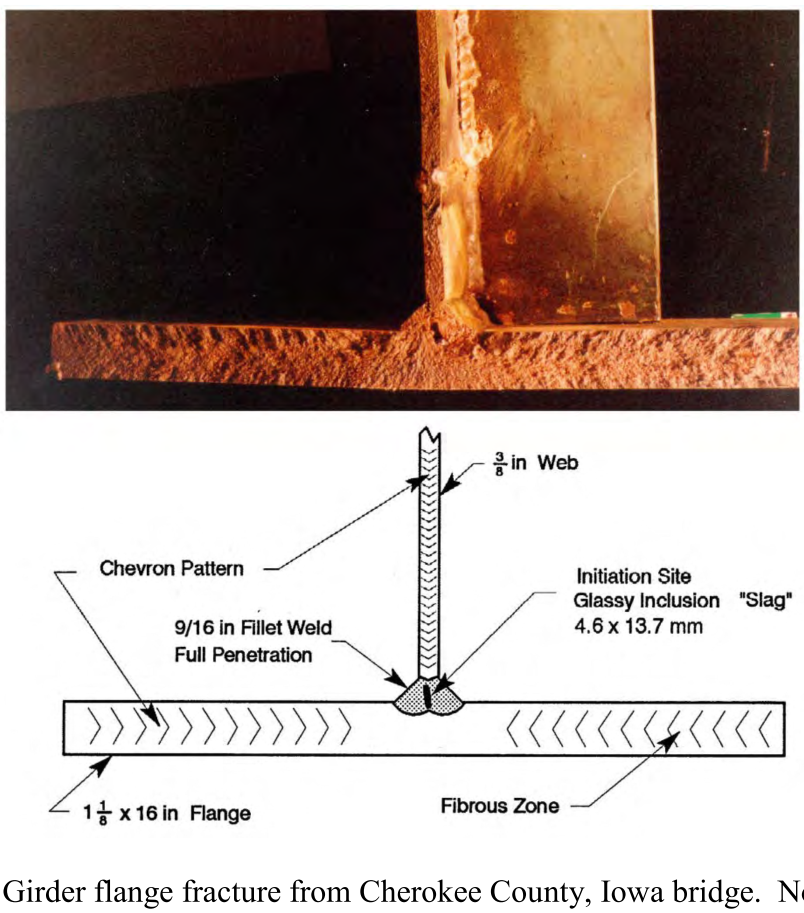


Figure 2. Photo. Girder flange fracture from Cherokee County, Iowa bridge. Note characteristic appearance of chevron marks in flange pointing back to fracture initiation.

It is also possible that fracture can occur directly without previous fatigue crack growth after years of service as was the case in the Hoan Bridge.[19] Brittle fracture can occur at high constraint details with little or no warning of impending fracture. Constraints develop because of welding vertical and horizontal attachments to girder webs, even when the web plate is thin. Intersecting welds create a triaxial stress state where the material is unable to yield through its thickness resulting in localized 3-dimensional stresses being generated that are much higher than the material's yield. The Hoan Bridge failure was attributed to brittle fracture without the presence of fatigue cracks and therefore is not discussed further in this manual but is discussed in Section 4.1.8 of the Federal Highway Administration (FHWA) Fracture Critical Inspection Techniques for Steel Bridges participant workbook.[20]

Sub-critical crack propagation in bridge elements may also occur from stress-corrosion cracking (SCC), which can be both a fatigue and fracture phenomena. Primarily, this is a concern only for very high-strength steels, e.g. steels with yield strengths much greater than 100 ksi (690 MPa). Stress corrosion cracking involves electrochemical dissolution of metal along active sites under the influence of tensile stress.[10] As hydrogen is liberated in the process, this failure mechanism may also involve hydrogen cracking. SCC has occurred in prestressing cables, A490 bolts and A514 plate that were produced out of specification.[21, 22, 23] This manual does not address assessment, repair or retrofitting for SCC.

FRACTURE CONTROL PLAN

In 1967, the Point Pleasant Bridge over the Ohio River in West Virginia, commonly referred to as the Silver Bridge, collapsed due to brittle fracture of one of the non-redundant eyebars supporting the main span suspension system.[24] This eyebar was truly a fracture-critical member in that the loss of this one steel tension element caused the collapse of the bridge. SCC may have caused the initial flaw in the eyebar, which was not typical bridge steel but rather heat-treated AISI 1060 steel.[25] Because SCC should not occur in ordinary bridge steels produced in accordance with the current specifications, it is not discussed further in this manual.

In response to the collapse of the Point Pleasant Bridge, a fracture control plan for bridges was implemented in 1975 to provide a higher level of safety for non-redundant structures.[10, 26] Note, the minimum Charpy impact requirements were adopted in 1974, one year before the fracture control plan. There are two main aspects to the current AASHTO/AWS fracture control plan: 1) control of cracks (through detailing and enhanced fabrication inspection protocols) and; 2) specification of minimum material properties for steel and weld metal measured by CVN testing. Both aspects are integral to providing an acceptably low probability of brittle fracture.

The AASHTO/AWS Bridge Welding Code sets standards and procedures for fabrication that minimize the potential for weld discontinuities in new construction.[2] The provisions of the Bridge Welding Code should keep discontinuity size sufficiently small to prevent sudden fracture and assure that the fatigue crack growth threshold is not exceeded under service loads. A higher standard is set for fabrication procedures of fracture-critical members.

INSPECTION

Periodic in-service inspection provides another opportunity to assure safety by detecting cracks before they grow to a critical size.[3,27-29] The Federal Highway Administration has mandated a twenty-four month inspection interval for all highway bridges located on public roads which is intended to detect problems before they become critical to structural performance. Bridges with FCMs and/or those with a history of fatigue problems are sometimes inspected more often if they are determined to be at greater risk for fracture. By federal regulation, bridges with FCMs receive "hands-on" inspection to assure that inspectors are in close proximity to effectively identify problems. Since its inception, the inspection program has proven a successful means of ensuring safety in steel bridges.

Because of the repetition of details in a bridge, when one crack is found it is likely that similar details may also exhibit similar conditions. Therefore, the first and most urgent step in planning repairs and retrofits is to conduct a special in-depth inspection to thoroughly inspect the bridge for other cracks. This inspection is typically done visually, but sometimes integrates non-destructive evaluation (NDE) technologies such as magnetic-particle testing, dye-penetrant testing, or other techniques, especially for fracture-critical members. The focus of inspection for additional fatigue cracks should be on elements in tension similar to the one that cracked, with priority given to those details with high live-load stress ranges and/or those that are on FCMs.[3,27,29]

Elements for which the applied stress range remains in compression need not be inspected closely for fatigue cracks, since crack extension away from the detail or complete fracture is typically not possible. Due to tensile residual stresses from welding, a crack can still occur in a structural element that undergoes cyclic loading even if the applied stress range remains in compression. However, these cracks will arrest as they grow away from the welds because the crack tip will enter a compressive stress zone.[9]

SELECTING A REPAIR AND RETROFIT STRATEGY

After inspection is completed and any fracture assessment is conducted, it is necessary to determine the type and cause of any cracks discovered.[5] In some cases this may require extraction of the crack by taking a drilled core to enable examination of the crack surfaces. It is possible that a newly discovered crack will have existed since the fabrication of the bridge, in which case it may be due to any of a number of causes, such as hydrogen cracking of welds. If a fabrication defect has been exposed to typical service temperatures and loading and has not yet propagated by fatigue or caused a fracture, it may be safe to delay repair of the defect and to merely monitor it to ensure that it does not begin to propagate, provided an analysis can support that decision.

For common redundant bridges, the best choice is often not to repair a crack but rather monitor the crack. This is especially true for cracks caused by out-of-plane distortion at secondary elements. However, "monitoring" must be more than simple visual inspection. It must include in-depth inspection techniques, which can reliably identify the location of crack tips, and growth over time. Even when it is determined that a crack should be repaired, it is not always necessary to perform the repair immediately. Fracture mechanics principles can be used to estimate the time that it will take before a sub-critical crack grows to the critical crack size that can cause fracture. However, determining a critical crack size requires knowledge of a material's toughness. (See references 5, 10, 11, 13, and 14.) Nevertheless, fatigue cracks are usually repaired within months of their discovery.(5)

Chapter 3 presents commonly used repair and retrofit techniques for fatigue-prone details. Some of the same treatments or modifications of a structural detail are used in both repairs and retrofits. The distinction can be made that a repair is intended to arrest the propagation of an existing fatigue crack, while a retrofit is intended to upgrade the fatigue resistance and prevent the future occurrence of fatigue cracking. Retrofits are usually carried out at details similar to those that have previously caused cracking.(5)

Improved fatigue design specifications for steel bridges were implemented in 1975 largely as a result of research performed for the National Cooperative Highway Research Program (NCHRP) at Lehigh University.(30, 31) Bridges designed prior to 1975 were not designed and detailed adequately for fatigue and often included coverplated beams, web attachments, and other details with low fatigue resistance. Many such bridges with fatigue-prone details were built into the late 1970s prior to widespread implementation. In addition, weld quality has significantly improved as a result of the fracture control plan requirements. Thus, inspection and maintenance is especially critical for bridges designed before 1975 due to the poor details often used and the possibility of poor weld quality. In addition, many older structures are experiencing higher traffic volumes and truck weights than they were originally designed to accommodate increasing the number of fatigue cycles and the stress range at fatigue-prone details and often resulting in fatigue cracks. Repair of these older bridges for fatigue cracks due to primary loading (and not from distortion) is covered in Chapter 6 of this manual.

Another very significant fatigue problem for bridges designed before 1985 is the attachment of connection plates. Diaphragms and cross-frames are used to brace or stiffen primary girders of a bridge during construction before the concrete slab has hardened and in negative moment regions of continuous spans. They also help to distribute lateral load (such as wind or earthquake load) among girders. These diaphragms are connected to the girders by connection plates, which are typically welded to girder webs and flanges in modern designs.

Prior to 1985, connection plates were not welded to the tension flange of girders to avoid a perceived fracture prone detail. Although prohibition of flange connection may have been good practice in the early days of welding (1940's), there is certainly reduced concern about a transverse fillet weld on the tension flange as a result of modern steel fabrication processes. This prohibition also had undesirable consequences. By not attaching a connection plate to the tension flange, a small, unstiffened segment of web, which is commonly referred to as the "web gap", was created. Web gaps are often subjected to out-of-plane distortion when there is differential deflection between adjacent girders, leading to the creation of fatigue cracks.[32, 33] In fact, distortion-induced cracking is estimated to be the cause of over 90% of all fatigue cracking.[34] Skewed and curved bridges tend to be more prone to this problem due to an exacerbated differential deflection between girders and the inherently larger crossframe forces. The potential for web gap cracking is increased in negative moment regions where the tension flange is fully restrained by the concrete slab. Attachments for internal diaphragms in box girders are also known to be susceptible. Repair and retrofit of this type of cracking is presented in Chapter 4.

Since 1985, the AASHTO bridge design specifications have required a positive attachment between the connection plate and both flanges of the girder, which has eliminated the out-of-plane distortion problem. As a result, bridges designed after 1985 exhibit very few problems from out-of-plane distortion. Unfortunately, a large percentage of bridges were designed before 1985 and are potentially susceptible to web gap cracking. For example, 85% of steel bridges in the State of Minnesota were built prior to 1986.[35]

Although it is intended for this manual to be completely read and understood before specifying any retrofit and/or repairs, the flowchart shown in Figure 3 will help the decision making process of repair and retrofit for common redundant bridge members.

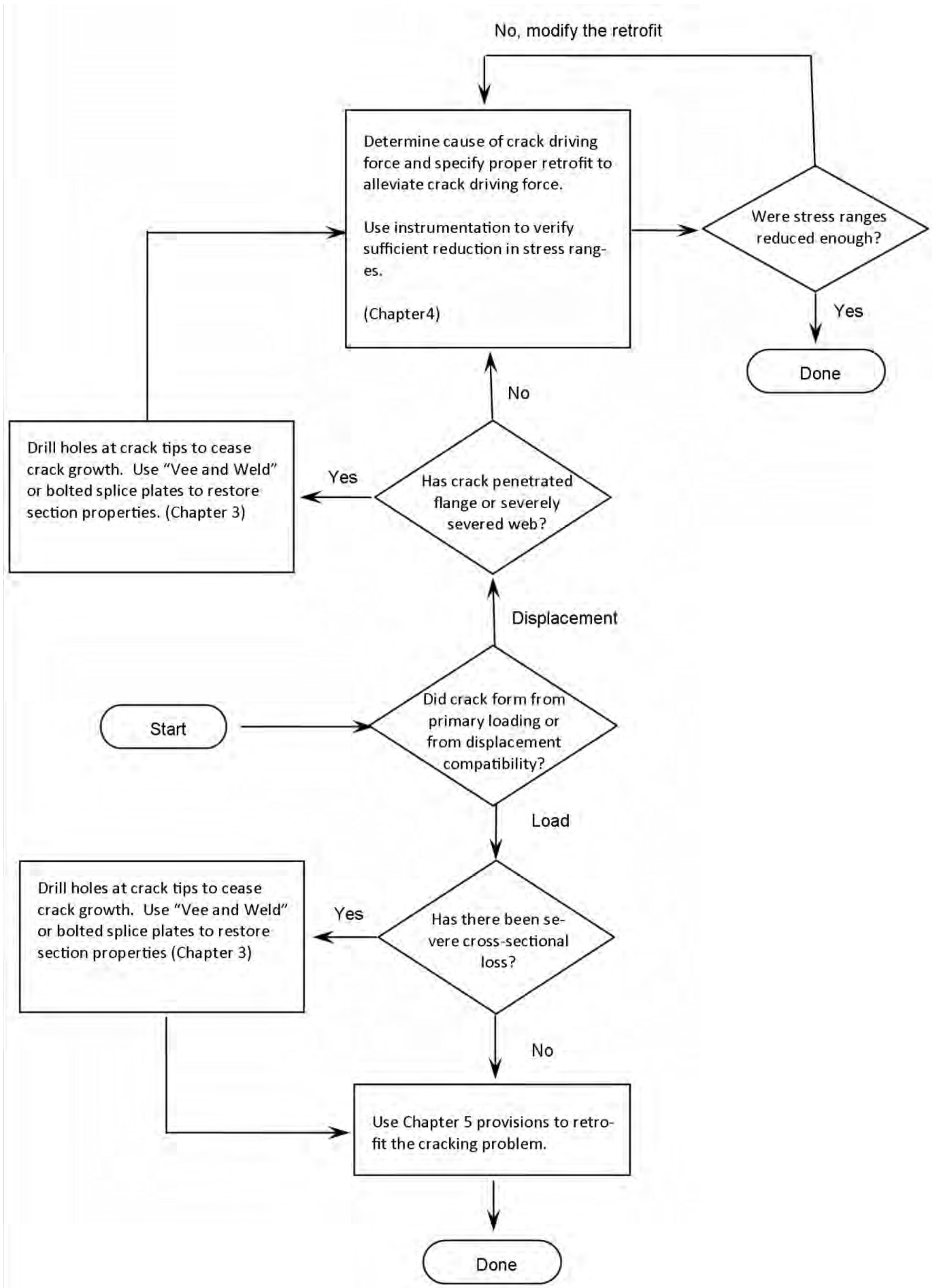


Figure 3. Flowchart. General flowchart for fatigue retrofits

CHAPTER 2. FATIGUE ASSESSMENT

In the context of repair and retrofit, assessment if often necessary to explain the presence of cracking, and/or to predict the remaining life until cracking may develop. The approach to evaluating and assessing bridge members for fatigue is empirical and is based on tests of full-scale specimens of members or components with welded, bolted or other details. Such tests indicate that:

- Strength and type of steel have only a negligible effect on the fatigue resistance expected for a particular detail. (See references 5, 30, 31, and 36.)
- Welding process (SMAW, GMAW, FCAW, SAW, and ESW) typically does not have an effect on the fatigue resistance.[37]
- The primary effect of fatigue loading can be accounted for in the live-load stress range. Or in other words, the dead load or locked-in stress levels are not significant. (See references 5, 30, 31, and 36).

The insignificance of steel type and weld metal on fatigue resistance greatly simplifies the development of fatigue design rules since it eliminates the need to generate data for every type of structural steel. The reason that the dead load has little effect on fatigue resistance is that, locally, very high residual tensile stresses are created when a weld cools. Irrespective of the dead load stress level, the maximum stress at weld toes due to the combination of live load, dead load and residual stress is always at or beyond the material's proportional limit in tension. Therefore, the maximum and minimum stress levels that define the true stress range at weld toes is mostly unchanged by the magnitude of dead load stress.

Experimental tests to determine the fatigue resistance of welded details are usually performed at constant load amplitude, whereas the loading on bridges has variable amplitude. Results for testing under variable-amplitude loads have shown that Miner's rule can be used to convert a number of variable stress ranges into an effective constant-amplitude stress range which will cause the same fatigue damage per number of cycles.[38,39 40] The AASHTO specifications use load factors to account for the variable nature of the load spectrum for proper comparison to resistances determined experimentally.

Three possible analytical approaches, depending on the level of refinement in the stress analysis, can be used for assessment of the fatigue limit state of a detail; 1) nominal stress approach, 2) local stress approach, and 3) fracture mechanics. The difference in level of analysis is illustrated in Figure 4, which shows a plate under uniform stress with a longitudinal fillet-welded attachment. The nominal stress is the uniform stress away from the attachment that can be calculated using simple bending and axial load equations.

The plate stress becomes amplified closer to the attachment from two sources of stress concentration. First, there is the effect of global geometry of the connection, which produces an elevated stress referred to as the geometric stress concentration. The product of the nominal stress and the geometric stress concentration factor is the local structural stress or "hot-spot" stress.

Then there is a further stress concentration due to the local geometry of the weld toe; this is referred to as the local notch stress concentration. This local notch stress concentration is dependent on random parameters, such as weld profile and ripples in the weld toe, and typically cannot be accurately characterized through analysis. Fracture mechanics can be applied to analyze qualitative aspects of the problem; however, many assumptions are required to provide meaningful quantitative results.

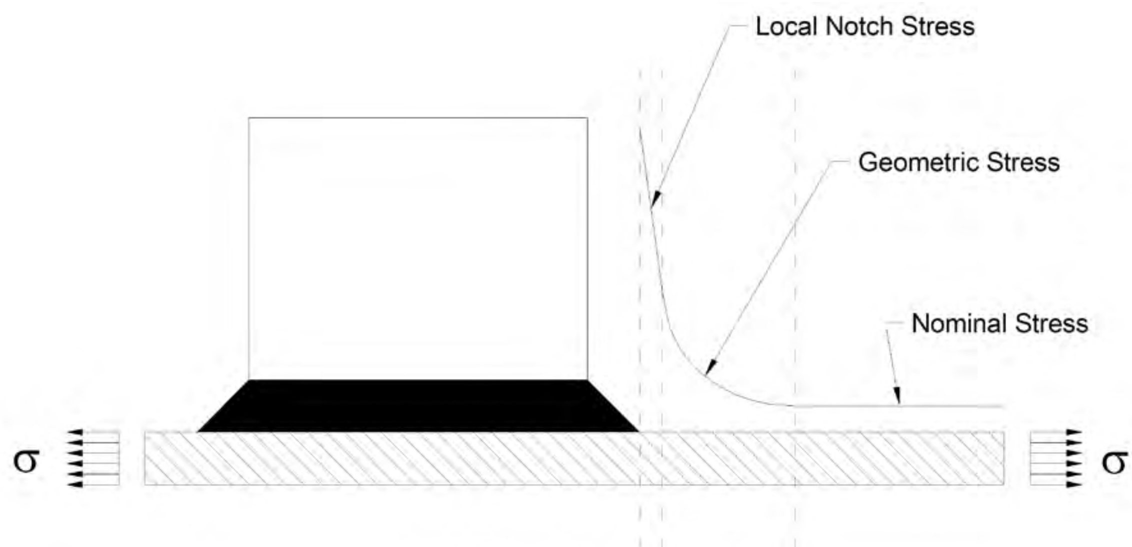


Figure 4. Illustration. Difference between stresses near a welded detail.

Consider also a uniform flat plate panel with a butt weld and weld reinforcement transverse to the nominal stress. In this case, there is a local notch stress concentration at the weld toe but no geometric stress concentration. However, if there were a tee-joint or misalignment of the plates, this would give rise to the geometric stress concentration. These three different levels of stress located in the area around a weld are associated with the three aforementioned types of fatigue analysis, respectively.

NOMINAL STRESS APPROACH

The nominal stress approach is the simplest way to assess the fatigue limit state at a detail and, as such, is included in design specifications from a variety of industries. This approach determines fatigue resistance and applied stress based on calculating a nominal stress near the detail being considered using simple equations for bending and axial load. The local stress concentrations that occur at the detail and at the notch are accounted for in the specified resistance. This design procedure relies on data obtained from full-scale fatigue tests of similar details.

The test data and resulting limits are presented in stress range (S) vs. number of cycles to failure (N) curves (referred to S-N curves). Failure is commonly defined as the development of a through-thickness crack. Because weld profiles, residual stresses, and weld discontinuities are highly variable from specimen to specimen, the fatigue data based on nominal stresses has a large amount of scatter, and the S-N curves used in design represent a lower bound of the data. Specifically, the S-N curve for design (plotted as a line in log-log space) is located two standard deviations below the mean of the data, by conducting the regression on the logarithm of the cycles to failure. This is equivalent to a 97.5 percent survival rate. This lower bound approach provides safety in the methodology.

Different S-N curves are used to evaluate the fatigue resistance of different details depending on their geometry. Each S-N curve represents a particular detail category. Details with similar fatigue strength are put in the same category. S-N curves are well developed for simple details such as longitudinal welds, transverse butt welds, cover plated beams, etc. Figure 5 shows the S-N curves from AASHTO Specifications, as well as American Welding Society (AWS), American Railway Engineers and Maintenance of Way Association (AREMA), and American Institute of Steel Construction (AISC) specifications. (See reference 1, 4, 41, and 43.) In the nominal stress approach, the effect of the geometric and local notch stress concentrations are included in the test data, and the designer does not need to account for them. Variations in the combined stress concentrations are reflected in the different detail categories.

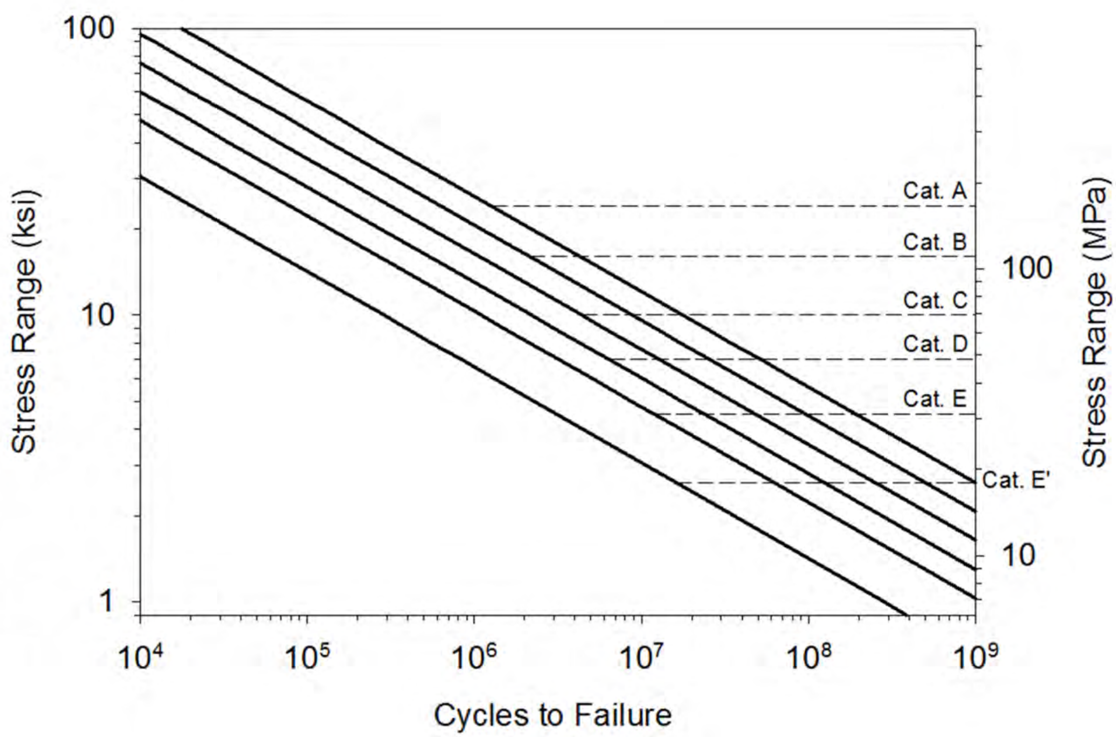


Figure 5. Graph. Nominal stress S-N curves used in AASHTO, AISC, AWS, and AREMA specifications.

In addition to the S-N curve, each detail category has a constant-amplitude fatigue limit (CAFL). The CAFL is a stress range below which no fatigue cracks are expected to occur in tests conducted with constant-amplitude loading. In the AASHTO LRFD Bridge Design Specifications, the CAFL is referred to as the fatigue threshold.[1] These limits are shown as the horizontal lines on the right side of the S-N curves in Figure 5. Depending on the detail category, the CAFL begins at 2 to 20 million cycles.

With the publishing of the 2010 5th Edition of the AASHTO LRFD Bridge Design Specifications, fatigue is addressed by two sets of design criteria, or limit states: the "Fatigue I" limit state and the "Fatigue II" limit state.[1] These two limit states represent the two distinct regimes of fatigue behavior. The Fatigue I limit state represents infinite fatigue life performance (i.e., the bridge can safely carry an infinite number of truck-load induced fatigue cycles). In other words, if this limit state is satisfied the bridge will not experience cracking during its 75-year design life no matter how many stress cycles are applied. This is typically referred to "infinite life." The Fatigue II limit state represents finite fatigue life performance. In other words, if the Fatigue I limit state is not satisfied and cracking is expected at some time in the future, the Fatigue II limit state establishes a limit on stress as a function of number of design stress cycles.

If the average daily truck traffic (ADTT) is 750 or more, an excess of 20 million cycles will occur in the 75-year lifetime of a bridge. Therefore, most bridges will be designed using the infinite-life approach to keep most stress ranges below the CAFL. Full-scale variable-amplitude fatigue tests show that if less than 0.01 percent of the stress ranges in a distribution are above the CAFL, fatigue cracking will not occur (infinite life).[39] Consequently, the stress range associated with an exceedance probability of 0.01 percent is referred to as the maximum fatigue stress range.

The infinite-life approach is particularly useful if measured stress ranges are obtained for a bridge as part of the assessment of fatigue cracks and their repairs. The stress ranges must be measured for a long enough period such that a statistically meaningful sample of the traffic is obtained. If there are seasonal variations in the traffic, this must be accounted for. Otherwise, stress ranges must be measured in the season with the heaviest loads. A large enough sample must be acquired to accurately determine the maximum stress range with 0.01% rate of exceedance. Studies have found that the stress range histograms usually converge after two to four weeks of data collection, though in certain circumstances one week may be sufficient.[34] The data collection period is also a function of cost which may ultimately determine how long monitoring is conducted. Cycles from pick-up trucks and cars should be ignored. Typically, as demonstrated by full-scale, variable-amplitude fatigue tests,[39] stress ranges 0.25-0.33 times the CAFL are ignored because commuter automobile traffic is inconsequential to fatigue life. Ignoring these large numbers of smaller stress ranges yields a conservative estimate of the effective stress range calculation.

The analysis using the infinite-life approach is quite simple. Cracking can be explained if the measured maximum stress range exceeds the CAFL of the detail examined. The repair or retrofit will be effective if the CAFL for the resulting detail exceeds the measured maximum stress range.

In the AASHTO LRFD Specifications, the effective fatigue load is equal to 75% of the design truck portion of the HL-93 live load with a constant rear axle spacing of 30.0 ft., yielding a gross vehicle weight (GVW) of 54 kips (240 kN).[1] The effective fatigue load produces the equivalent fatigue damage of the actual variable amplitude live load spectrum. When the fatigue load is applied in a structural analysis, the resulting stress range is the effective stress range, which can then be used in a fatigue analysis as if it were a constant-amplitude stress range. The maximum fatigue load is 150% of the design truck, yielding a GVW = 108 kips (480 kN).

Weigh-in-motion (WIM) data indicate that the maximum fatigue loading implied by the AASHTO LRFD provisions clearly has a much higher frequency of exceedance than 0.01%. The apparent inconsistency in the exceedance level of the maximum loading was intentional and was a result of "calibrating" the LRFD bridge specifications to give fatigue design requirements that are similar to those of preceding specifications. The theoretically low exceedance level of the maximum loading implied by the LRFD code was used because it is known that other aspects of the design process are conservative, such as the distribution factors used in line-girder analysis.

While measured stress ranges from actual traffic are usually preferred, a bridge may be evaluated for fatigue and for repair using analysis only. If line girder analysis is to be used for the evaluation of fatigue cracking or repairs, then a maximum loading of 150% of the truck portion of the HL-93 live load is appropriate. However, if a more refined methodology such as grillage analysis is used, a more realistic maximum loading should be selected. Based on a review of nationwide WIM data, a more realistic maximum loading is better approximated by 225% of the truck portion of the HL-93 live load, or a truck weighing approximately 162 kips (720 kN).[44,45] The AASHTO Manual for Bridge Evaluation (MBE) provides methods to quantify “minimum,” “mean,” and “evaluation” fatigue life.[3] Also, methods to account for future growth and uncertainties in analytical methods are provided.

LOCAL STRESS APPROACH

The local (or hot-spot) stress approach is similar to the nominal stress approach, but the S-N curves in this approach are based on the range in the geometric stress, also called the hot-spot stress. To produce the S-N curves, full-scale tests are performed and the hot-spot stress ranges are either measured (i.e. strain gauges) or calculated (i.e. refined finite element models) and plotted against the cycles to failure. In effect, the geometric stress concentration is taken out of the fatigue resistance and, instead, is accounted for in the analysis. This has the advantage of collapsing all the S-N curves for different categories into a single baseline S-N curve, but it increases the complexity of the analysis. The test data and the baseline S-N curve still include the effect of the local notch stress concentration, which may be impossible to accurately calculate and, therefore, must still be treated empirically. Furthermore, the CAFL is not the same for each detail.

For the limited number of bridge connection details shown in the AASHTO LRFD Bridge Design Specifications, it is possible to perform full-scale tests on all of these and characterize their fatigue resistance with a handful of categories.(1) Therefore, the nominal stress approach has been satisfactory for the bridge industry, and others as well. However, for tubular structures used in the offshore oil industry, the nominal stress approach breaks down. Fatigue resistance of tube-to-tube connections is dependent upon tube shape, tube diameter, tube thickness, and overall connection geometry resulting in an infinite number of possible connections. Not every possible tube-to-tube connection can be tested to define a unique S-N curve for each. Therefore, the hot-spot stress approach was developed as the only practical alternative for tubular structures.(41, 42) As for bridges, the hot-spot stress approach is useful in understanding the cracking that occurs from secondary stresses (e.g. web gap cracking) since these secondary stresses are not addressed by typical bridge analysis.

The geometric stress location is often in high strain gradient fields and, as a result, measurement of the geometric stress involves considerable uncertainty. To address this problem, guidelines have been established to extrapolate a stress concentration factor (SCF) from multiple strain gauge measurements.(46) Another simple approach is to define the hot-spot stress as the stress measured with a 1/8-inch (3 mm) strain gauge placed as close as practically possible to the weld toe, i.e., centered about ¼-inch (6 mm) from the weld toe.(47) This is essentially the definition used by AWS D1.1.(47)

The baseline curve is sensitive to the definition of the hot-spot stress. British Standard (BS) 7608 defines a Category T curve, along with a correction factor based on the plate thickness, and AWS uses the X2 curve.(48) As it turns out, these curves are similar to the AASHTO Category C curve, which is the nominal stress S-N curve associated with butt welds with reinforcement (not ground flush) in a flat uniform plate. (47) This makes sense since the stress at the weld toe of this detail would include the local stress concentration but would not include any effect of geometric stress concentration. Therefore, the hot-spot stress is equal to the nominal stress in this detail. For assessment of web gap details and other bridge details using the hot-spot approach, it is recommended that the AASHTO Category C curve be used together with a hot-spot stress range measured or calculated ¼-inch (6 mm) from the weld toe.(47)

In design, the hot-spot stress approach involves the calculation of geometric SCF using parametric equations or finite elements analysis (FEA).(46 ,49) However, FEA results are highly mesh dependent, since the geometric stress is often in an area of high strain gradients.

The disadvantage in this method lies mainly in the variability between different hot-spot stress definitions and varying baseline S-N curves. Another problem involves the CAFL. The hot-spot approach implies that all details will have a CAFL at the same number of cycles, while full-scale fatigue tests show that the CAFL occurs at different orders of magnitude of cycles for different categories. As a result, a conservative approach is to ignore the CAFL for hot-spot stress analysis.

FRACTURE MECHANICS APPROACH

Fatigue crack growth can be calculated using the Paris Law. The Paris Law relates the number of cycles to the stress intensity parameter through the relationship shown in Figure 6.

$$\frac{da}{dN} = C(\Delta K)^m$$

Figure 6. Equation. Paris Growth Law.

In the equation, a = the crack size (mm or inches), N = number of cycles, C = material constant, ΔK = the range in stress intensity factor, (MPa-$m^{1/2}$ or ksi-$in^{1/2}$), and m = material constant equal to 3.0 for carbon steels. The stress intensity factor is a measure of crack tip stress and strain field magnitudes. The differential equation can be solved to uniquely define the number of cycles to propagate a crack from one size to another.

However, the complexity of this approach is rarely warranted in design. It is never known what the initial defect size is for the analysis, although previous studies have suggested that assuming an initial semi-elliptical crack with a depth of 0.25 mm (0.01 inch) gives results consistent with test data. (50) Furthermore, the analysis of very small cracks requires accurate knowledge of the notch stress concentration factor and how it decays with increasing crack size.

This type of analysis can be used whenever there is an existing visible crack.(5,13) In this case, the crack is usually large enough that the notch stress concentration is not significant.(13)

CRACK DETECTION TECHNIQUES

Detecting cracks can sometimes be an unwieldy process and may at times seem like a black art. Identifying cracks in painted or rusted structures can be very difficult sometimes requiring supplemental techniques to expose them. This section will briefly outline the common methods of crack detection as they will be referred to later in this manual. A keen eye is the best tool for finding cracks. When cracks cycle open and closed, the crack surfaces rub against each other, creating a fine steel powder that easily oxidizes when exposed to the environment. This often leads to rust staining, or discoloration, as the oxidized material bleeds from the cracks. This can offer quick visual detection of problem areas, but should not be used as the sole means of detection. Two examples of this are shown in Figure 7. To further aid the human eye, two common nondestructive techniques used to expose cracks are dye penetrant and magnetic particle inspection.

Dye penetrant is a three-part system applied to an area where a crack is suspected. Each component is usually contained in small, pressurized cans that can be easily transported in the field. The first step is cleaning/degreasing the area to remove surface contaminants, use a wire brush to remove heavy corrosion, but avoid grinding as is tends to smear out the crack. Then a liquid dye (commonly red in color) is sprayed onto the surface, and this dye seeps into the cracks. After a specified time (~60-90 seconds), the excess dye is wiped away from the surface. A white developer is then sprayed in the same area the dye was applied. The dye within the crack is then drawn out by the white developer, clearly showing the crack as seen in Figure 8.

Magnetic particle inspection, often referred to as "mag" particle inspection, uses a handheld electromagnetic yoke shown in Figure 9 to find cracks. The yoke induces a magnetic field in the area of interest utilizing the physical principle that magnetic fields cannot span over discontinuities. The magnetic field becomes disrupted at a crack and the lines of flux must flow around the crack. This creates a concentration of the magnetic field around the crack. A fine iron filling powder is sprinkled in the area of interest while the yoke is magnetized, and the powder is only attracted to areas with a magnetic concentration, like the crack shown in Figure 10. The technique is good at finding exposed cracks, and in some circumstances cracks that have not yet propagated to the surface. Magnetic particle testing can work through paint, but not reliably and is not recommended. Paint should be removed to provide optimum results.

Other nondestructive techniques can be used, but the availability becomes more limited and cost will increase over dye penetrant and magnetic particle testing. Typically this is because other techniques will require the use of expensive transducers and electronics and highly trained operators. Other techniques that have been successively implemented are eddy current, ultrasonic testing (either manual or automatic), and time-to-flight-diffraction.

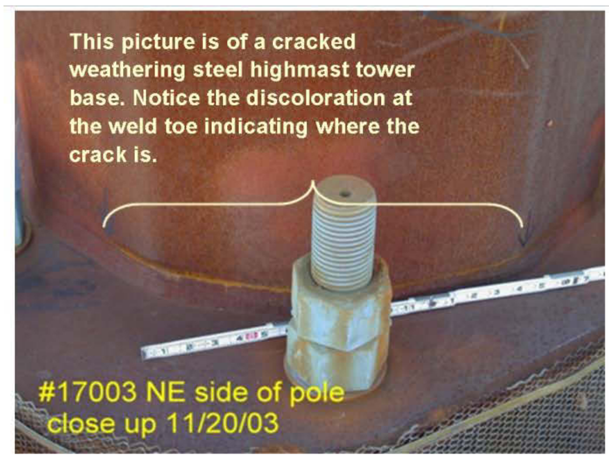


Figure 7. Photo. These two pictures show how cracks can sometimes be found from discoloration or rust staining. Photo courtesy of the Iowa Department of Transportation.

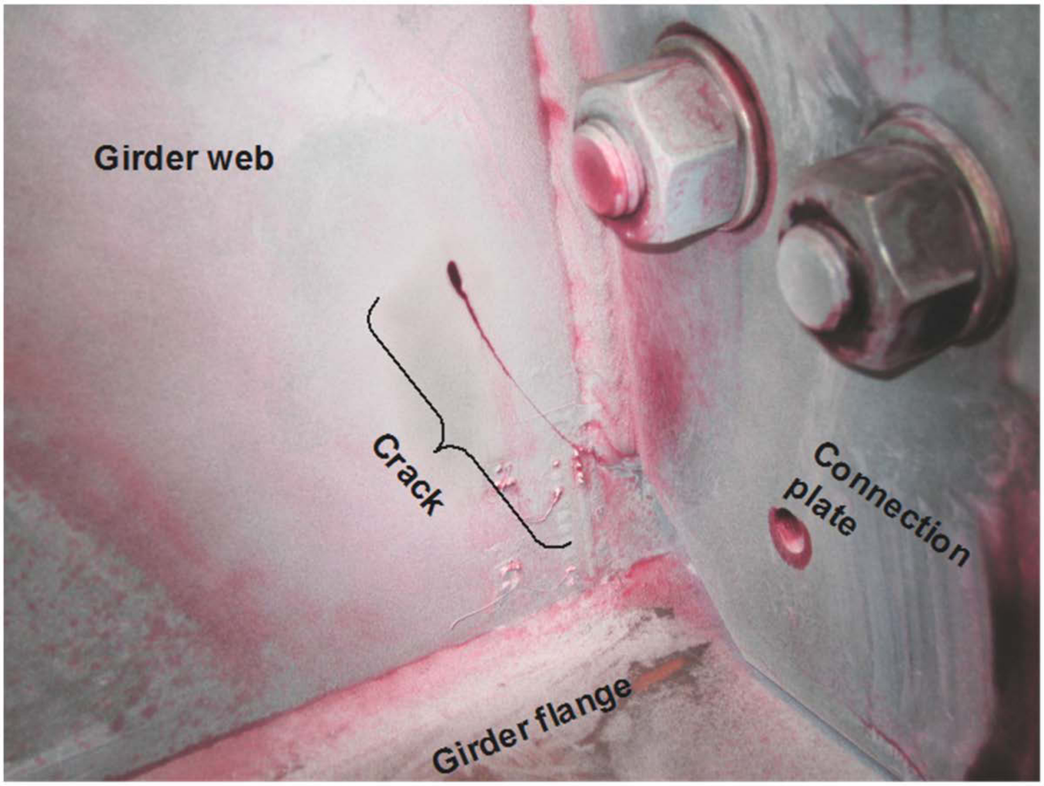


Figure 8. Photo. Crack exposed with red dye penetrant.

Figure 9. Photo. Mag particle yoke (left). Powder bulb for sprinkling iron powder (right).

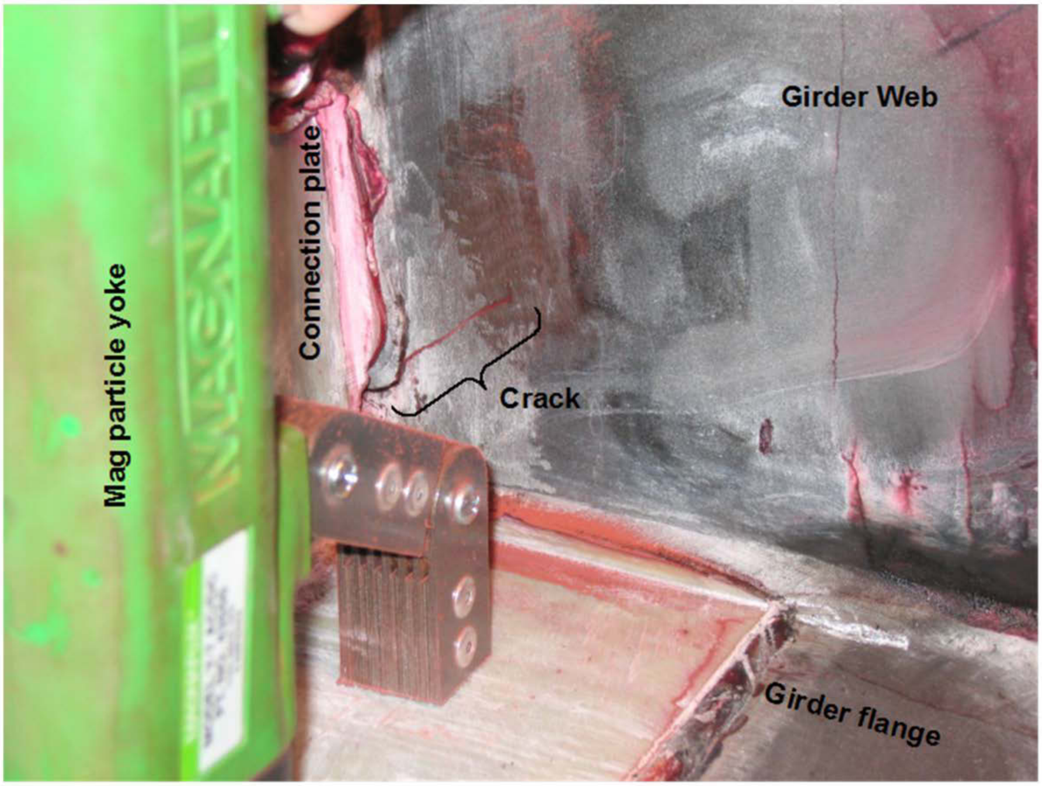


Figure 10. Photo. Crack exposed using magnetic particle testing.

CHAPTER 3. REPAIR AND RETROFIT METHODS

This section presents commonly used repair and retrofit techniques for fatigue critical details. A repair is intended to arrest the propagation of a fatigue crack, while a retrofit is intended to upgrade the fatigue resistance and prevent fatigue cracking. Many different methods are used for repair of fatigue cracks and retrofit of fatigue-prone details. The choice of method depends on the circumstances of the fatigue cracking and may depend on the availability of certain skills and tools from local contractors who would perform the repairs. However, the same treatments or modifications of a structural detail are often used in both repairs and retrofits and, therefore, the methods will be discussed independent of how they are being used.

Repair and retrofit techniques can be categorized into three major categories: 1) surface treatments, 2) repair of through-thickness cracks, and 3) modification of the connection or the global structure to reduce the cause of cracking.

SURFACE TREATMENTS

Weld toe surface treatments include grinding, gas tungsten arc (GTA, also called plasma remelting) of the weld toe, and impact treatments. These techniques can be used as "weld improvement" retrofit methods to increase the fatigue strength of uncracked welds. With any of these treatments, the improvement in fatigue strength can be attributed to one or a combination of the following:

- Improvements in the weld geometry and corresponding reduction in the stress concentration.
- Elimination of some of the more severe discontinuities from which the fatigue cracks propagate.
- Reduction of tensile residual stress or the introduction of compressive residual stress.[51]

The easiest and least expensive treatment is hammer peening, which is a very effective and commonly used impact treatment. Some of the methods, including hammer peening, can also be used for the repair of shallow surface cracks up to 1/8-inch (3 mm) deep. Once the treatment is applied to welds that have already been in service, the remaining fatigue life is at least as long as that provided by original detail when new (assuming the stress-range histogram stays relatively constant). In other words, the damaging effect of prior fatigue loading cycles is removed. In most cases, these treatments result in fatigue resistance that is at least one fatigue category greater than the original detail. Therefore, the next greatest AASHTO S-N curve can be used to predict the renewed life of the repaired detail. Surface treatments only affect the weld toes, so fatigue cracks may still develop from the weld roots, though it is rare.

Reshape by Grinding

Grinding can be used to totally remove portions of a detail containing small cracks, particularly cracks at the edges of flanges or other plates. The gouge created by grinding should be tapered with a 2.5:1 slope and finish grinding should be performed parallel to the applied cyclic stresses (that is, sparks from the grinding operation should fly in the direction of the primary stress, thus causing grinding scratches to be parallel to the primary stress). For example, an edge crack ½-inch (13 mm) deep should be removed by faring in the grinding along the edge for a total length of at least 2.5 inches (65 mm). Consideration should be given to the possible reduction in net section area.

When a weld is first made, small microcracks form at the weld toe as the weld pool cools to ambient temperature and contracts. Under cyclic loading, these microcracks begin to propagate and become small fatigue cracks. Studies have shown that grinding away these microcracks does not yield an appreciable increase in fatigue strength, perhaps because the defects introduced by the grinding are comparable to the initial microcracks.(52)

In offshore structures with tubular joints, where the welds are very large, grinding is effective at shaping the weld and enhancing fatigue strength by reducing the associated stress concentration factor. Welds reshaped with a burr grinder have been found to have a 50% larger allowable fatigue design stress range over their untreated counterparts.(53) Extensive grinding of welds in cover plate details has also been shown to be effective.(54,55) However, such extensive grinding is considered overly expensive as a retrofit technique for bridges.

Grinding is also useful as a finishing process for larger repair and retrofit techniques, such as hole drilling (burr removal) and welding (toe finishing). Two types of grinding methods are disc and burr grinding.

Disc Grinding

Disc grinding is an effective means to remove metal. However, if the grinder operator is not careful, too much material may be removed. In addition, if the operator loses control of the grinder, the metal may be gouged.

Figure 11 depicts using a disc grinder to grind a weld toe. The goal of the grinding is to remove a small amount of material. As shown in Figure 11, the depth of the ground area should be a minimum of 1/32-inch (0.8 mm) below the original weld toe, and a maximum of 5/64-inches (2 mm) or 5% of the plate thickness.(56) Only one pass of the grinder should be necessary. Typical grinding wheels range in size from 4 to 9 inches (101to 230 mm) and are made from Aluminum oxide with 24-grit roughness. Figure 12 shows a picture of two typical disc grinders.

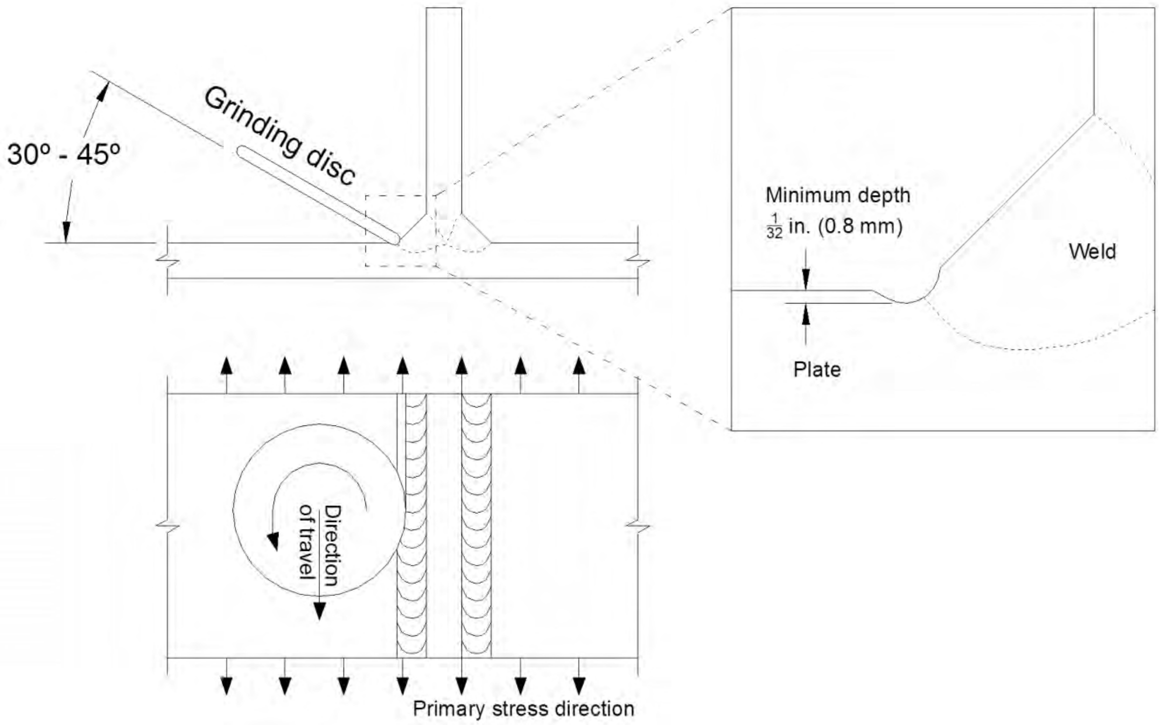


Figure 11. Illustration. How the grinder should be held relative to the material being ground. Reproduced from Gregory et. al.(56)

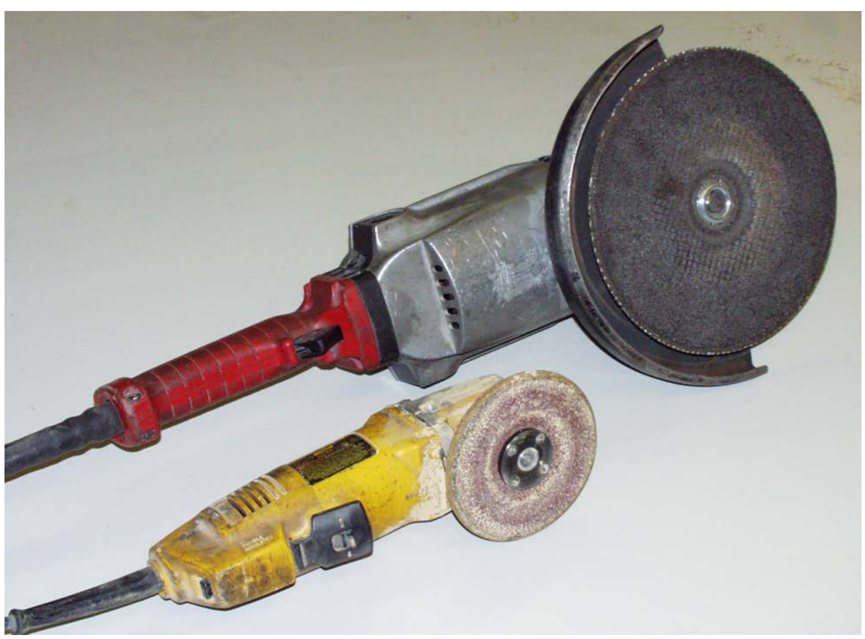


Figure 12. Photo. Typical disc grinders.

Burr Grinding

Burr grinding has advantages and disadvantages over disc grinding. Burr grinders are easier to handle and have the ability to work in more confined regions than grinding discs. Burr grinders are excellent for grinding the terminations or edges of welds and for enlarging small drilled holes, especially if the hole desired is not exactly circular. However, the cutting rate of burr grinders, 3 ft/hr (1 m/hr), is slower than that of grinding wheels,6 ft/hr (2 m/hr).(56) The burr grinding operation is similar to disc grinding; however the tool is different, and it needs to be held differently.

Figure 13 shows a diagram of how the burr grinder should be held during the grinding of a weld toe. Tungsten carbide rotary burr tips with a radius of 3/16 to 5/16-inch (4.7 to 8 mm) are recommended. It is good practice to finish the ground area with a stone attachment in the burr grinder, though experimentally it does not yield appreciable gains in performance.(53) Figure 14 shows a typical burr grinder.

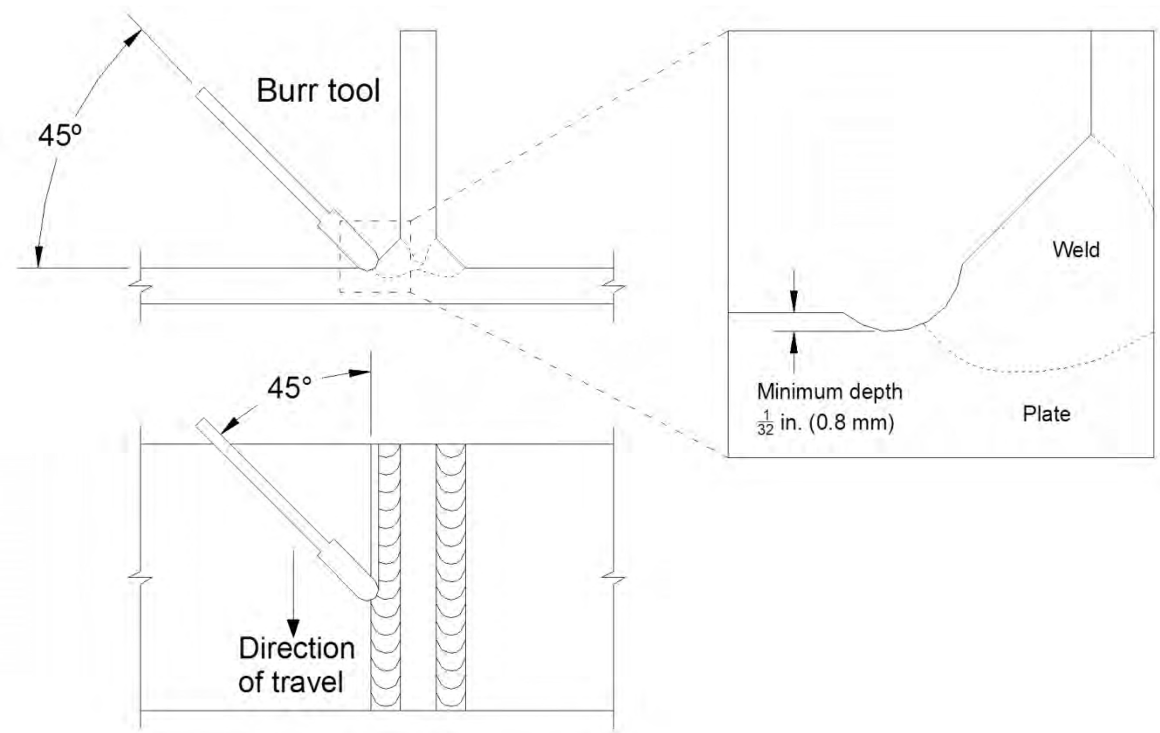


Figure 13. Illustration. How to hold burr grinder relative to material surface. Reproduced Gregory et. al.(56)

Figure 14. Photo. Typical burr grinder and carbide burr tips.

Gas Tungsten Arc (GTA) Remelting

GTA remelting is an autogenous welding operation along the weld toe and the base metal. This is accomplished by manually moving a gas shielded tungsten electrode at a constant speed along the weld toe and just melting the metal without addition of new filler metal. The GTA remelting process tends to remove slag intrusions as well as reduce the stress concentration at the weld toe.(52) GTA remelting may also be used as a repair procedure for cracks up to approximately 3/16-inch (5 mm) deep with good results provided the crack is fully removed.(52) However, the effectiveness requires great operator skill and adequate penetration, both of which may be difficult to attain for a field retrofit. It is recommended that the operator have skills greater than or equal to those of someone working in a fabrication shop which may make this retrofit technique cost prohibitive in comparison to other techniques.

Impact Treatments

Impact treatments work by introducing compressive residual stress near the weld toe. Because the compressive residual stress lowers the effective tensile stress range locally on crack-like defects, it is most effective when conducted under dead load. In this case, the impact treatment only has to be effective against live load. If dead load generated tensile stresses are applied after impact treatment, the local compression induced by the treatment may be overcome. Therefore, impact treatments are best suited as retrofit techniques. There are different types of impact treatments, but only two will be discussed as they pertain to steel bridges: 1) Air hammer peening, 2) Ultrasonic impact treatment.

Air Hammer Peening

Hammer peening is a simple and effective technique for weld improvement. The method uses an air hammer with a blunt tip (commonly used by welders to remove slag from welds, see Figure 15) to plastically deform the weld toe creating a compressive state of stress where tensile residual stress was produced during the welding process. This lowers the tensile stress range the weld toe experiences under live load and extends the fatigue life. The hammer peening process changes the appearance of a weld, giving the weld toe a shinier surface as shown in Figure 16.

Hausammann et al. studied the effect of air pressure and number of passes on the resulting deformation and associated residual stresses.(57) Best results were obtained with lower pressures (40 psi (280 kPa)) and greater number of passes (up to six). Using those parameters, a depth of plastically deformed grains greater than 0.02-inch (0.5 mm) was consistently obtained. The recommended depth of penetration is 0.03-0.06 inches (0.8-1.6 mm) as shown in Figure 17. However, the depth should be varied depending upon the thickness of the element being peened. Peening thinner materials should require less penetration and must be done with caution so that the thickness of the material is not reduced too much. The depth of compressive residual stress extends from two to four times the depth of the plastically deformed grains, i.e. from 0.04-0.08 inches (1-2 mm) deep. Peening has been shown to be effective in repairing surface cracks up to 0.12-inch (3 mm) deep.(57,58)

Peening is also useful for improving the fatigue performance of welds without detectable fatigue cracks. Experiments have shown that the fatigue resistance is increased by at least one detail category.(57,58) Fatigue resistance of repair welds can also be improved by up to 175% by peening after repair welding.(58,61) The benefit of peening welds without detectable defects is primarily due to the introduction of a residual compressive stress. For example, Harrison found that welds stress relieved after peening resulted in no significant increase in fatigue life, proving the enhancement of the fatigue resistance can be attributed to the compressive residual stresses.(62)

It might be expected that blunting or eliminating many crack-like defects at the weld toe, such as slag inclusions, would improve fatigue resistance. However, the peening operation also introduces numerous lap-type defects. The depths of these defects range from 0.002-0.01 inches (0.05-0.25 mm), the same depth as typical weld toe slag inclusions. Unfortunately, like the slag inclusions these defects serve as a site for crack initiation. Greater improvement is reportedly obtained when the peening operation is followed by light surface grinding, which apparently removes many of these lap-type defects.

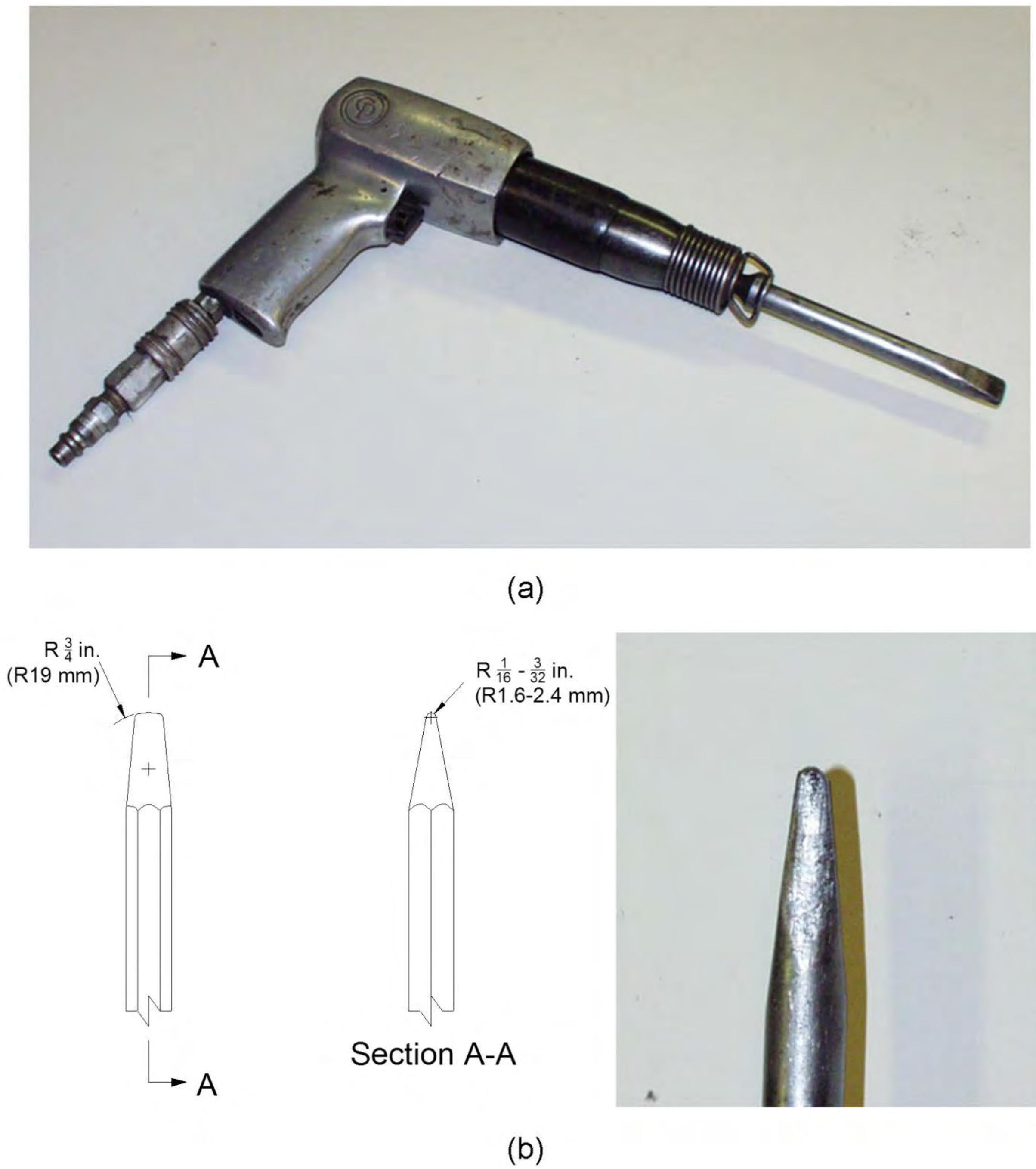


Figure 15. Photo. (a) Pneumatic hammer/chisel. (b) Close-up detailing of peening tip. Reproduced from Hausammann et. al. [57]

Figure 16. Photo. A peened weld toe, note the geometry and luster change at the weld toe.

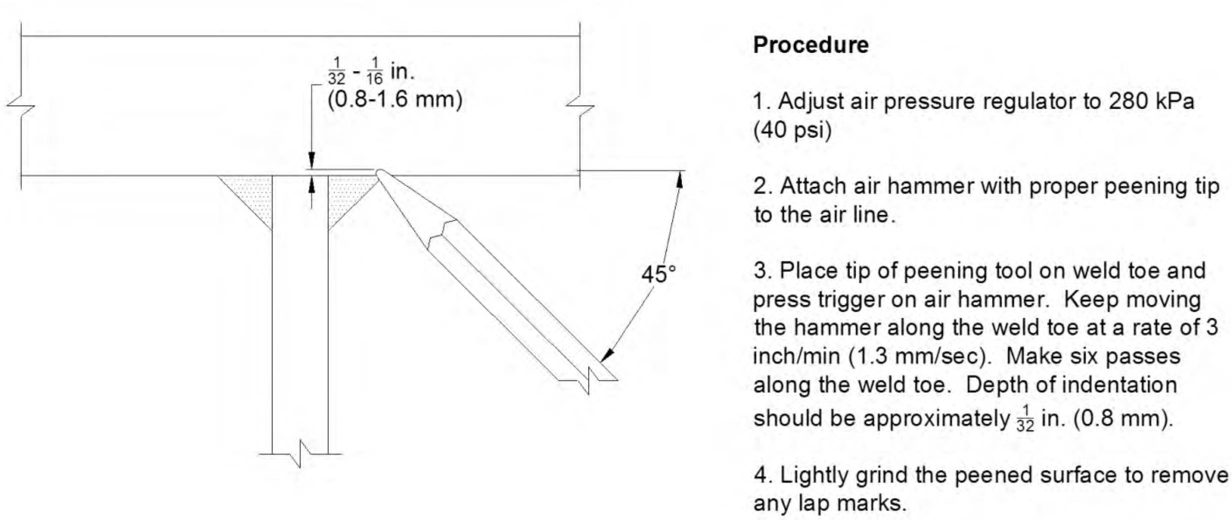


Figure 17. Illustration. Hammer peening procedure showing depth of penetration. Reproduced from Hausammann et. al.[57]

Ultrasonic Impact Treatment

As an alternative to hammer peening, ultrasonic impact treatment (UIT) is an attractive option. Hammer peening is a noisy and physically enduring task. Ultrasonic impact is relatively quiet, and the equipment is much easier to handle. Ultrasonic impact treatment involves low-amplitude (20-50 microns), high-frequency displacements (27-55 kHz). UIT has proven more effective than hammer peening at improving the fatigue performance of welded joints, so it can be assumed that UIT will increase the fatigue strength of cover plate details and transverse stiffeners by at least one detail category.[63,64] The effectiveness of UIT is enhanced at lower stress ranges and low mean stresses. If used as a retrofit, the dead load of the structure is already present, so the fatigue category may be upgraded by more than one detail. However, the equipment is currently proprietary, requires a licensing agreement, and may be more expensive to use over conventional hammer peening. Additional research into the effectiveness of the method on full-scale welded bridge girders is continuing. At the time this manual was published, UIT technology was still proprietary, so manufacturer directions for treating welds must be followed.

HOLE DRILLING

Hole drilling is perhaps the most widely used repair method for fatigue cracks. The hole drilling method requires placing a hole at the tip of the crack, removing the sharp notch at the crack tip. However, the hole needs to be of sufficient diameter to be successful in arresting the crack. As a rule of thumb, larger holes are better as long as strength and stiffness of the structure or connection is not compromised. Sufficient hole diameters typically in the range of 2 to 4 inches (50.8 to 101.6 mm) have proven successful through field experience, but depending on application a 1 inch (25.4 mm) hole may also be sufficient. Larger holes are not always practical and many owners are uncomfortable placing such a large hole in a member.

A more engineered hole-size can be attained using the equation in Figure 18.[65] Where ΔK is the stress intensity factor range, σ_y is the yield stress of the material, and ρ is the radius of the hole. ΔK may become difficult to calculate in certain situations and a simplifying assumption would be to use the center-cracked plate solution under uniform stress shown in Figure 19. In this equation, S_r is the nominal stress range where the crack tip is located, and a is the half-crack length (the distance between crack tips is $2a$). Using this simplified approach, the equations in Figure 18 can be reduced to the form shown in Figure 20 which directly output the required hole diameter (inches for U.S. Customary units and millimeters for SI units). Depending on the inputs, the equation could predict a required hole diameter less than 1.0 inch (25.4 mm), which is not practical and a minimum hole diameter of 1.0 inch (25.4 mm) should always be enforced. If the crack originates from a free surface (also known as an edge crack), the center-cracked plate solution assumed by the equation in Figure 20 yields unconservative hole diameters. In the case of an edge crack, hole diameters predicted by the equations in Figure 20 should be increased by 25%.

$$\frac{\Delta K}{\sqrt{\rho}} \leq 10.5\sqrt{\sigma_y} \text{ (for } \sigma_y \text{ in MPa)} \qquad \frac{\Delta K}{\sqrt{\rho}} \leq 4\sqrt{\sigma_y} \text{ (for } \sigma_y \text{ in ksi)}$$

Figure 18. Equation. Required crack tip curvature to arrest crack growth.

$$\Delta K = S_r \sqrt{\pi a}$$

Figure 19. Equation. Center-cracked plate solution for ΔK.

$$D = \frac{S_r \pi a}{8\sigma_y} \geq 1.0 \text{ in.} \quad (\text{for } \sigma_y \text{ in ksi, } a \text{ in inches})$$

Figure 20. Equation. Required hole diameter to arrest crack growth in imperial units.

$$D = \frac{S_r \pi a}{55\sigma_y} \geq 25.4 \text{ mm} \quad (\text{for } \sigma_y \text{ in MPa, } a \text{ in mm})$$

Figure 21. Equation. Required hole diameter to arrest crack growth in metric units.

Holes should be plugged with a rubber or wooden plug so that drivers or pedestrians passing under the bridge do not become uncomfortable seeing daylight through a series of holes. For box girders, it becomes important to plug large holes to prevent birds and other animals from inhabiting the interior of the box. Fecal or nesting materials left from birds and animals can lead to expedited corrosion and/or an environmental concern within the box. If the hole size is from 1.0 to 1.5 inches (25.4 to 38.1 mm) in diameter, it is recommended to place a fully-tensioned, high-strength bolt into the hole (either A325 or A490), with hardened washers on both sides. In addition to plugging the hole, the bolt introduces favorable compressive stresses around the entire hole, which may contribute to preventing further crack growth.[58,65] Some engineers have expressed concern that the bolt prevents inspectors from observing if cracks are reinitiating. Experimental observations have found that the hardened washer may crack along with the base material if a crack reinitiates.[66]

Cold expansion of the hole is another way to introduce the beneficial compressive residual stress around the hole. Once the hole is drilled, a tapered mandrel (also referred as a drift pin) slightly larger than the hole can be forced through by hitting the pin with a hammer. As the pin passes through the hole, the hole plastically deforms creating the compressive field.

Materials and Equipment:

Drill bits: Two types of drill bits exist for drilling steel: annular cutter (or hole saw) and twist bits, both are shown in Figure 22. Twist bits should only be used for holes up to 1.0 inches (25.4 mm) in diameter. Larger holes are possible with twist bits depending on the type of magnetic-base drill used, but cutting rates are slower and the finish is not as clean. Ideally, carbide-tipped or high-speed steel annular cutters (a.k.a. hole saws) should be used to core larger holes. These cutters leave a cleaner cut surface, have much higher cutting rates, and use a more compact magnetic-base drill which is attractive for field retrofits. The carbide-tipped annular cutters should be used exclusively when drilling through welds or drilling high-strength material (AASHTO M270 Grade 100, ASTM A514 or T1 steel). **Flame-cut holes should never be used to remove crack tips.** The flame cutting operation can introduce a gouged surface condition that may initiate new fatigue cracks.

Drill: A portable drill with an electromagnetic base is recommended. Refer to manufacturer's recommendations to determine optimum type of drill, motor, mounting options, etc.

Figure 22. Photo. Annular cutter (left). Twist bit (right).

Procedure:

1. Identify the tip of the crack using a recommended method (see Chapter 2, Crack Detection Techniques). Magnetic particle and red dye penetrant are the easiest and most widely used. Mark the tip of the crack with a center punch.

2. For through thickness cracks, it is advised to drill a small reference hole (~1/8 inches (3 mm) in diameter) to check the crack tip location on the opposite side and adjust the positioning of the coring machine accordingly. Position the drill/coring bit such that trailing edge of the bit removes the crack tip. This ensures the tip has been removed to avoid leaving a small crack, which would continue to grow, as illustrated in Figure 23. If cross-section loss is a major consideration, the bit can be centered over the crack tip but it increases the likelihood of missing the crack tip.

3. Select proper diameter of drill bit using equations from Figure 20. Larger holes than predicted with these are always allowable, but a minimum of 1.0 inch (25.4 mm) should be enforced.

4. After drilling, it is imperative that the surfaces around the hole be ground smooth. The drill bit typically leaves burrs in the metal as the bit exits the hole, and these burrs can lead to new crack initiation sites. All hole edges should be ground smooth to an ANSI roughness of 500 of less. This surface roughness can be achieved using 80-100 grit sandpaper, or finer. In addition, if twist bits are used, the surface within the hole should be ground smooth, as these imperfections can initiate cracks. A small abrasive drum works well for finishing the inside of a hole.

5. Fill the hole with a plug or tensioned bolt if also desired. Always prime and paint bare surfaces.

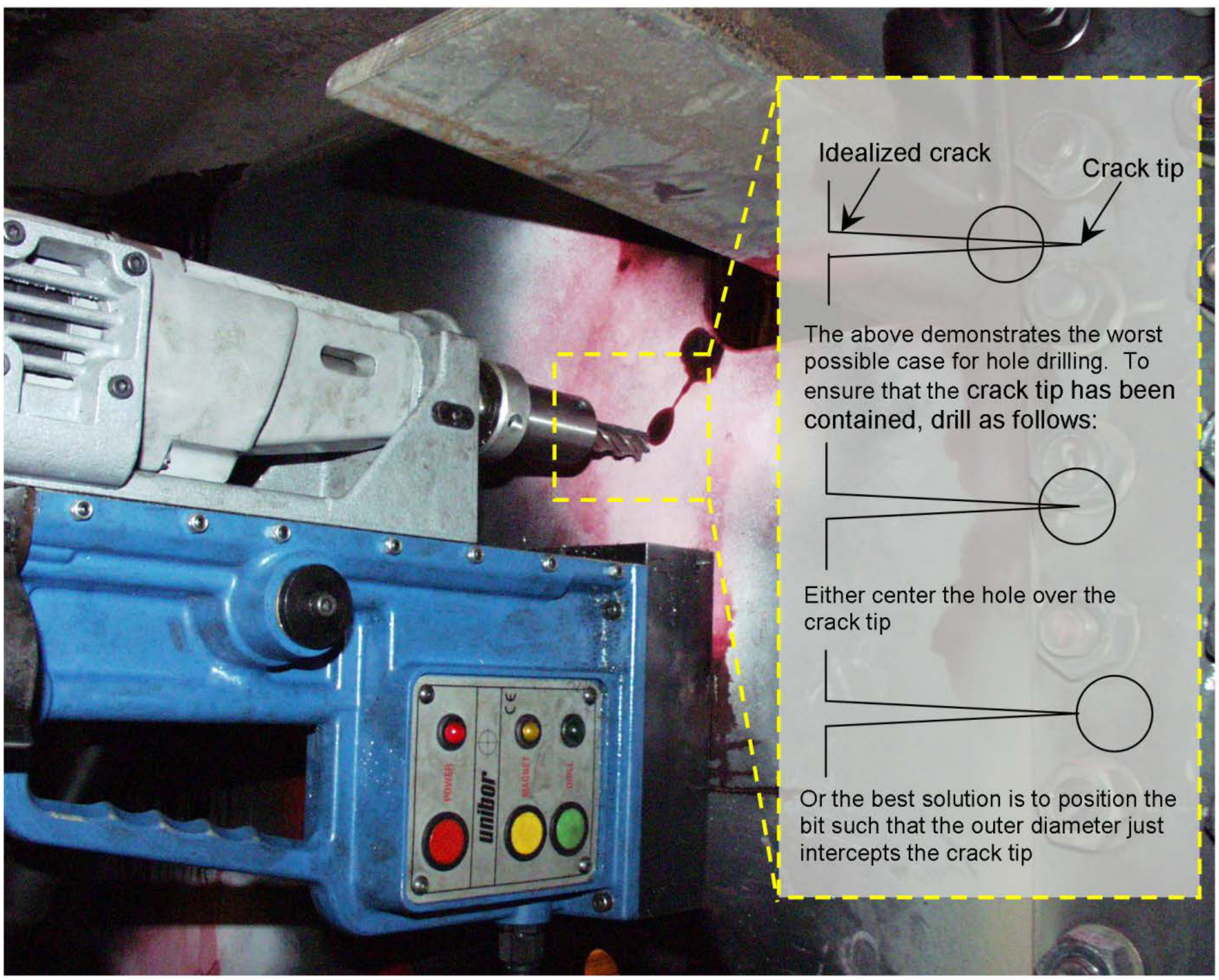


Figure 23. Illustration. Crack tip identification with red dye penetrant and proper drill bit placement.

VEE-AND-WELD

Vee-and-weld is a method of weld repair for long, through-thickness cracks. Once a crack is identified, material is removed along the crack length, through three-quarter the thickness of the section that is cracked in the shape of a *V*. The V-shaped groove is then filled with weld metal, and the process is repeated on the other side of the section, so the entire crack is replaced by weld. The preferred method of material removal is air arc gouging, but grinding also works. The disadvantage of using a disc grinder to remove the cracked material is that the crack can become blurred (or smeared) as more material is removed. This increases the possibility of hiding or masking the crack path and leaving an embedded flaw within the repair weld. When air-arc gouging, the crack seems to open up as the material is removed, making the crack path easy to follow. Figure 24 shows a cracked rolled beam flange that was carbon arc gouged. In the figure it can be seen that an access hole had to be cut in the web to facilitate complete crack removal. In repairs on a bridge member, it is advisable to core drill holes to make the access hole, or grind smooth any flame cut edges so the access hole does not initiate fatigue cracks.

The vee-and-weld repair should be specified with care as the quality of welds made in the field is generally lower than those made in a fabrication shop because of environmental conditions and awkward positioning. Welders should be experienced, certified, and at least as skilled as those working in a fabrication shop.

Vee-and-weld repairs are most effective when used to repair a cracked weld detail, not cracks in unwelded base metal. The repair weld usually exhibits the same fatigue life as the original uncracked weld detail. However, vee-and-weld repair of base metal where there originally was no weld is rarely effective. In this case, a fatigue crack usually reoccurs in a period of time less than the time it took the original unwelded detail to crack.[58-60] In this case, the vee-and-weld repair must be used in conjunction with other techniques to reduce the stress range that resulted in the original crack.

A few studies have looked at the fatigue resistance of repair welds.[67-69] These studies were done on plates and highway bridge girders with welded attachments. The studies concluded that, at best, the fatigue resistance of a repair weld can only be as good as the original detail itself. However, failure to completely remove the crack and poor weld quality will result in a lower fatigue resistance. If post-weld improvement techniques are used (peening, grinding, GTA remelting), then fatigue resistance is likely to increase. Studies have shown that when stringent non-destructive testing is used, original butt welds can be as good as Category C details. Full-penetration weld repair of cracked butt welds restores the detail's fatigue life provided the crack was completely removed during the repair.[8] This kind of non-destructive testing was not used in recent ship structures research regarding weld repair, consequently the original butt welds were shown to be AASHTO Category D details.[58] Surprisingly, the repair welds were also found to be Category D, regardless of whether a two-sided or one-sided weld was used and whether the latter was made with or without a backing bar. The cyclic loading prior to a repair apparently has no effect on the fatigue life of the repair. In addition, multiple cycles of repair (up to four) have no detrimental effect on the restored fatigue strength. Weld access holes were also shown to be Category D details.

Materials and Equipment

Arc gouging: Air arc gouging is performed using standard equipment. Consumables ranging from 0.19-0.25 inches (4.8-6.4 mm) in diameter can be used depending on the thickness of base metal.

Weld process: Repairs can be made using the shielded metal arc welding (SMAW), flux-cored arc welding (FCAW), or gas metal arc welding (GMAW) process depending on availability and site conditions. Special attention should be given to the development of the welding procedures (preheat, welder certification, etc.) to ensure a high quality field weld.

Consumables: All-position, bridge quality, consumables should be chosen so that repairs can be made at any angle. Since the deformation of the repair weld will be constrained by surrounding structure, the strength of the finished repair weld is typically not an important issue. The certified nominal minimum ultimate tensile strength should not exceed 70 ksi (480 MPa) (i.e. E70XX). Undermatched welding consumables with ultimate tensile strength less than 60 ksi (420 MPa) (i.e. E60XX and less) are available and should be used if possible to reduce residual stresses in highly constrained areas. A low-hydrogen SMAW electrode such as E6018 or E7018 is recommended to help prevent hydrogen cracking. If using the FCAW process, gas shielding is preferred to keep the diffusible hydrogen to a minimum, although suitable self-shielded consumables may also be used. The consumable chosen should have a minimum Charpy V-notch value of 25 ft-lbs (34 Joules) at -20 °F (-30 °C) unless a lower value can be justified using the provisions of the AASHTO specifications.(2)

High-strength steel: A high risk of hydrogen cracking exists when repair welding high-strength steel (yield strength 70 ksi (480 MPa) or greater). Weld repairs should only be used for high-strength steel if other types of repairs, such as bolted doubler plates, are absolutely not possible. Undermatched consumables should be used when welding high-strength steel to reduce the residual stresses caused by welding. Minimum preheat requirements per AWS D1.5 and low hydrogen practices must be met for welding high-strength steel to reduce the possibility of enbrittlement cracking.(2)

Procedure

1. Identify crack tips with dye penetrant or magnetic particle testing as shown in Figure 8 and Figure 10 respectively.

2. Grind or air arc gouge the length of the crack, through three-quarter the thickness of the cracked material. Make sure the channel has a distinct V- or U-shape before proceeding to the next step.

Note: Make sure to remove the entire crack for a partial-thickness crack. Often the crack can be seen by the gouger or welder and may appear to propagate during gouging or welding. The gouger and welder should be trained to report this and to take appropriate measures to locate the ends of the crack again and increase the extent of the repair if necessary.

3. Clean and remove all slag from the gouged channel. Fill in the gouged channel with a series of small stringer beads from appropriate weld metal following all specifications outlined in AWS D1.5. This completes the partial-thickness repair.

4. From the opposite side of cracked material, grind or air arc gouge out the remaining crack through three-quarters the thickness of the material being repaired, making sure to partially gouge out the weld metal just deposited.

5. Clean and remove all slag from the gouged channel with a grinder or air chisel. Fill in the gouged channel with a series of small stringer beads from appropriate weld metal following all specifications outlined in AWS D1.1. This completes the full-thickness repair.

6. Grind both sides of the new weld smooth to be flush with the existing material.

7. If necessary, prime and paint the bare surfaces. Document the location and repair procedure for inclusion in the bridge file for use during future inspections.

Figure 24. Photo. Air arc gouged flange of a beam for Vee and Weld Repair, which was completely cracked. The hole had to be place in the web to allow for complete gouging of the flange. The access hole has not been ground smooth as recommended in this manual.

ADDING DOUBLER/SPLICE PLATES

Another technique that can be used to repair through-thickness cracks is doubler or splice plates, hereafter referred to as doubler plates or doublers. Doubler plates add material to either increase a cross-section or provide continuity at a cracked cross-section. The philosophy of doublers for fatigue crack repair is to add cross-sectional area, which in turn reduces stress ranges. For instance, if a fatigue crack grows across the full depth of the bridge girder, there are two ways that it can be repaired. One, a vee and weld repair can be specified, but as stated in the previous section, the base metal that is weld repaired will most likely have a shorter fatigue life than the original detail. To ensure the weld repair will have adequate fatigue resistance, doubler plates can be added after the repair is made to decrease the stress range that contributed to the original cracking thus protecting the repair. One problem with this type of repair is maintaining the alignment of the two sides of the cracked section prior to the weld repair. As seen in Figure 25, the cracked surface usually develops buckles, making alignment difficult. Since doubler plates would be needed anyway to reduce the stress ranges, the plates should be designed thicker assuming the girder web is no longer transferring load. This technique is particularly useful when a full-depth crack forms in a bridge girder. In this case, the doubler plates are intended to restore the full cross-sectional properties of the uncracked girder. The design process is identical to that used for a bolted field splice connection. Doublers can also be used to restore the properties of a section that is heavily corroded.

Doublers can be attached by welding or by using high-strength bolts as shown in Figure 26. From a fatigue-resistance standpoint, the bolted doublers are always better than welded, because a high-strength bolted connection can be considered an AASHTO Category B detail. A welded connection will most likely result in a Category E condition or worse. As a result, this manual recommends specifying only bolted doubler connections.

Materials and Equipment

Material: Any grade of steel that meets the AASHTO M270 specifications may be used for detail modifications. Acceptable weld metals are the same as those described in the Vee-and-Weld repair method section (pg. 30). If bolts are used, then all bolting materials should conform to ASTM 325 or 490 standards. Washers should conform to ASTM F436.

Workmanship: Brackets, plates, and stiffeners may be flame cut. However, if a radiused corner is used, it must be ground smooth or saw cut.

Procedure

Note: This retrofit can either use pretensioned bolts or fillet welds to attach the doublers. However, care must be taken when specifying welded retrofits so as not to create a much worse fatigue detail than the repair is intended to replace.

1. Align both sides of the crack if there is gross deformation
2. Drill out crack tip with a minimum 1.0 inch (25.4 mm) diameter hole as described in the section on Hole Drilling repairs (pg. 27). A larger hole maybe required, but the purpose of the hole is to remove the crack tip, not to solely cease crack growth. The equations in Figure 20 should use the stress range calculated using the cross-section with the doublers in place to verify the 1.0 inch (25.4 mm) hole is sufficient.
3. Determine the required cross-sectional area of the doublers depending on whether the doublers will be intended to reduce stress ranges, restore a cracked section, or both.
4. Determine the number of bolts required to develop the capacity of the doubler plates assuming a slip-critical bolted connection. Then determine the pattern over which to distribute the bolts, conforming to all hole spacing and edge distance requirements as per AASHTO.(1) This will determine the final dimensions of the doubler plates. For welded doublers, size the welds to develop the capacity of the doublers.
5. Fabricate the doublers with all the holes drilled for bolted retrofits. For welded retrofits, round all the corners of the doubler plates to a minimum radius of 2

inches (50.8 mm) and proceed to step 6. A larger radius is always preferred, but according to the AASHTO specifications, this minimum radius will demonstrate Category D resistance as the fillet weld attaching the doubler would be a Category E or E' detail depending on thickness.[1]

6. In the field, clamp one of the doublers against the surface it will be attached to, and use the doubler as a drilling template. Drill all holes in the cracked structure.
7. After all holes are drilled, remove the doublers and clean all contacting surfaces to remove oil, grease, dirt, and cutting fluids to restore the friction surface of the slip-critical connection.
8. Reposition the doublers and fill all the holes with high-strength structural bolts. Initially snug tighten all the bolts making sure all surfaces are mated. Use turn-of-nut method to fully tension all the bolts after the snug tight operation, or use tension-controlled bolts. See Table 1 for turn-of-nut requirements published by the Research Council on Structural Connections.[70] For welded retrofits, weld the plates in place trying to avoid stop/start welds on the plate edge perpendicular to the direction of the primary stress field.
9. For welded retrofits, peen all weld toes that are perpendicular to the primary stress field to increase fatigue resistance as described in the section for Impact Treatments repairs (pg. 23).
10. Prime and paint the bare surfaces. Document the location and repair procedure for inclusion in the bridge file for use in future inspections.

Figure 25. Photo. Full girder depth fatigue crack of Lafayette Street Bridge in St. Paul, MN. Photo courtesy of the Minnesota Department of Transportation.

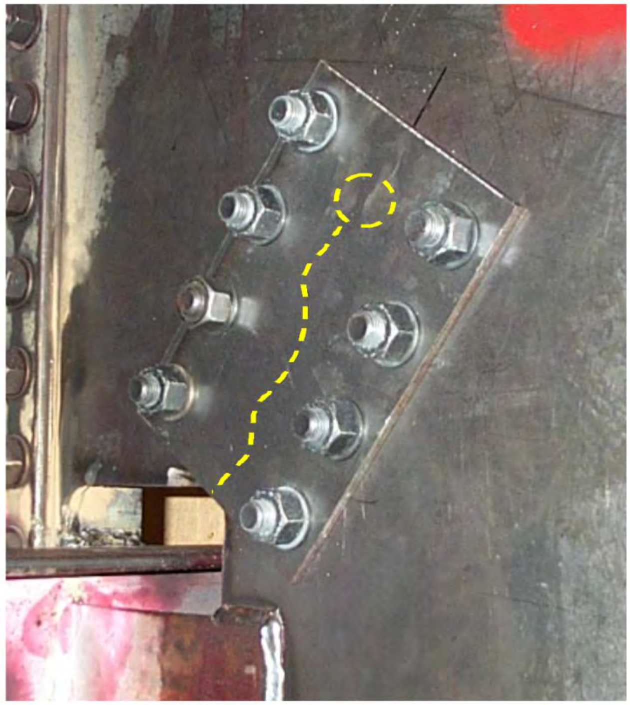


Figure 26. Illustration. Bolted doubler plate repair. Dashed line represents crack beneath doubler plate and circle is the hole drilled to remove the crack tip.

Table 1 – Nut Rotation From Snug-Tight Condition for Turn-of-Nut Pretensioning[a,b]

Bolt Length [c]	Disposition of Outer Face of Bolted Parts: Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20 [d]	Both faces sloped not more than 1:20 from normal to bolt axis [d]
$L \leq 4d_b$	⅓ turn	½ turn	⅔ turn
$4d_b < L < 8d_b$	½ turn	⅔ turn	$^5/_6$ turn
$8d_b \leq L \leq 12d_b$	⅔ turn	$^5/_6$ turn	1 turn

[a] Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For required nut rotations of ½ turn and less, the tolerance is ±30°; for required nut rotations of ⅔ turn and more, the tolerance is ±45°.

[b] Applicable only to joints in which all material within the grip is steel.

[c] When the bolt length exceeds $12d_b$, the required nut rotation shall be determined by actually testing in a suitable tension calibrator that simulates the conditions of solidly fitting steel.

[d] Beveled washer not used.

POST-TENSIONING

For fatigue cracks to propagate, the crack needs to open and close. Crack closure concepts can be used to slow down or eliminate fatigue crack growth. Post-tensioning forces can be applied to cracked beam sections, forcing crack faces to close together and push the effective stress range into the compression regime to hinder additional crack growth. A limited amount of research has verified the validity of this retrofit. One study applied post-tensioning forces to beams that had fatigue cracked cover plate details. The cracks were allowed to grow into the web before the post-tensioning was applied with prestressing strands. The strands were sized and stressed so that the cracked flange was always in compression during cycling. In addition, no efforts were made to blunt or remove the tip of the existing crack, such as drilling a hole. The repaired section was cycled, and the fatigue strength of the repaired section was found to be one category higher than the original detail.[71]

For repairing cracked sections using post-tensioning, it is recommend that the tip of the existing crack be removed by drilling in addition to applying a post-tensioning force. A number of options exist for applying post-tensioning force, depending on the amount of force needed, types of anchorages, etc. Post-tensioning could be applied by jacking prestressing strands or post-tensioning bars, or nuts can be torqued on high-strength threaded rods. Whatever method is chosen, proper corrosion protection must be applied to the post-tensioning system.

MODIFY THE DETAIL

When the stress range in a repaired region must be reduced for a repair to be effective, such as in a weld repair made at a location where there originally was no weld, then a detail modification is required. This could involve adding doubler plates to increase the cross-sectional area. If the detail contains a sharp corner, then introducing a soft-toe or radius can reduce local stresses considerably. Another option may be to change the overall geometry of the connection to reduce stress in the joint. Care must be taken, however, to ensure that any design modifications do not become fatigue problems themselves.

CHAPTER 4. FATIGUE DUE TO SECONDARY STRESSES

Many assumptions are made in the analysis and design of steel bridges. Some of these assumptions are rather simplistic and do not describe the real behavior of the bridge. Furthermore, design codes list procedures for designing individual structural elements and often do not take into account global system behavior. Neglecting the real behavior in design gives rise to secondary stresses, which could lead to fatigue cracking. For example, a "simple" shear connection using angle clips is assumed not to transfer moment in design; however, deep shear connections can develop a force couple between the bolts, and the as-built connection really does transfer moment. The development of a moment in a shear connection gives rise to secondary stresses. Fatigue cracks can form under load or displacement-controlled boundaries, but secondary stresses usually develop under displacement-control boundaries where two members are constrained to deform together, but not intended to do so in design.

OUT-OF-PLANE DISTORTION

Web-gap fatigue arises from secondary stresses induced in bridge girders from displacement compatibility between the girders and the bracing elements. Figure 27 depicts the most common form of this fatigue problem with differential girder displacements. This phenomenon can occur anywhere along the length of the bridge, but is by far most common in the negative moment region. The connection plates are typically not welded to the tension flange (at least on bridges designed prior to 1985). As one girder deflects more than its neighbor, it induces bending in beam-type diaphragms (or axial forces in crossframes), which is resisted through a force couple by the connection plate welded to the girder. In the negative moment region, the tension flange is embedded in the concrete deck, which restrains its lateral movement, therefore pulling of the lateral bracing element causes highly localized bending of the web gap. The web gap is the small, unstiffened portion of the web between the longitudinal flange/web weld and the weld of the connection plate.

Web-gap fatigue cracks form at ends of connection plates that are welded to girder webs. These plates are used to connect floor beams, diaphragms, and X-, or K-type crossframes between girders. The typical cracking pattern is a horseshoe-shaped crack originating from the bottom of the web gap and extending down both sides of the connection plate into the girder web, or a longitudinal crack at the bottom toe of the web/flange weld. Figure 28 shows a typical horseshoe-shaped crack. To properly repair web-gap fatigue cracks or retrofit the detail, the driving stress of the crack needs to be mitigated, i.e., the source of the out-of-plane bending needs to be eliminated or reduced. If the driving force is not eliminated, then the retrofit or repair is only a temporary procedure. The following sections describe methods for removing the out-of-plane bending force that drives the crack. *The one important note relevant to all web-gap fatigue retrofits is to maintain symmetry of the retrofit*. For example, if there is a connection plate on both sides of a girder web and a softening retrofit is to be used, then both connection plates must be cut back, not just one.

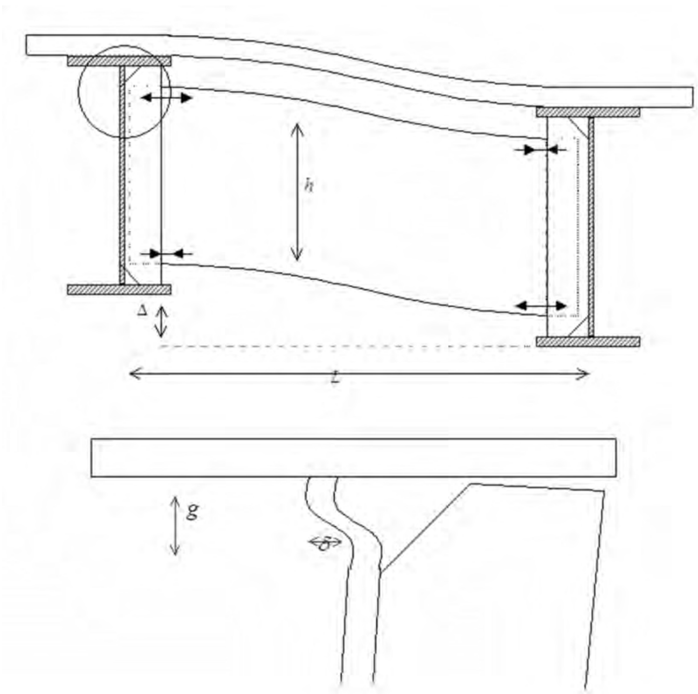


Figure 27. Illustration. Web-gap fatigue mechanism from displacement incompatibility. Top: Differential girder displacements cause a force couple to develop within the diaphragm. Bottom: Close-up view of web-gap deformation from differential girder displacement.

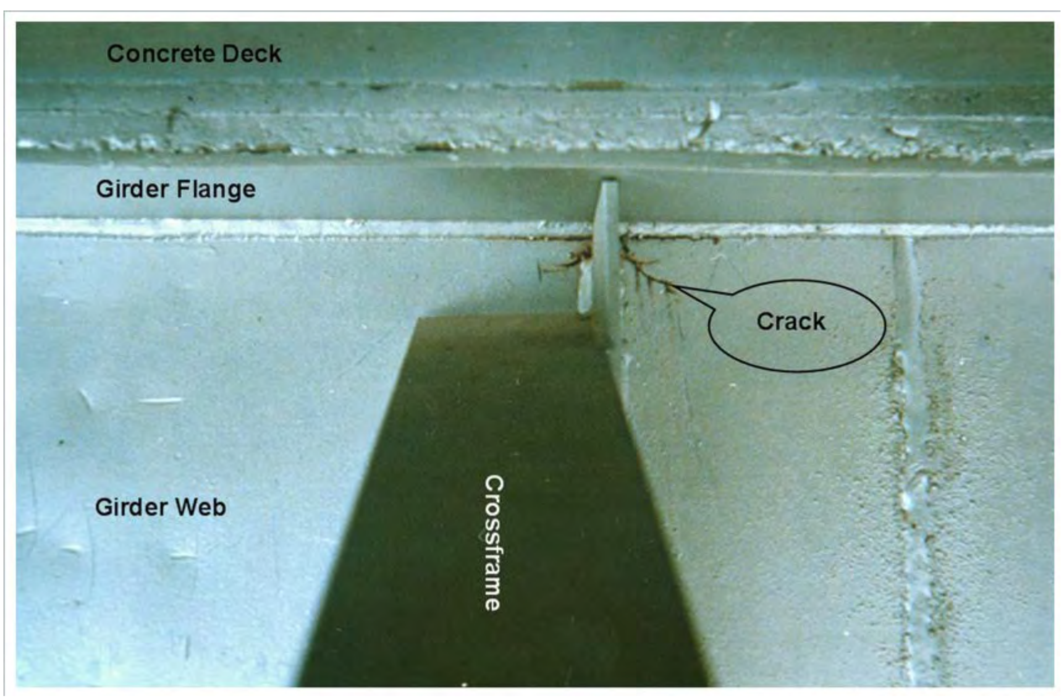


Figure 28. Photo. Typical horseshoe crack originating from web gap. Photo courtesy of Iowa Department of Transportation.

REPAIR METHODS SPECIFIC TO OUT-OF-PLANE DISTORTION

Hole Drilling

As explained in previous sections, hole drilling can be a very effective means of arresting crack growth. However, this method by itself may not be sufficient arrest the propagation. Research conducted at Lehigh University found that if the in-plane bending stress was less than 42 MPa (6 ksi) and the out-of-plane stress range was less than 105 MPa (15 ksi), then hole drilling is sufficient to eliminate the crack driving force.[(33)] It was also mentioned in the Fatigue Workshop that hole drilling *will not* work by itself for skewed bridges even if the aforementioned criterion is met, but may work for right angle bridges despite staggered or back-to-back diaphragms/crossframes. If any of the above criteria are not met, then drilled holes can be used to arrest crack growth, but *must* be used in conjunction with other methods to eliminate the crack driving force.

Many web-gap cracks have been blindly repaired/retrofitted with the hole drilling method, which will only work under special circumstances as described above. Some bridges have been retrofitted numerous times using this method, but the holes being drilled are just too small or the crack driving force is to high for the hole to work by itself. Therefore, cracks keep reinitiating from the previously drilled holes, leading to a "swiss-cheese" effect drilled in the vicinity of the web gap as shown in Figure 29.

Figure 29. Photo. Picture of multiple drilled holes in the vicinity of the web gap due to crack reinitiation from holes. Photo courtesy of Iowa Department of Transportation.

Diaphragm or Crossframe Removal

The majority of distortion-induced fatigue cracks in bridge girders are caused by secondary forces transferred by floor beams, diaphragms, or crossframes as one girder experiences more displacement than the one adjacent to it. One obvious technique to eliminate the crack driving force is to remove the diaphragm, or bracing system, that is causing the out-of-plane distortion. This can only be considered with a detailed analysis of the bridge system and the effects of such bracing removal and the adequacy of alternate load paths and girder strength and stability.

A few studies have determined the efficacy of such retrofits. One of these tests was performed on a three-span continuous bridge carrying westbound traffic on the I-20/59 interchange in Birmingham, Alabama.(72) All diaphragms in the positive moment region were removed, but the first line of diaphragms off the pier was kept as lateral-torsional bracing. The study found that the diaphragms only had a minimal role in the load distribution from one girder to another, and at most there was a 15% increase in bottom girder flange stress after removal. This was considered allowable due to the conservative design.

An analytical study of the IH-20 Midland County Bridges (east of Midland, Texas) examined the feasibility of selective diaphragm removal of this skew bridge.(73) The study found that bottom flange stresses increased 25%, and the loads in the remaining crossframes also increased, which was determined unacceptable. The slight difference between these two studies emphasizes the fact that diaphragm/crossframes should only be removed after rigorous analysis is performed with various truck and wind loadings, as well as consideration of stability conditions.

A couple of issues were raised during the discussion of this technique at the Fatigue Workshop:

- Most out-of-plane fatigue cracking occurs in the negative moment region where the top flange is restrained by the concrete deck and the connection plate is not welded to the top flange. In this case, it may be impossible to remove diaphragms because the bottom compression flange would be unbraced against lateral-torsional buckling. Before removing bracing in the negative moment region, the girders should be thoroughly checked for adequate unbraced length using AASHTO LRFD Bridge Design Specifications.(1)

- During the lifetime of the bridge, the concrete deck will have to be replaced. If the diaphragms are removed, no lateral bracing remains for the compression flanges of the girders after the deck is removed. This creates extra costs associated with deck replacement because temporary lateral bracing must be provided. It may not be cost-effective to remove bracing if deck replacement is scheduled within the next few years.

- The two DOT engineers who attended the Fatigue Workshop brought up practical reasons precluding the diaphragm removal procedure.(74, 75) The first reason is potential damage from bridge impacts. Many bridges are impacted by trucks each year, and the diaphragms distribute the load out to many girders and may minimize the amount of damage induced upon the whole bridge. The second concern of the DOT engineers dealt with record-keeping of diaphragm removal. If the state DOT begins an extensive program to remove diaphragms from bridges, there was a concern that accurate records may not be kept. Therefore, required temporary bracing may not be specified in the work plan during a deck replacement.

Diaphragm Repositioning

The approach spans of the I-35W bridge over the Mississippi river in Minneapolis, Minnesota is a typical multi-girder span with W-section diaphragms where the connection plate was not welded to the tension flange. The approach spans were experiencing web-gap cracking in the negative moment regions, and one girder suffered from near full-depth web cracking. The University of Minnesota performed a retrofit study for the Minnesota Department of Transportation. The retrofit specified removing the original rivets, lowering the diaphragms to a position as close as possible to the bottom flange, and only replacing four high-strength bolts into the existing holes as shown in Figure 30.

The intent of the retrofit was as follows: In the negative moment region, only the lower girder flange in compression needs to be supported. Therefore, the diaphragm only needs to brace the bottom compression flange, and could be repositioned further away from the top tension flange. Only four high-strength bolts were used in the retrofit because the diaphragm is only needed for bracing, not load distribution. This bridge was 12.2 m (40 ft) above a parking lot, so there were no concerns regarding future impacts that may require more bolts for load distribution. Removing the majority of the rivets significantly reduced the transfer stiffness of the diaphragm, reducing the out-of-plane displacements of the web gap. Load tests confirmed stress ranges were reduced by a factor of two after retrofitting one line of diaphragms.[(76)] Based on the results of trial retrofitting, the remaining diaphragm lines in the negative moment region were retrofitted. The real benefit of this retrofit was probably from removing most of the rivets, and not so much from the repositioning of the diaphragm. The next section will describe bolt loosening techniques, which may better describe this retrofit.

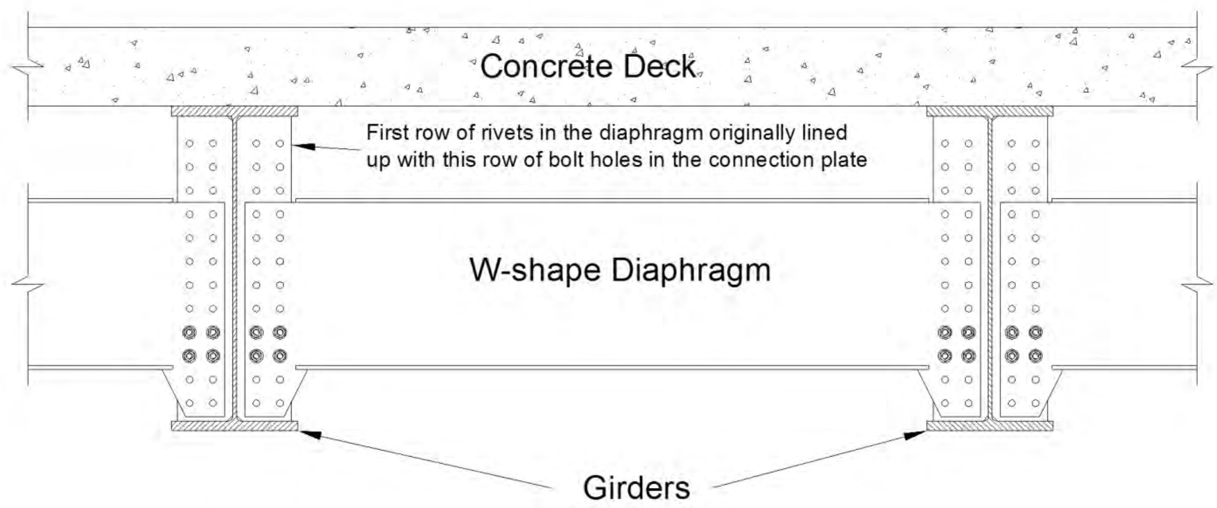


Figure 30. Illustration. Schematic of diaphragm repositioning retrofit specified on Minnesota Bridge No. 9340.

Bolt Loosening

An attractive retrofit is to loosen bolts that connect the bracing elements to the connection plates. Field tests conducted on bridges with out-of-plane distortion problems have measured the relative displacements between the connection plate and the girder flange associated with the distortion. Fortunately, the magnitude of the relative displacement between the girder flange and connection plate is small and lies in the approximate range of 0.03-0.76 mm (0.001-0.030 inch).[77] All holes in fabricated structural members are drilled or punched to a size larger than the fastener to be inserted into the hole. AISC defines a standard oversized hole to be 1.59-3.18 mm (0.063-0.125 inch) over the nominal diameter of the bolt.[43] Since the extra space provided inside the hole is more than a magnitude of the out-of-plane displacement, loosening the bolts can reduce out-of-plane displacements by eliminating the force transfer by the diaphragm/cross frame. The benefit of this retrofit is under service conditions, the diaphragm/crossframes are inactive (i.e. they are not engaged and causing out-of-plane distortion), but they are still present and can be activated under extreme events (e.g. truck collision).

Iowa State University in Ames, Iowa conducted experimental research on the bolt loosening retrofit on many Iowa bridges.[78, 79] Each used different types of bracing elements; I-beam diaphragm, channel-type diaphragm, X-type crossframe, and K-type crossframe. In most cases, the strains in or around the web gap were reduced by approximately 30-80% when the bolts were loosened, using known truck loads. The results could not conclusively say that bolt loosening is always effective for K-type crossframes, as web-gap strains sometimes increased. The bridge with X-type bracing was monitored long term, and the above results were found to be stable over extended time periods.

For most of the bracing schemes, multiple tests were run with all bolts tight, some bolts loose, and all bolts loose. Sometimes there was little additional strain reduction between the partially loose and all loose situations, indicating this retrofit can be highly variable. In all cases, all the bolts were loosened to a point where the bolt could easily move within the hole. A loose bolt is defined as one full turn of the nut after the bolt is loosened enough to allow a worker to turn the nut without using a wrench (i.e., backed off one turn from hand tight). Once the nut is loose, some mechanical method must be used to prevent the nut from turning off due to vibration from truck traffic. This can either be done with use of a jamb nut, hammering or peening the threads on the bolts, or placing a small tack weld at the end of the bolt. The Iowa DOT reported that in many cases the paint in the bolt threads is sufficient to prevent the nut from vibrating off.

In practice, because of fabrication tolerances and erection misalignments, the bolts may already be in bearing within the hole, so loosening the bolts may not provide enough flexibility within the system to yield any benefit. This retrofit must be validated on a case-by-case basis because the efficacy of this retrofit will be dependent upon how many bolts are loosened, and how loose each bolt becomes. It is recommended that bolts be sequentially loosened until the desired strain reduction is attained within the web gap. This could range from only having to loosen the bolts near the top girder flange for X-type bracing to having to loosen all bolts for an I-beam diaphragm, it is bridge and bracing scheme dependent.

Web-Gap Stiffening

Another viable retrofit for web-gap distortion is to stiffen the area by positively attaching the connection plate to the girder flange. The positive attachment either eliminates or greatly reduces the relative displacements between the connection plate and flange, thus lowering the out-of-plane bending stresses. This retrofit strategy has been found effective in systems not transferring a large moment to the connection plate, such as diaphragm or crossbracing systems. In cases where floor beams are framing into connection plates, the moment being transferred is too large to be eliminated by stiffening, and softening techniques should be used. Four types of positive connections are possible: all welded, all bolted, a hybrid of welds and bolts, and adhesives.

Welded Attachment

The most effective means of eliminating relative displacements between the girder flange and connection plate would be to weld the connection plate to the flanges. For many years, a stigma was associated with transverse welded attachments, especially to girder tension flanges. Current AASHTO codes now require transverse welds on both the tension and compression flanges for positive attachment of the connection plate.(1) Experience with this retrofit has not always been successful because the added weld has fatigue cracked itself, which could cause more problems than it solves.(81)

Although this seems to be an easy retrofit to specify, certain issues make this retrofit difficult to implement. First, high-strength steels are difficult to weld, so if the flange is made from A514 or

T1 steel, this retrofit might be undesirable. Second, most web-gap distortion cracking occurs near the flange that is embedded in concrete (typically in the tension flange of negative moment regions). The large mass of concrete causes it to act like a heat sink, which makes it difficult to establish the proper preheat and maintain interpass temperatures. For this reason, the all-welded retrofit was only specified in one location in the Poplar Street Complex shown in Figure 31, and other methods were used elsewhere on the bridge. This situation was particularly difficult because at the connection plate being welded was directly below a curb walkway with 406 mm (16 inches) of concrete above it. However, this problem may not be as severe if the flange is not embedded in concrete. Third, if there is a significant amount of dead load on the bridge, temporary shoring may be needed to counteract the effect of the girder softening under high flange temperatures, though these situations would be rare. At 600 °F (316 °C) the strength of the steel begins to decrease with an increase of temperature, hence excessive heat may create a reduced effective area to resist loads, but preheat temperatures are only around 300 °F (148 °C). However, this should be of little concern when welding to girder flanges embedded in concrete. Finally, if the connection plate is going to be welded while the bridge is in service, it may be difficult to achieve a decent root pass since the connection plate and girder flange will be moving relative to each other while traffic is on the bridge. It is recommended to shut down lanes of traffic that are directly over and adjacent to the girder being welded at least for the root pass.[56] Once an acceptable root pass is in place, welding can continue under full-traffic loading.

Since the weld is now resisting displacements, it needs to be sized to resist a horizontal load. On right (non-skewed) bridges, the weld should be designed to withstand a force of 20 kips (89 kN) as per AASHTO C6.6.1.3.1.[1] As the skew angle of the bridge increases, so should the value of this force, and the load should at least be doubled for highly skewed bridges. The weld should be as long as the connection plate is wide where it intersects the flange, but should not wrap around the connection plate, nor intersect the longitudinal flange-web weld. Figure 32 demonstrates the detailing of this fillet weld.

If the original connection plate was fabricated with a tight fit against the flange, the connection plate can be directly welded to the flange as described above. Often there is a slight gap between the connection plate and flange. In these situations, Figure 33 offers detailing options using additional plates or spacers. Care should be given that the width of the spacer block not be wider (direction in-line with primary bending stress of girder) than 1.5 in. (38.1 mm). Widths greater than this would make the detail less than AASHTO Category C, which is the category of a connection plate-to-girder weld. In situations where the retrofit is to weld a tight-fit connection plate, it may be advantageous to cut back a small portion of the connection plate and use an additional plate as in Figure 33. Contaminants and paint may be present in this tight-fit area, which could adversely affect the weld quality. Removing a small portion of the connection plate allows for the welded area to be cleaned and ground. A generic all-welded retrofit procedure is outlined in Figure 34.

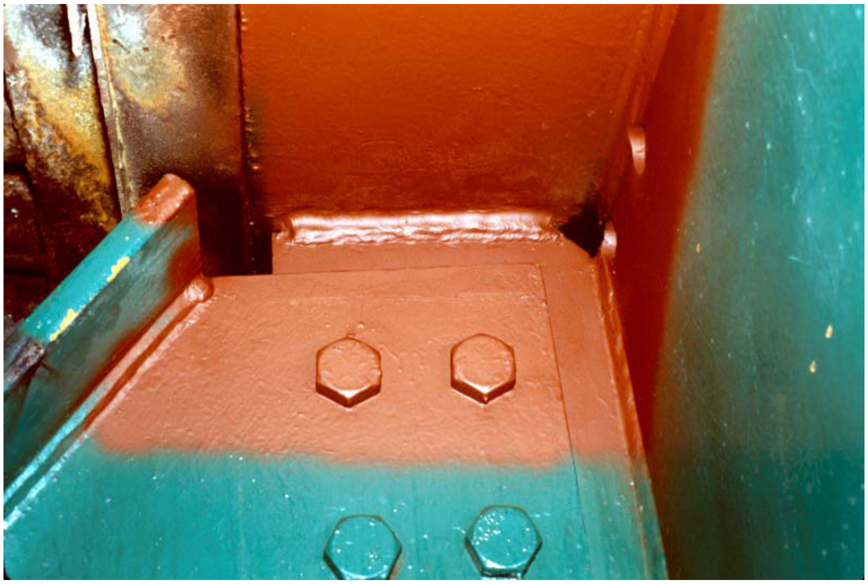


Figure 31. Photo. Retrofitted connection plate detail welded directly to girder flange, Poplar Street Complex in East St. Louis, Missouri. Still in service 20 years after retrofitting [82] Photo courtesy of Wiss, Janney, Elstner Associates.

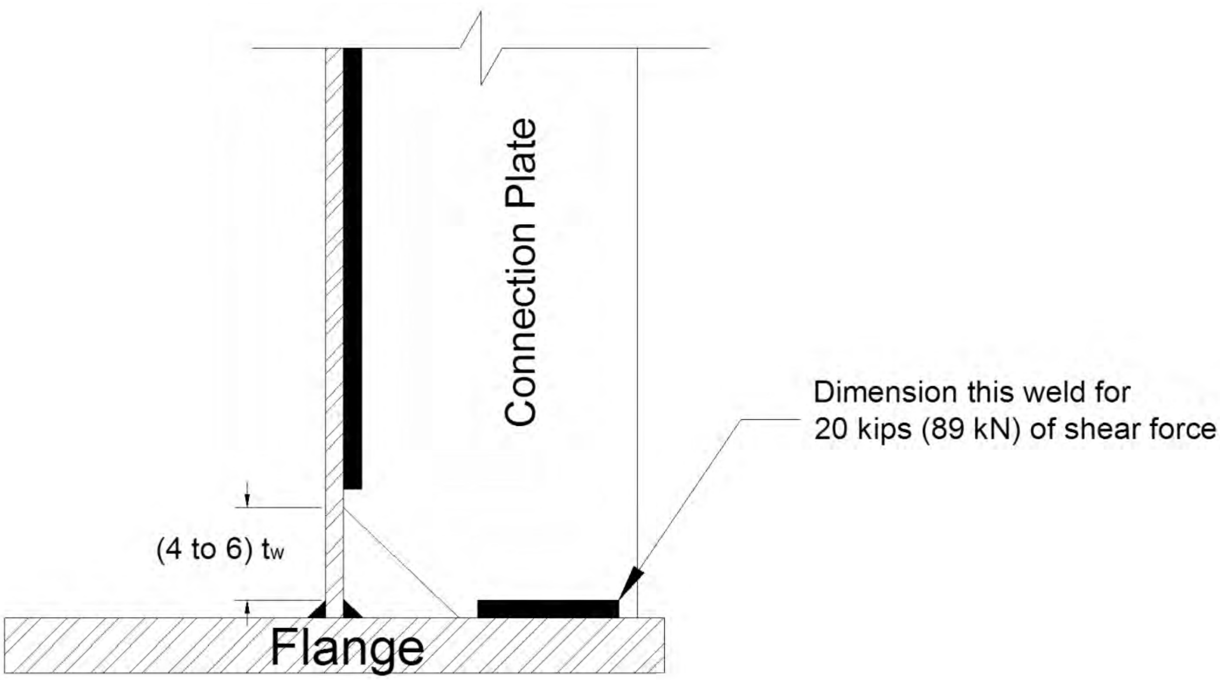


Figure 32. Illustration. Connection plate-to-girder flange fillet weld detailing. Reproduced from Keating.[83]

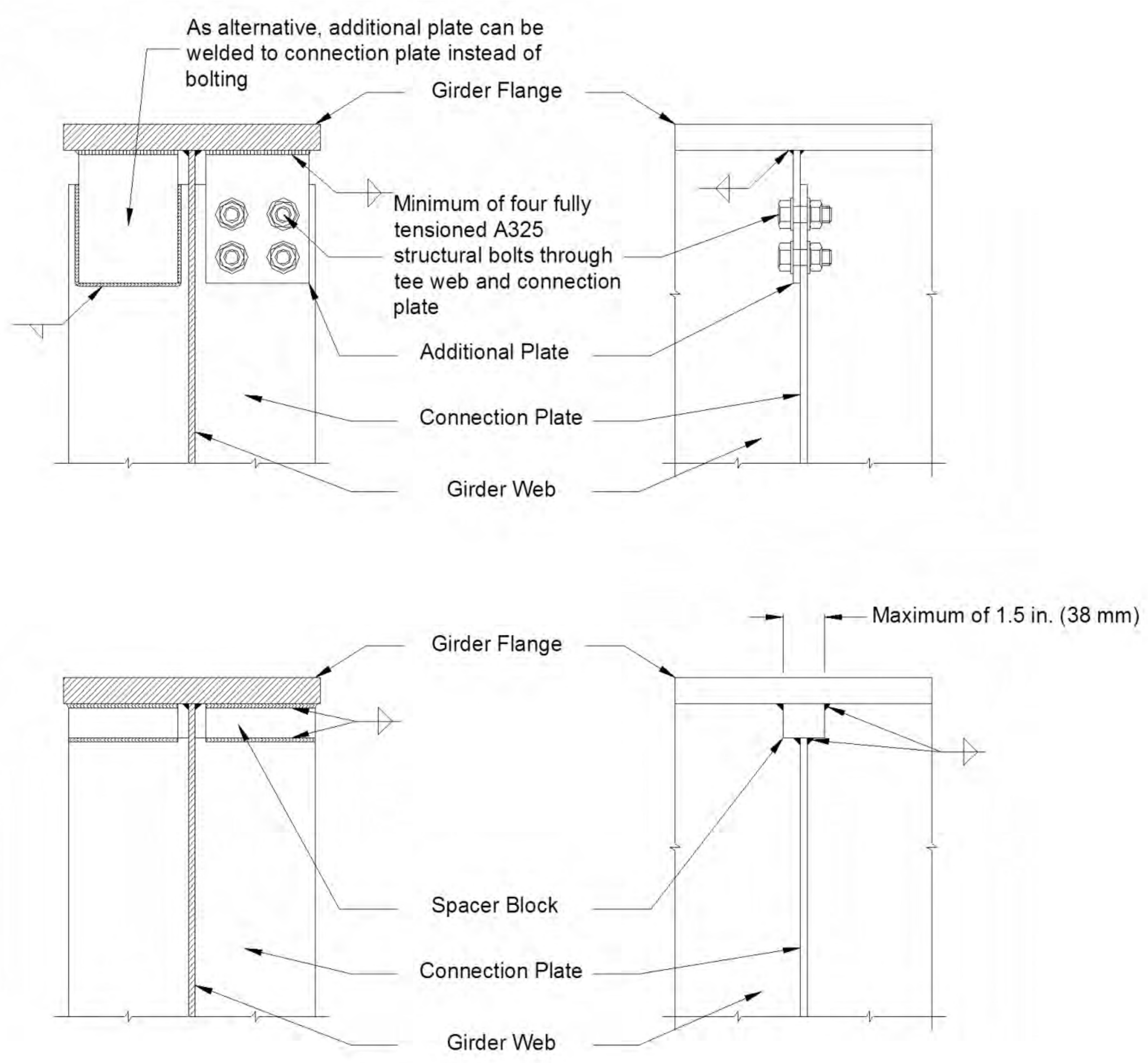
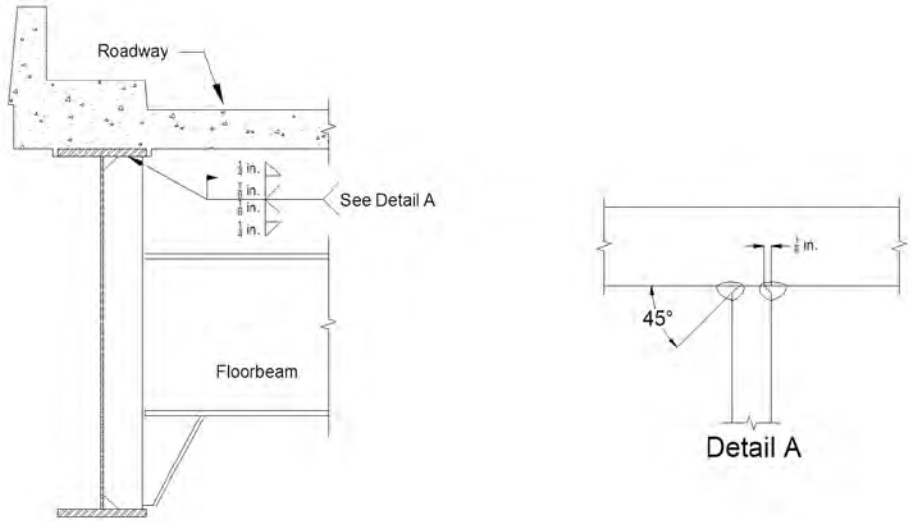


Figure 33. Illustration. Schematic of welded connection plate-to-girder flange retrofit used a lap plate or spacer block. Reproduced from Keating.[83]



Procedure

1. UT girder top flange in the vicinity of the connection plate to check for plate laminations and pre-existing cracks. If laminations or cracks are found, then flange cannot be welded.

2. Grind off paint from girder flange in an area of 2.0 in. (50.8 mm) on each side of the connection plate. Repeat procedure to remove 2.0 in. (50.8 mm) of paint from connection plate too.

3. Grind the end of the connection plate to obtain an approximate $\frac{1}{8}$ in. (3.2 mm) bevel as shown in Detail A. Doing this gives more area of weld along the connection plate.

4. Since most connection plates are milled to bear, contaminants and paint can build up in the tight fit area between the top flange and connection plate. These contaminants can affect weld quality and should be removed if possible. One method found to work in the field is to spray the tight fit area with a lubricant and heat area to minimum preheat temperature. This idea comes from field experience, and spray lubricant was found to be most effective in burning off the contaminants.

5. Eliminate or significantly slow traffic from roadway at location to be welded. Install small fillet welds in the ground grooves of the connection plate using SMAW low hydrogen E7018 electrodes. Once the root pass is in place, the weld should be ground smooth and inspected for any cracks that may have occurred due to welding under fluctuating loads. If the root weld passes inspection, subsequent welds maybe placed. Minimum preheat and interpass temperatures shall be followed in strict accordance with AWS D1.5 requirements. Preheat girder flange to a minimum of 250ºF (120ºC) prior to welding and maintain a minimum 225ºF (107ºC) interpass temperature. All welding should only be performed by a certified welder.

6. Once the weld is sufficiently cooled, remove traffic restrictions.

7. Reinspect weld with non-destructive techniques to ensure the weld has not cracked.

8. Hammer peen weld toe if desired.

9. Make sure surfaces are clean and apply proper surface coatings.

Figure 34. Illustration. Work plan for all welded retrofit. Reproduced from Koob et. al.[82]

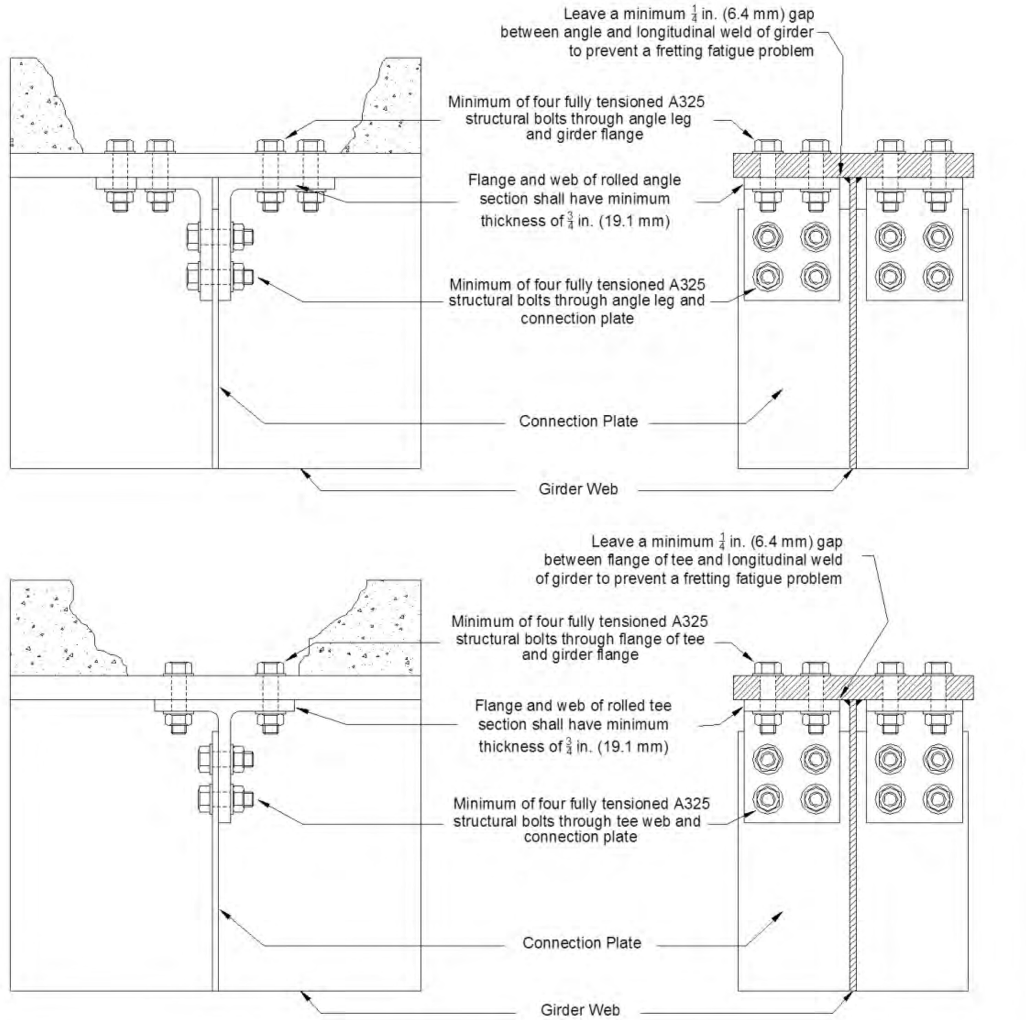
Bolted Connections

Instead of welding to the flange, other types of connections can be used to provide more rigidity to the web-gap area, without the use of field welding. These would involve bolted connections using angles, or tee sections, between the connection plate and the girder flange. This general concept can be seen in Figure 35. The retrofit can only be implemented if net section fracture does not control the strength of the girder since holes will have to be drilled into the girder flange.

The stiffness of these connections is very important to reducing out-of-plane bending, and no bolted connection will ever be able to completely "lock-up" all out-of-plane displacements as well as a welded detail. At a minimum, when using a tee section, at least two bolts are needed transversely across the flange, and on each side of the connection plate, for a minimum of four bolts total into the girder flange. If two angles are going to be specified, then four bolts are required through each angle into the girder flange. A minimum of four bolts should be used to connect the angles or tee stems to the connection plate, placed in two rows and two columns. The legs of angles and flanges/stems of tees should have a minimum thickness of 0.75 in. (19.1 mm) because smaller sections have been found to be too flexible.(33)

The bolted tee connection is preferred over the double-angle approach because it provides more stiffness. The tee should be rolled, not built-up from two plates welded together. In one Iowa DOT experience, they specified a tee stiffening retrofit but gave the contractor discretion in using rolled or built-up tees. The contractor used welded, built-up tees, which cracked.(74) If the connection plate was originally fabricated with a tight fit to the girder flange, it will need to be cut back to allow the flange of the tee section to be slid into place.

Difficulty with bolted connections occurs when the girder flange is embedded into concrete (typically this is going to be the tension flange in the negative moment region) because the concrete deck interferes with bolt installation. A portion of the deck may have to be removed to allow for the bolt installation as shown in Figure 35. However, if the concrete deck is haunched, bolt installation may be possible with only removing the haunched portion without full deck excavation as depicted in Figure 36. This would prevent traffic disruption.



Procedure

1. Using appropriate traffic-control measures, remove portion of concrete deck directly above installation location.

2. If a tee connection is to be used, flame cut away portion of connection plate to allow for clearance of the tee flange. Care should be taken not to gouge the web and flange of the girder during the flame-cutting procedure. Use a die grinder to remove the remnants of the weld flush to the web.

3. If the web has fatigue cracks, idendify the crack tips with magnetic particle or dye pentrant tests. Drill 1.0 in. (25.4 mm) diameter holes at the crack tips.

4. Sandblast the areas that will be covered by the angles or tee to meet SSPC-SP10.

5. Position prefabricated angles or tees and mark hole centers using a transfer punch on both flange and connection plate.

6. Drill four 1-$\frac{1}{16}$ in. (27.0 mm) holes into girder flange.

7. Drill four 1-$\frac{1}{16}$ in. (27.0 mm) holes into connection plate.

8. Thoroughly clean all bearing surfaces on girder flange, connection plate, and angles/tees to remove remnants of cutting fluids.

9. Install angles and place 1.0 in. (25.4 mm) bolts through connection plate and girder flange. Tighten all bolts to minimum preload by turn of the nut method.

10. Apply proper surface coatings to bare steel, and replace removed portion of concrete deck.

Figure 35. Illustration. Work plan of bolted double angle and tee connection to stiffen web gap. Adapted from Keating.[81]

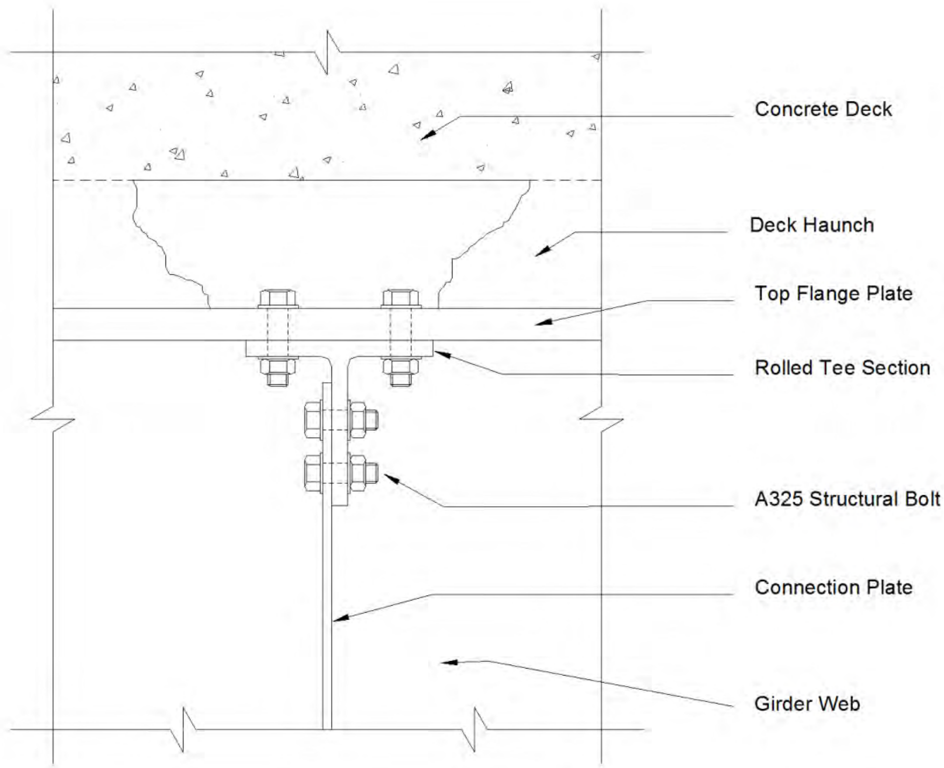


Figure 36. Illustration. Schematic of concrete deck haunch removal to allow for bolt installation. Taken from Keating.[(83)]

If partial deck removal is not an option, it is possible to drill and tap holes into the girder flange, as shown in Figure 37. This may not always be possible if the girder flange it too thin. It is recommended that the finished tapped hole give the bolt or stud a minimum of 1.0 in. (25.4 mm) embedment depth. It may be difficult to find short enough bolts to fit only through the angle and girder flange or tee and girder flange, without hitting the concrete slab above. However, with more preparation a hammer drill could be used to continue the hole into the concrete after the hole is drilled in the steel. The more attractive solution shown in Figure 37 is to use high-strength threaded studs with nuts and washers. This way the shorter length is more easily attainable, and tightening is easier on a nut rather than the head of a short bolt. The only other problem with the tapped hole is full-tensioning of the bolt may not be possible because of lower strength flange material. This retrofit was specified on the Poplar Street Bridge Complex in East St. Louis, Missouri and also the Neville Island bridges carrying I-79 near Pittsburgh, Pennsylvania.[(82)] Prior to field implementation, experiments were conducted on the pull-out strength of A325 bolts in A36 grade steel. It was determined that the correct embedment length was 1.25 in. (31.8 mm). However, based on the results shown in Table 2, it is recommended that 1.0 in. (25.4 mm) be considered the minimum depth. If flange thickness allows it, the depth should be increased.

Implementation of this procedure is the same as outlined above, however the procedure outlined in Figure 35 must be modified with the following:

6. Drill 0.875 in. (22.2 mm) holes into the girder flange (this operation will require the use of high-speed twist bits, so all the material in the hole is removed). Follow drilling operation by tapping the holes with 1-8 UNC bottoming tap, making sure minimum thread embedment is followed. Install A325 threaded studs after first applying Locktite compound (or equivalent) to prevent loosening of the stud from the girder.

Table 2 – 1.0 in. (25.4 mm) Bolt Embedment Length Pull-Out Tests of A325 Bolts From A36 Steel. Reproduced from Koob et. al.[82]

Test Number	Thread Embedment (in. / mm)	Maximum Load (kips / kN)	Failure Mode
1	0.25 /6.35	7 / 31	Thread Slippage
2	0.50 / 12.70	28 / 125	Thread Slippage
3	0.50 / 12.70	35 / 156	Thread Slippage
4	0.75 / 19.1	59 / 262	Thread Slippage
5	0.75 / 19.1	64 / 285	Thread Slippage
6	0.75 / 19.1	67 / 298	Thread Slippage
7	1.00 / 25.4	85 / 378	Thread Slippage
8	1.00 / 25.4	88 / 391	Bolt Fracture
9	1.00 / 25.4	86 / 382	Thread Slippage
10	1.00 / 25.4	87 / 387	Bolt Fracture
11	1.25 / 31.75	89 / 396	Bolt Fracture
12	1.25 / 31.75	90 / 400	Bolt Fracture
13	1.25 / 31.75	88 / 391	Bolt Fracture
14	1.25 / 31.75	83 / 369	Bolt Fracture

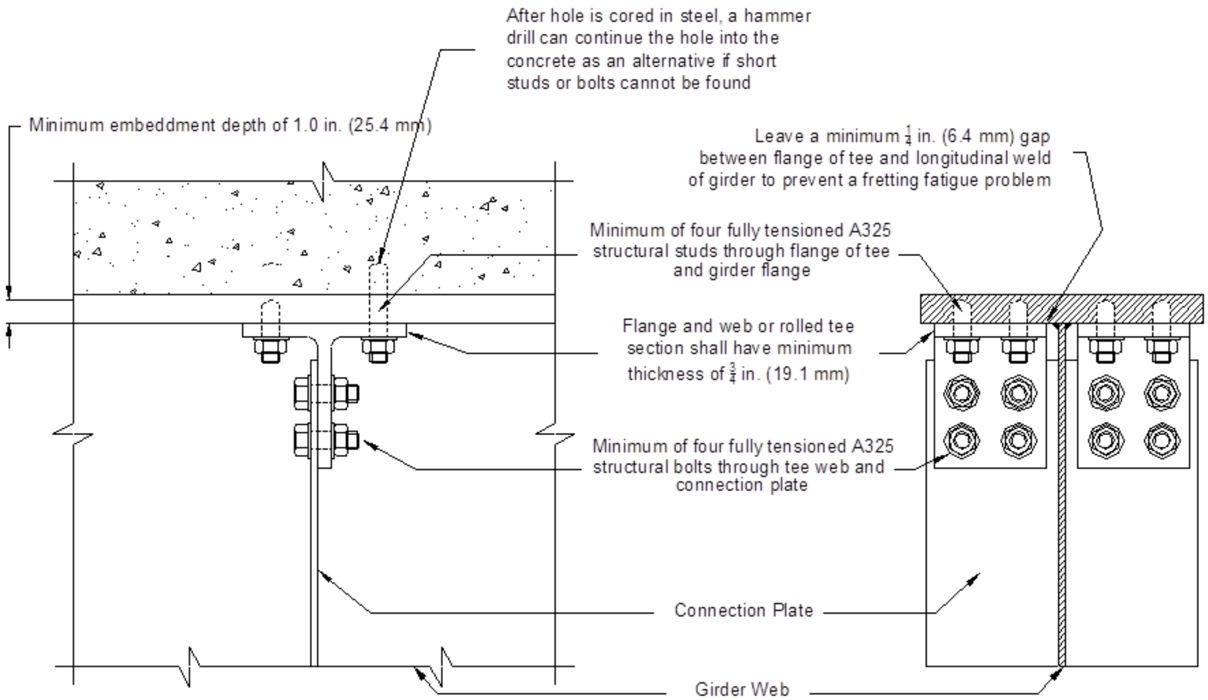


Figure 37. Illustration. Schematic of drilled and tapped holes to connect the angles or tees to the girder flange. Adapted from References 82 and 83.

Hybrid Connections

The third type of connection is a hybrid using both field welding and bolting. These types of connections may become more attractive if clearance issues become a problem with all-bolted connections. A typical hybrid would involve bolting a thick plate to the girder flange, but then welding the connection plate to the thick plate as shown in Figure 38. The plate should have a minimum thickness of 22.2 mm (0.875 inch) and be bolted to the girder flange with at least four bolts in the same pattern described in the *Bolted Connections* section above. The weld between the connection plate and the bolted plate needs to be sized according to the provisions set forth in the *Welded Connections* section above. The theory behind this connection is if the weld is going to fatigue, then the crack is contained and cannot directly grow into the girder.

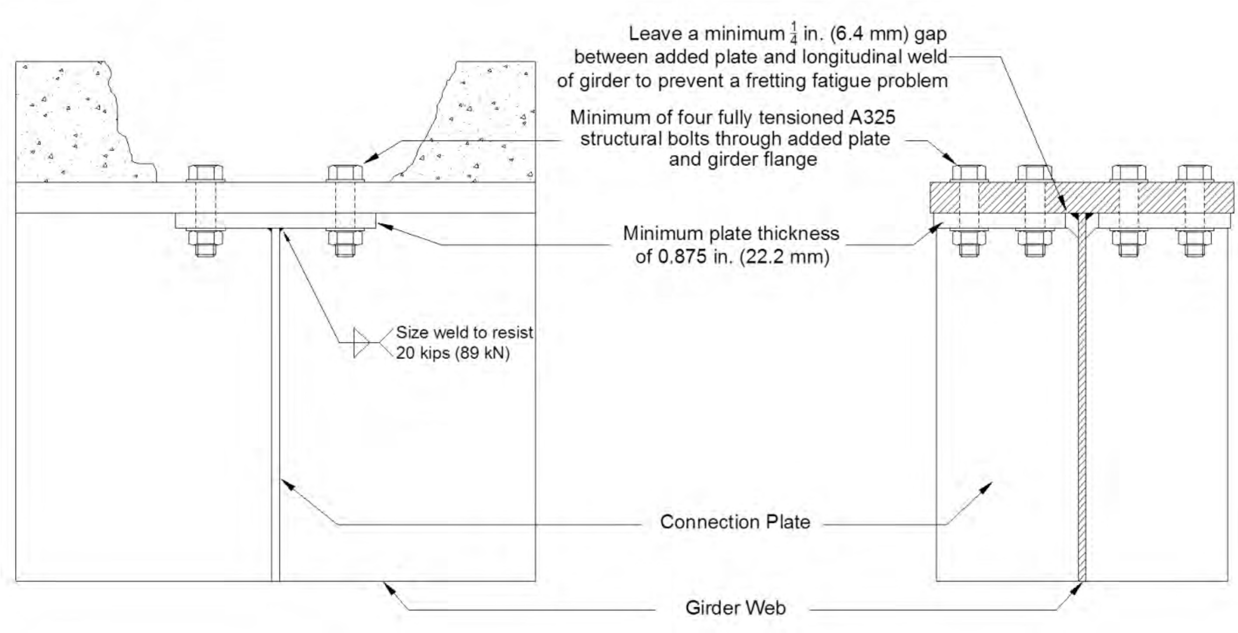


Figure 38. Illustration. Typical hybrid connection with bolts through the flange, but connection plate is welded. Adapted from Reference 83

Adhesives

The conundrum with the bolted stiffening retrofit is the concrete deck must often be excavated so bolts can be provided to attach the angle or tee to the embedded girder flange. Partial removal of the deck is expensive and requires traffic control, making this retrofit unattractive. One innovative idea is to glue the angles or tees to the girder flange so the concrete deck will not have to be excavated to provide positive attachment. However, because the long-term effect of steel-to-steel adhesives is not truly known, this retrofit is not meant to be a permanent fix without further research. There has been some long-term monitoring of steel-to-steel adhesives that found the adhesive strength decreased less than 5% and the stiffness less than 10% over a 271 day period in an outdoor Minnesota environment from February to August.[84] This is not a very long period, but it would suggest that the adhesive retrofit should be serviceably for at least a one year period. When the deck is scheduled for replacement, holes could then be drilled into the girder flange and connection angles or tee to make a bolted connection. A proper adhesive must meet three requirements for this retrofit work:

- Field applicability. The adhesive must be capable of room temperature curing and be viscous enough to deal with the surface irregularities of the bridge and retrofit angle while also being able to maintain a vertical position during the curing process.
- Substrate compatibility. The adhesive must be able to bond steel to steel.
- Environmentally durable.

This retrofit has been studied with two bridges on the I-494 Interchange in the southern suburbs of Minneapolis, Minnesota.(84) Both bridges had an approximate skew angle of 50 degrees. Both bridges had unstiffened web gaps. One had bent plate diaphragms, while the second utilized cross-frames. The field retrofit used the adhesives to connect angles to both the connection plate and the girder; however, a small stainless steel bolt was inserted through the angles and connection plate. A stainless steel bolt was used for corrosion purposes only. The role of the small bolt was to add redundancy in the event of adhesive failure to prevent the angle from falling into traffic below. Field testing found stress ranges within the web gap decreased by 40% with the glued angle in place, which should yield a four and a half times increase in fatigue life. A general procedure for attaching angles with field adhesives can be found in Figure 40.

Lab studies found that gluing one angle to the connection plate and girder web, as shown in Figure 39, ceased crack growth (crack tips were not drilled out) for about 500,000 cycles, and then started to grow again, at an in-plane stress range of 12.0 ksi (82.7 MPa).(84) This phenomena was thought to be the glue layer delaminating under the high in-plane bending stress. Adhesives are strong in shear, but are susceptible to peeling forces. Since the web distortion does not subject the angles to pure shear (there is some bending), the adhesive layer is susceptible to failure from peeling forces. To verify the delamination theory, the angles were bonded with new adhesive and crack growth completely ceased (again, stop holes were not drilled, though one is shown in Figure 39) when the in-plane stress range was reduced to 4.0 ksi (27.6 MPa). Note that the experimental retrofit tested the lower bound from recommendations set forth by this manual as only one 0.75 in. (19.1 mm) thick angle section was used to stiffen the web gap.

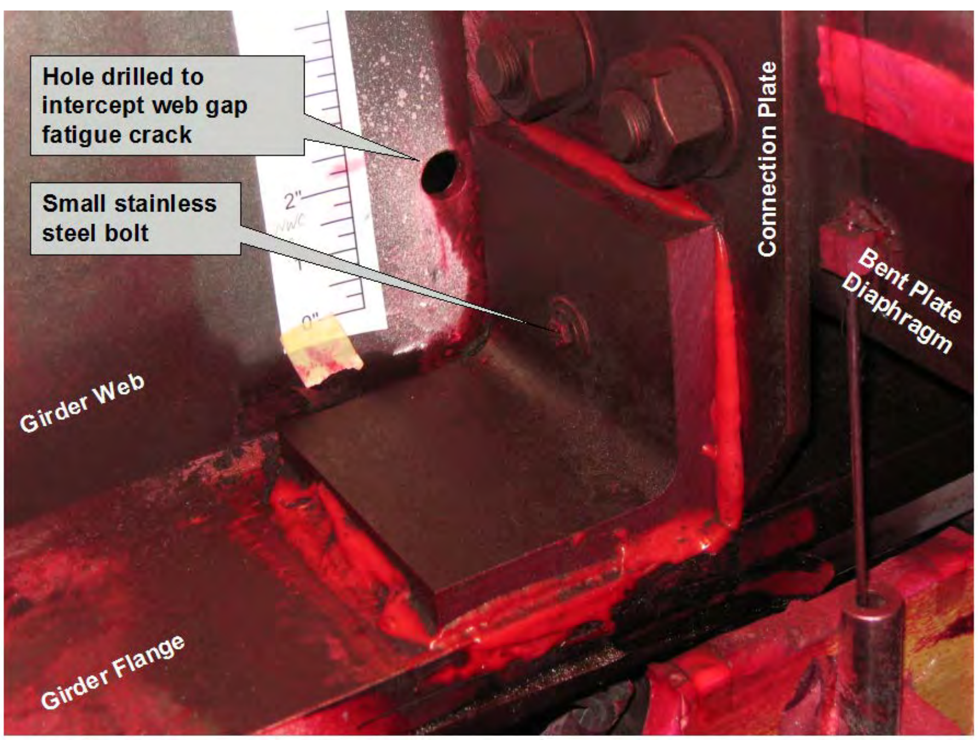
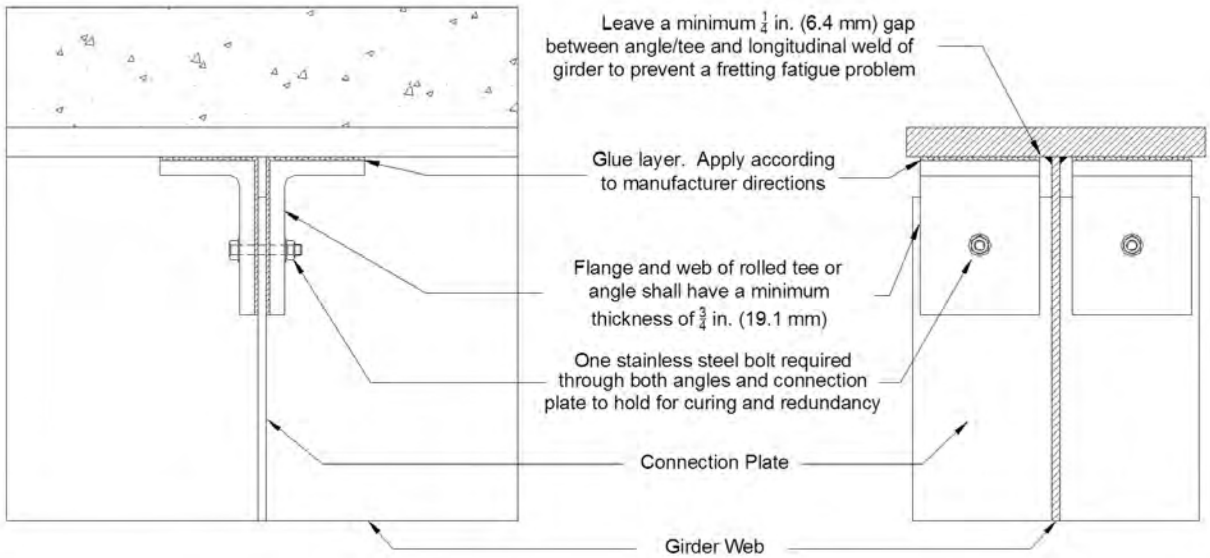


Figure 39. Photo. Glued angle from experimental testing at University of Minnesota.



Procedure

1. If a tee connection is to be used, thermally cut away portion of connection plate to allow for clearance of the tee flange, if neccessary. Care shall be taken not to gouge the web and flange of the girder during the flame-cutting procedure. Use a die grinder to remove the remnants of the weld flush to the web.

2. If the web has fatigue cracks, idendify the crack tips with magnetic particle or dye penetrant tests. Drill 1.0 inch (25.4 mm) diameter holes at the crack tips.

3. Grind or blast away all paint, primer, and corrosion and expose bare steel in all areas to be covered by angle/tee.

4. Position prefabricated angles or tee and mark hole center on connection plate for stainless steel bolt to pass through. Drill hole through connection plate.

5. Thoroughly clean all gluing surfaces with acetone (or by manufacturers recommendation) to remove surface contaminants. Keep wiping the surface with clean white, acetone-soaked towels until no residue is observed on the towels.

6. Mix glue according to manufacturer's directions. Sparingly sprinkle glass beads atop glue layers to create a uniform bonding surface. Position the angles/tee and tighten stainless steel bolt until the glass beads can first be heard breaking.

7. Apply proper surface coatings to bare steel.

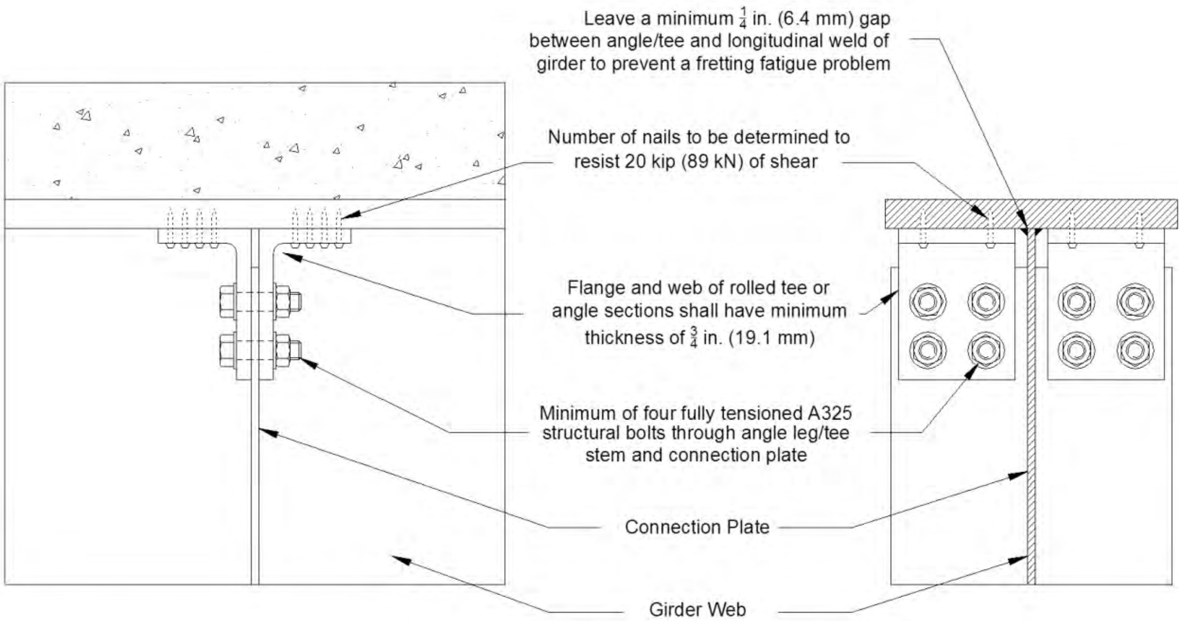
Figure 40. Illustration. Work plan for stiffening retrofit of web gaps with adhesives. Adapted from Hu.[84]

Nails

By far the newest, and quite possibly the best alternative for stiffening the web gap is powder-actuated fasteners to attach the angle or tee to the girder flange. Powder-actuated fasteners are high-strength nails that are embedded into other materials, propelled by an explosive discharge. This retrofit offers a positive and permanent connection that can easily be performed under traffic, without the need to excavate the concrete deck. Extensive fatigue testing has been done for base metal with embedded nails. Testing was completed on base metal ranging in thickness from 0.20-1.97 in. (5-50 mm) and included the effects of imperfections from incorrectly shot nails. The lower bound of the data, conducted at high load ratios, had an EC3 Class 90 fatigue categorization, which correlates to the AASHTO Category C curve.(85) Since connection plates welded to girder flanges are also Category C, there appears to be no fatigue issues by shooting nails into girder flanges. However, in the base metal tests published by Niessner and Seeger, cracking always initiated at the entry point of the nail, never from the tip of the nail.(85) For distortion-induced fatigue cracking in negative moment regions, the tip of the nail will be embedded at a higher stressed location than the entry point of the nail (because the linear strain gradient is a maximum at the top of the flange) thus there are some concerns whether this may reduce the published fatigue strength from tension coupon tests.

The nailed retrofit has been experimentally tested, but not to failure. The unretrofitted web gap originally had an in-plane stress in the flange of 13.0 ksi (89.6 MPa), and an out-of-plane bending stress that varied by location from 14.0 ksi (96.5 MPa) to 30.0 ksi (206.8 MPa). The nailed retrofit was used after the specimen had developed out-of-plane fatigue cracks in three different connection plate locations. Cycling continued with the nailed retrofit and at an in-plane stress of 12.0 ksi (82.7 MPa), one retrofit accumulated an additional 451,000 cycles and another 682,000 cycles. The third retrofit was in a lower moment region and lasted an additional 2,000,000 cycles at an in-plane stress range of 4.0 ksi (27.6 MPa).(84) Plotting these cycles counts using the in-plane stresses against AASHTO fatigue curves shows at best, the retrofits attained Category E resistance. Though it was not brought to failure, it gives a benchmark performance.

In Figure 32 it was recommended that stiffening retrofits be designed to resist a minimum out-of-plane force of 20 kips (89 kN). This would require approximately 20-30 nails to be shot into the girder flange through the legs of the two angles, or flange of the tee section. However, the manufacturer's recommend nail shear resistance should be used to formally determine the required amount of nails. Nailing through the connection plate is probably possible when using a tee stiffening retrofit, but as the nail punches through the connection plate on an angle stiffening retrofit, it may misalign the angle on the opposing side. Therefore, there is no benefit to nailing angles to the connection plate, and it should be conventionally bolted as shown in Figure 35. A generic retrofit procedure is shown in Figure 41, and a completed connection plate is seen in Figure 42.



Procedure

1. If a tee connection is to be used, flame cut away a portion of the connection plate to allow for clearance of the tee flange, if neccessary. Care shall be taken not to gouge the web and flange of the girder during the flame-cutting procedure. Use a die grinder to remove the remnants of the weld flush to the web.

2. If the web has fatigue cracks, identify the crack tips with magnetic particle or dye penetrant tests. Drill 1.0 in. (25.4 mm) diameter holes at the crack tips.

3. Sandblast the areas that will be covered by the angles or tee to meet SSPC-SP10.

4. Position prefabricated angles or tees and mark hole centers on connection plate.

5. Drill four 1-$\frac{1}{16}$ in. (27 mm) holes into connection plate.

6. Thoroughly clean all bearing surfaces on girder flange, connection plate, and angles/tees to remove remnants of cutting fluids.

7. Position angles and place bolts through connection plate. Tension all nuts by turn of the nut method.

8. Shoot appropriate number of powder-actuated fasteners through stiffening element into girder flange.

9. Apply proper surface coatings to bare steel.

Figure 41. Illustration. Work plan for web gap retrofit using nails.

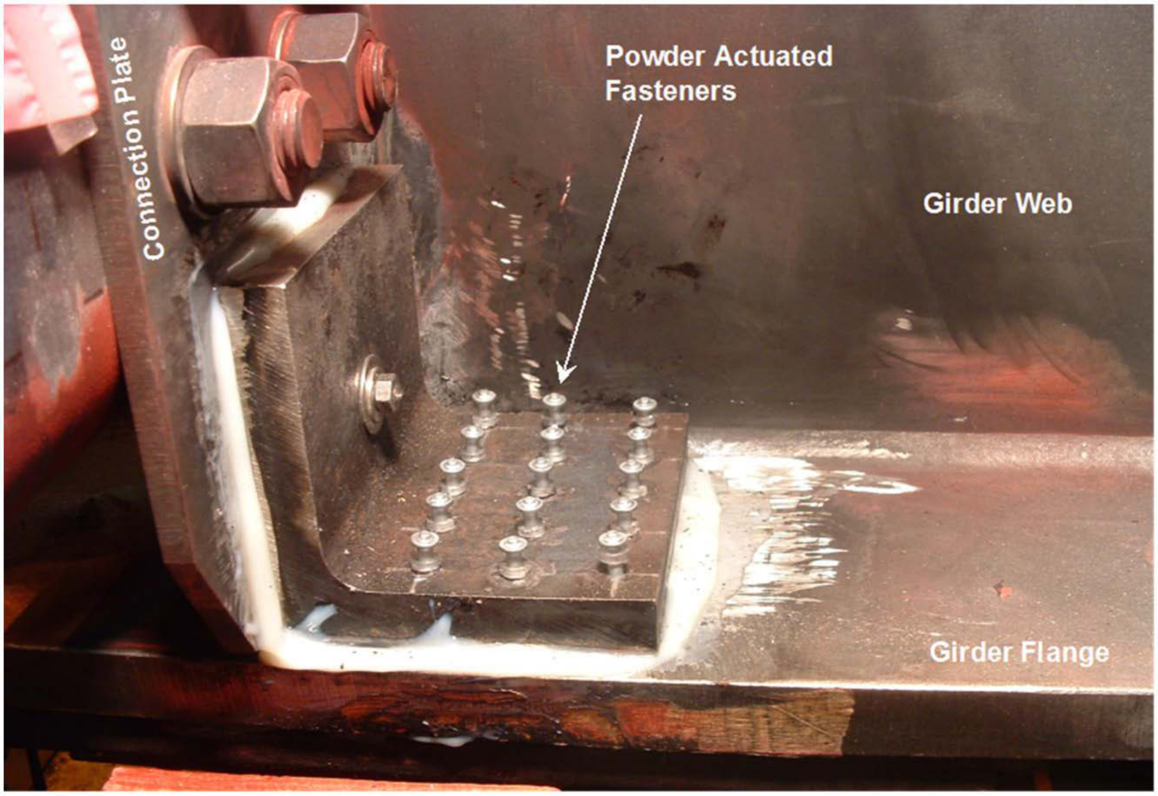


Figure 42. Photo. Completed nailed angle stiffening retrofit.

Web Gap Softening

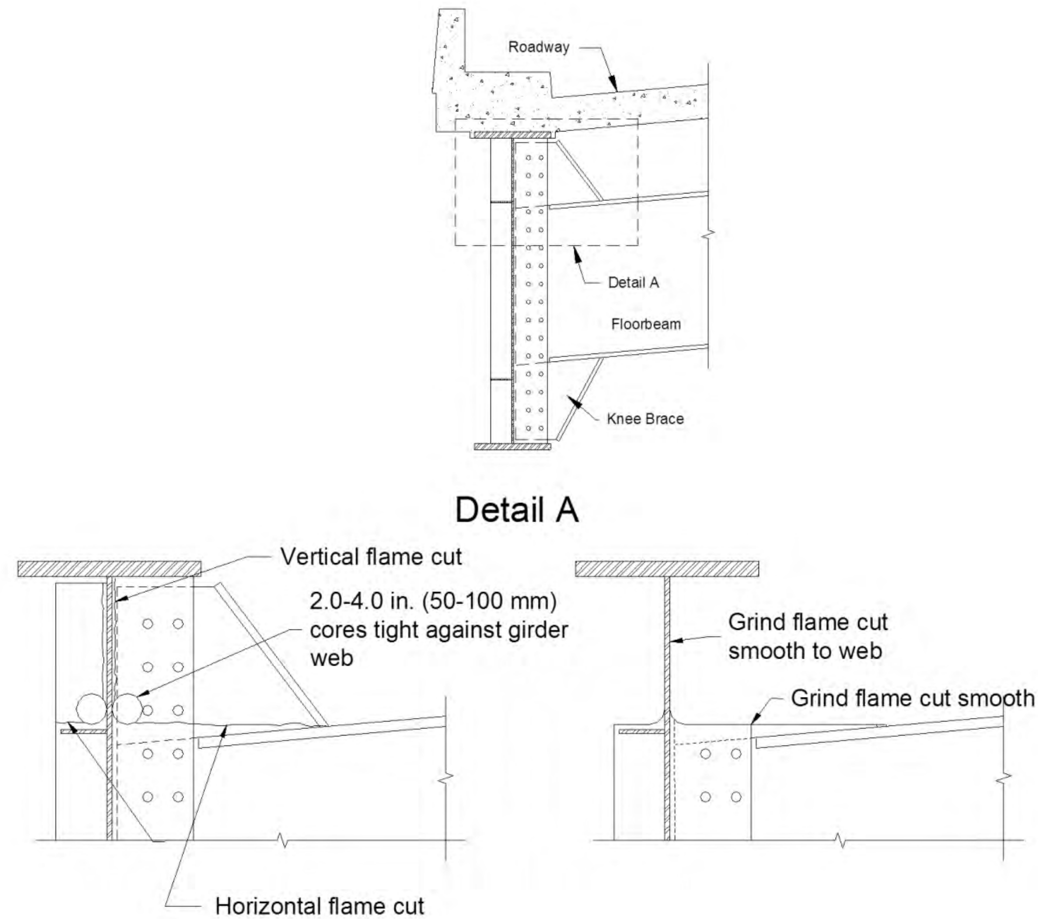
The second option for direct retrofitting of web-gap fatigue is to soften the area by cutting away material to make the web gap more flexible. This is typically done by cutting out a portion of the connection plate, or using a large diameter hole, each of which will be discussed in more detail.

Gross Material Removal

The first method of softening the web gap is to cut away a portion of the connection plate. Removing a piece of the connection plate drastically increases the web-gap length, which in turn reduces the stress due to the out-of-plane distortion. It is recommended that the length of new gap be one-sixth the web depth but no greater than 15.0 in. (381 mm). This retrofit is recommended for deep floor beams, which have high rotational rigidity and are transferring a large moment to the web, but is not limited to this case only. Figure 43 shows the detailing used in the Poplar Street Bridge Complex in East St. Louis, Missouri, to soften the web gaps. Figure 44 shows a picture of the detail after the retrofit has been completed.

It is imperative that the portion of the connection plate left attached to the web after flame cutting be *ground smooth flush with the web (i.e. remove all residual weld material from the web plate)*, and not just ground smooth (i.e. only smoothing the jagged edges left from flame cutting).

The softening retrofit was used on the Lexington Avenue Bridge (I-35E over the Mississippi River) in Southwest St. Paul, Minnesota; however, the contractor said it was too difficult to completely grind the leftover flush with the web. Shortly after the retrofit was completed, fatigue cracks were found to reinitiate, and stop holes were eventually used to core out the original weld terminations of the connection plate as shown in Figure 45.



Procedure

1. Drill a minimum 2.0 in. (50 mm) diameter hole into the connection plate so the hole is tangent to the girder web and the new web gap length is one-sixth the girder web depth, but not greater than 15.0 in. (381 mm). Remove all bolts that will interfere with the coring operation.

2. Make vertical flame cuts through connection plates on both sides of the web. Cut the connection plates far enough away from the web to avoid gouging the web plate. Utmost care should be taken such that the girder web and top flange are not gouged by the torch. Temperature of the web plate shall not exceed 800ºF (425ºC).

3. Make horizontal flame cuts through both connection plates beginning tangent to the drilled hole and out to the extremity of the connection plate. Cut off portion of knee brace if applicable.

4. Grind the excess material left on the girder web smooth, making sure not to gouge the girder web. Make sure all transitions are ground smooth. All ground surfaces should have a ANSI roughness of 250 or greater.

Figure 43. Illustration. Work plan for web gap softening used on Poplar Street Bridges in East St. Louis, Missouri. Taken from Koob et. al. [82]

Figure 44. Photo. Completed web gap softening retrofit on Poplar Street Complex. Photo courtesy of Wiss, Janney, Elstner Associates.(82)

Figure 45. Photo. Web softening technique of Lexington Avenue Bridge. Photo courtesy of Minnesota Department of Transportation.

Large Hole Retrofit

Instead of cutting a large portion of the connection plate away (which is time-consuming because of extensive grinding and not possible for X-type bracing schemes), large diameter holes can be drilled close to the web-gap area, as opposed to small ones as described in the beginning of this chapter. This retrofit is quicker and easier than bolting or cutting away the connection plate. The large hole also has the ability to remove, intercept, or capture multiple cracks or holes in the vicinity of the web gap, and the large radius prevents crack reinitiation but also softens the web gap by removing a lot of material. A large hole is defined as a diameter equal to or greater than 3.0 in. (76 mm). This type of detail can be seen in Figure 46, and has largely been implemented on bridges in Missouri, Iowa, Illinois, and California.[77,82,86] Iowa has specifically used the retrofit to address the "swiss-cheese" situation shown in Figure 29 to remove multiple holes and crack tips with 3.0-4.0 in. (76-101 mm) holes. This retrofit is most beneficial when cracking is occurring in bearing stiffeners, since they are a reaction point they cannot be cut away to soften the web gap and cannot be stiffened since they are already fully welded to the flange. The hole should be cut on both sides of the connection plate so that it intercepts all welds by a minimum of 0.125 in. (3.2 mm). Figure 47 shows a large hole retrofit being perform showing how the hole intercepts the welds.

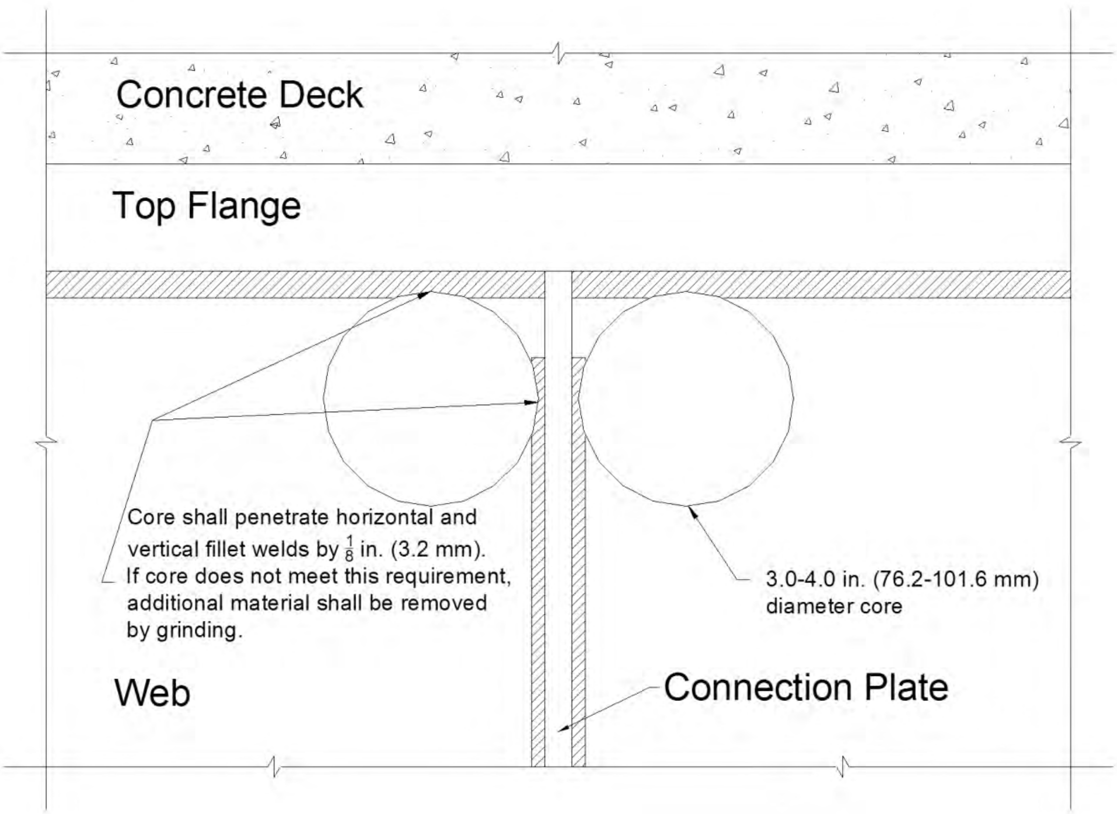


Figure 46. Illustration. Schematic of typical large diameter hole retrofit. Reproduced from Koob and McGormley.[86]

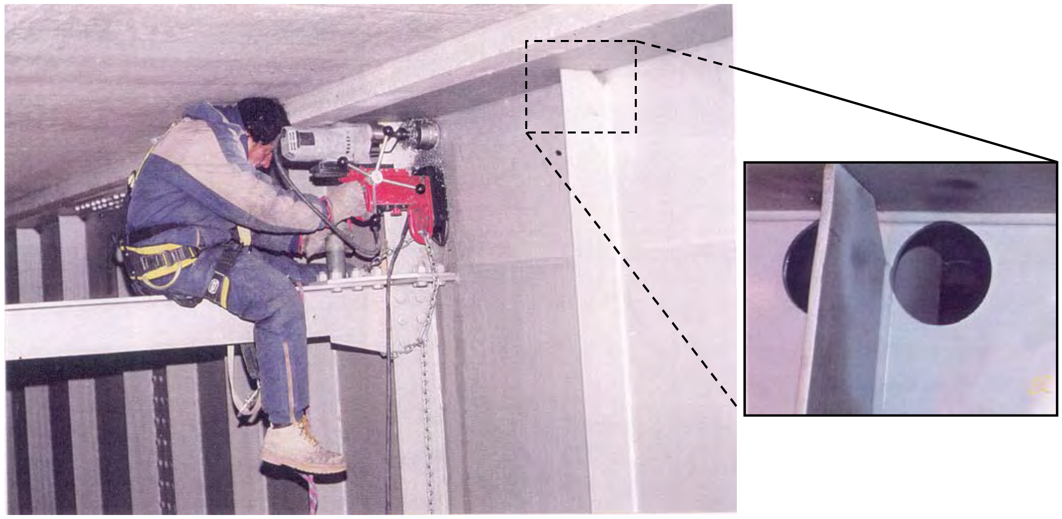


Figure 47. Photo. Implementation of large hole retrofit on I-80 Bridge over Missouri River. Photo courtesy of Iowa Department of Transportation. Photo courtesy of Iowa Department of Transportation.

TIE GIRDER/FLOOR BEAM CONNECTIONS

Another form of web-gap fatigue occurs in tied arch bridges in the connection region between the floor beam and the tie girder. Many bridges have exhibited a cracking pattern along the web/flange weld of floor beams in the small web-gap area of the connection to the tie girder (see Figure 48). The cracking occurs because the concrete deck is not composite with the tie girder, and high secondary stresses arise due to lack of displacement compatibility between the tie girder and the floor beam. The deformation mode is not elementary, but Figure 49 will aid in the explanation. Typically, in a tied arch bridge, the tie girder (or lower truss chord) is not composite with the deck, and all loads from the deck are transferred from the deck via stringers and floor beams. The floor beams usually frame into the tie girders with simple shear connections using either connection plates or bolted angles as shown in Figure 49. The reactions from the floor beams cause the tie girder to bend and rotate. The problem is the deck is fairly rigid, and since the stringers are composite with the deck, the top flange of the floor beam becomes restrained. However, as the tie girder rotates from the bending, a torque is induced on the floor beam, causing a severe deformation of the small web gap in the floor beam (see Figure 49). The web gap deformation is reverse curvature bending of the floor beam web, hence the tensile stresses perpendicular to the floor beam web/flange weld.

Previous retrofits of these details have followed stiffening and softening strategies. Stiffening techniques force the floor beam and tie girder to displace together. This has been done either by making the tie girder composite with the deck, or by providing a rigid connection between the top flange of the floor beam and the tie girder. Directly welding the floor beam flange to the tie girder should be avoided since it creates a Category E detail in tie girder, which could create more problems than it fixes. Making the girder composite to the deck was used in the I-283 bridge in Harrisburg, Pennsylvania. This retrofit involved adding a continuous connecting element between the deck and the girder for defined lengths at the supports (15 ft. (4.6 m)). This is not only difficult and expensive, but in recent years has proven ineffective in removing the crack driving force. Another stiffening retrofit was used in the Prairie Du Chien bridge carrying US 18 over the Mississippi River between Wisconsin and Iowa.[5] The tie girder in this bridge was a built-up box section, which required extra detailing for the stiffening retrofit to carry the load into an internal diaphragm of the box section as shown in Figure 50. Since the I-283 stiffening retrofit has not been completely reliable, and the Prairie Du Chien bridge repair was expensive and difficult to implement, this manual recommends the softening technique for this cracking phenomena. As described previously, the softening technique requires removal of material to create a longer web gap, thus allowing the deformation to occur over a longer length to reduce the stress range. Figure 51 details a generic softening retrofit, but since it is generic, analysis and engineering common sense will be required to properly detail a retrofit. Special care must be taken when specifying the number of bolts to be removed so that the shear capacity of the connection is not compromised. A similar retrofit was used in the Birmingham Bridge in Pittsburgh, shown in Figure 52.[87] Notice the specific retrofit used on the Birmingham Bridge cut away the remaining portion of the connection angles instead of leaving empty bolt holes as shown in the generic retrofit. This is sometimes necessary pending the outcome of stress range verification from load tests.

Though it was not exclusively a tie girder/floor beam cracking problem, a similar retrofit as shown in Figure 51 was used to fix floor beam cracking issues in the Girard Point Bridge outside Philadelphia.[88] The Girard Point Bridge is a truss bridge where the floor beams frame into the vertical truss members. The first retrofit involved flame cutting a U-shape out of the floor beam web to increase the web-gap length. However, data obtained from controlled load testing revealed that the stress ranges were still high enough that cracks could reinitiate. The first repair was modified with the hypothesis that the double angle connection was still restraining the web gap too much. The modification then required a large hole to be drilled in the outstanding legs of the angles, and flame cutting away the extra portion of the angles. The modification then increased the length for the floor beam web to flex over in the longitudinal axis of the floor beam. Truck testing verified the retrofit modification. Both of these retrofits are shown in Figure 53. The retrofit case of the Girard Point Bridge demonstrates the extreme importance of instrumentation and load testing to validate and assess the performance of any retrofit.

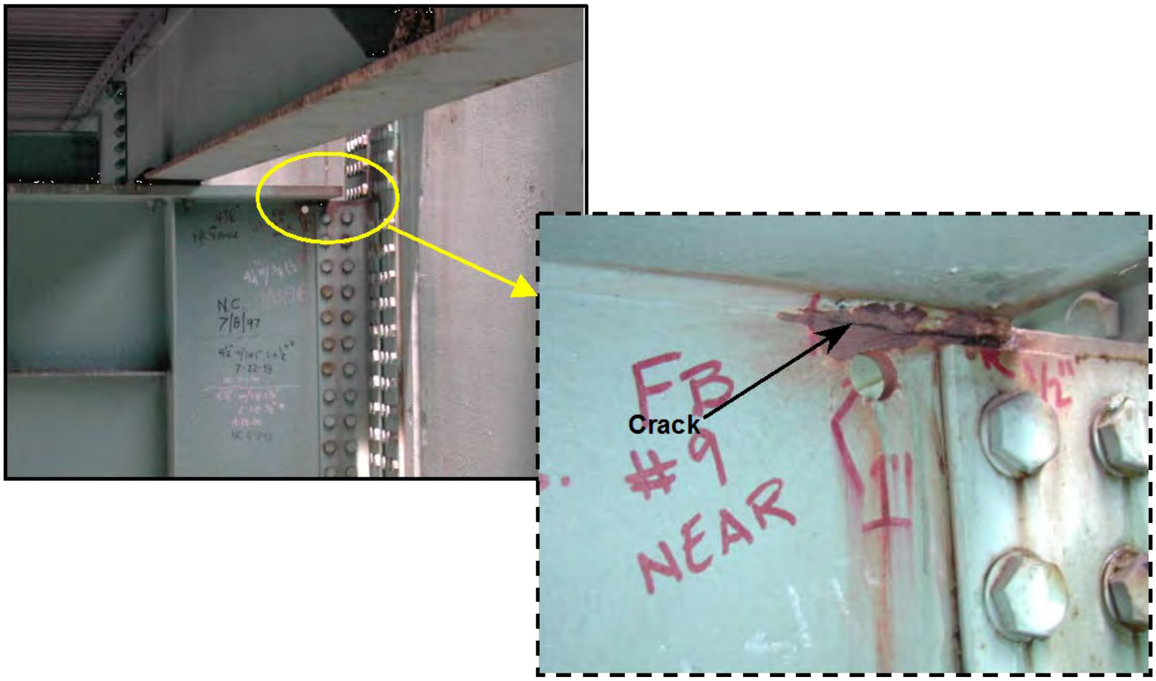


Figure 48. Photo. Cracking in the web gap at a tie girder/floor beam connection on Birmingham Bridge in Pittsburgh, Pennsylvania. Photo taken from Connor et. al.[87]

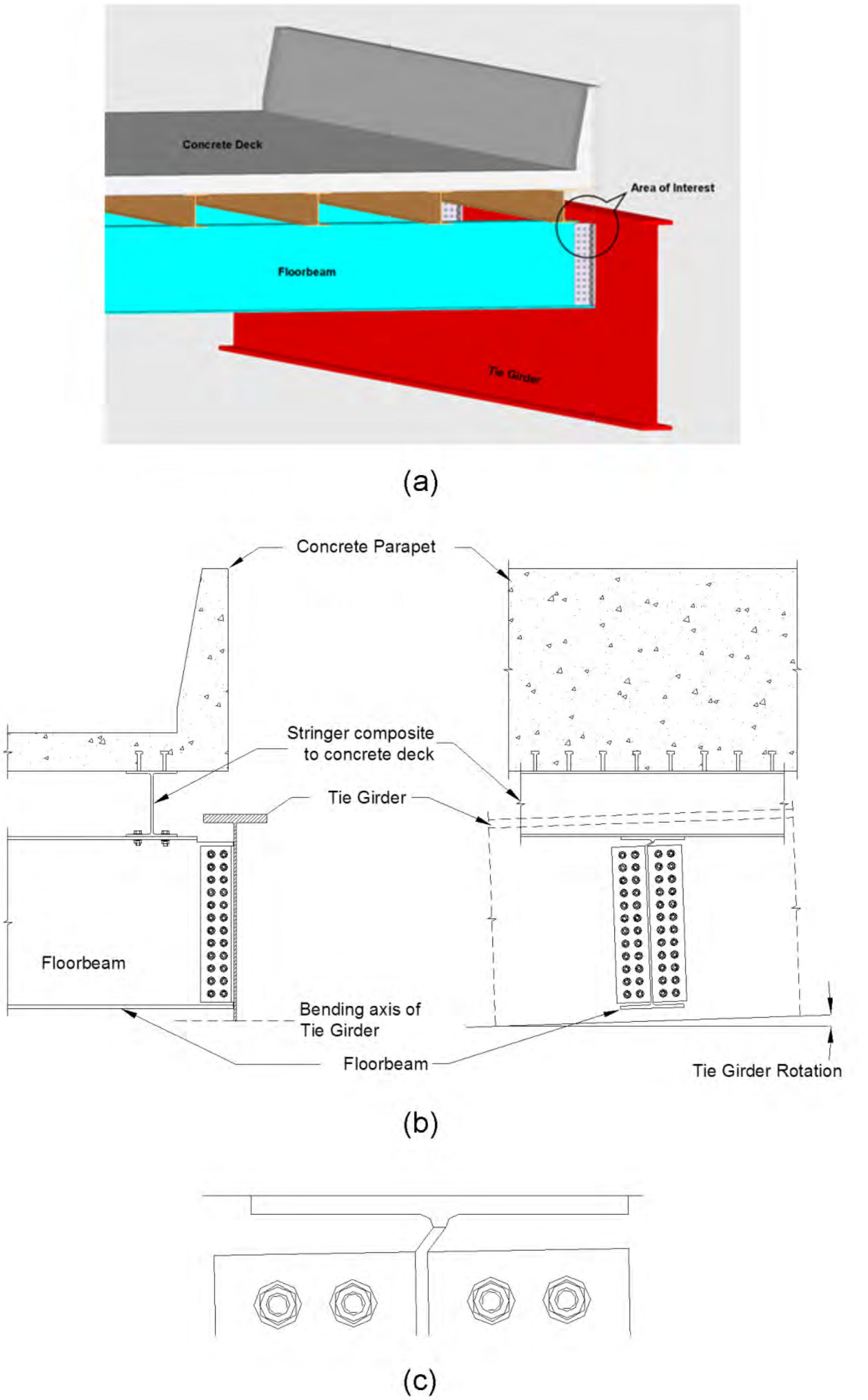
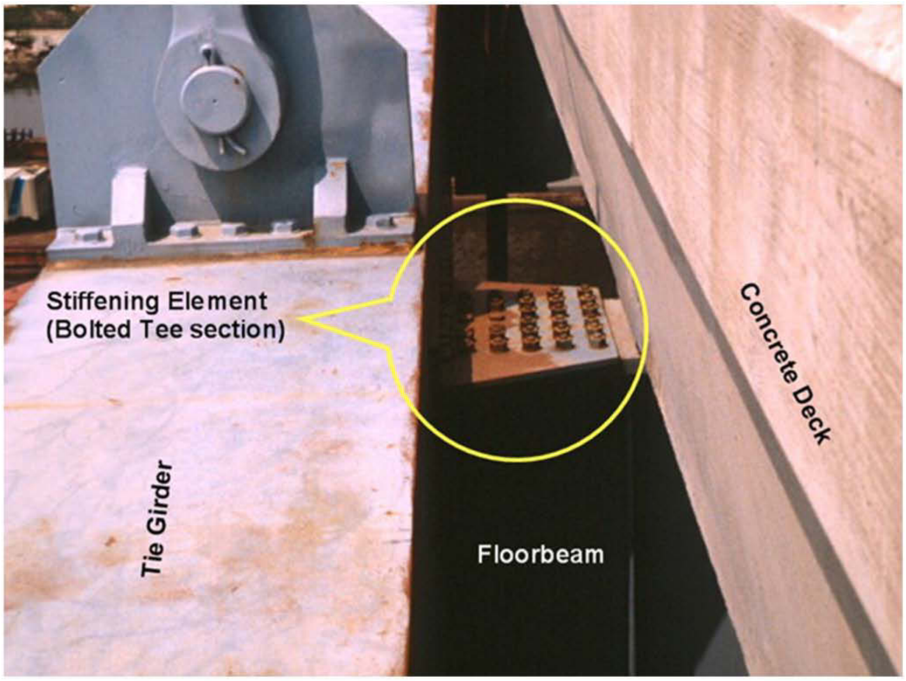
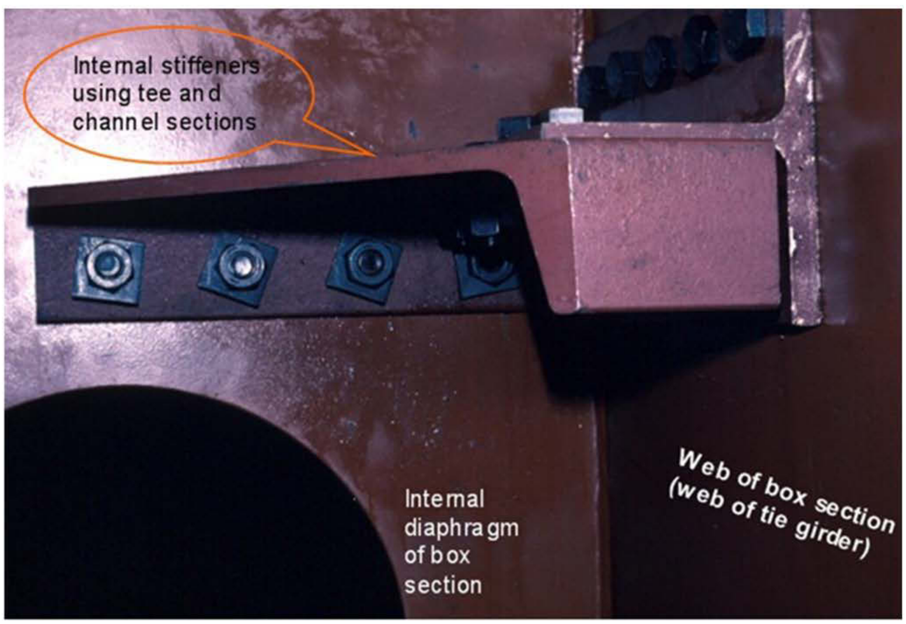


Figure 49. Illustration. Schematic of tie girder to floor beam cracking driving force. (a) A generic deck system of an arch bridge using a tie girder. (b) Close-up view of members showing deformation caused by tie girder rotation. (c) Close-up view of web-gap deformation of floor beam web.

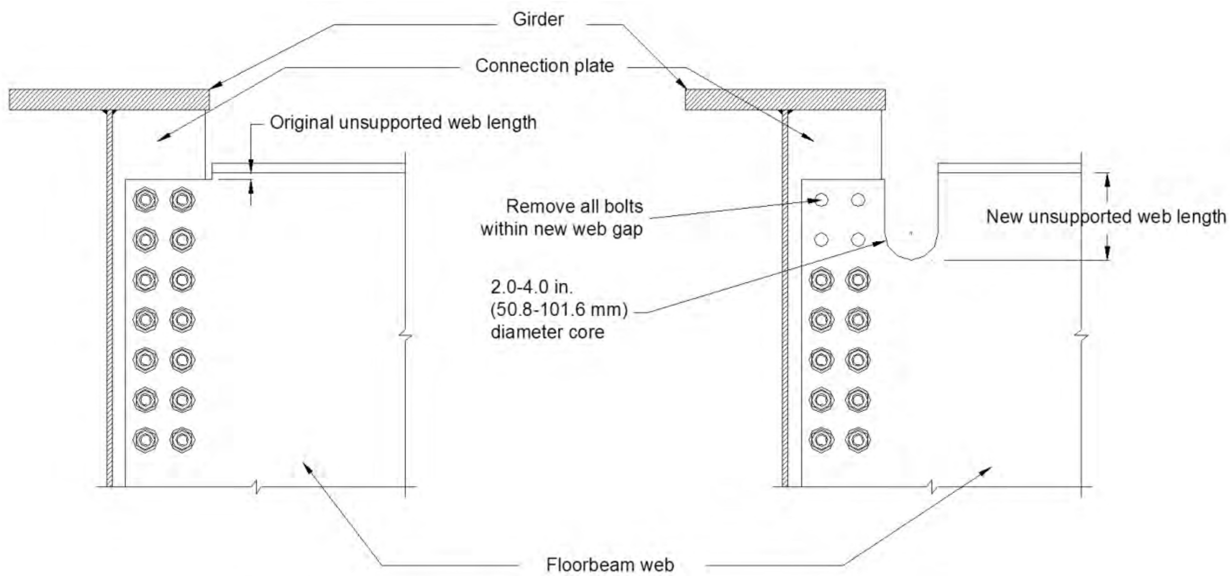


(a)



(b)

Figure 50. Photo. (a) External stiffener used in Prairie Du Chien Bridge (US 18 between Wisconsin and Iowa) to alleviate floor beam crack driving force. (b) Stiffeners required to transfer the force from the external tee stiffener to the internal diaphragm of the box girder. Photo courtesy of the Minnesota Department of Transportation.



Procedure

1. Determine the new web-gap length. One recommendation is to increase the length by 10-20 times the original web-gap length. This may be controlled by the number of bolts that can be removed. The shear resistance of the connection cannot be reduced below the loads applied within the connection.

2. Core a 2.0-4.0 in. (50.8-101.6 mm) diameter hole through the floorbeam web, close to the connection plate, but avoid removing material from the connection plate. The placement of this hole will determine the new web-gap length.

3. Flame or plasma cut away the portion of the floorbeam remaining, making the cut tangent to the core.

4. Grind the flame-cut surfaces smooth to remove future crack initiation sites.

5. Prime and paint all bare steel to prevent corrosion.

Figure 51. Illustration. Work plan to retrofit tie girder/floor beam connection cracking.

Figure 52. Photo. Tie girder/floor beam connection retrofit on Birmingham Bridge, Pittsburgh, Pennsylvania. Taken from Connor et. al.[87]

Figure 53. Photo. First retrofit performed on Girard Point Bridge in Philadelphia, Pennsylvania (left) and modified retrofit (right). Taken from Mahmoud and Connor.[88]

CANTILEVER BRACKET CRACKING

The last form of secondary stress fatigue occurs in floor beam cantilevered brackets. Figure 54 shows a cross-section of a two-girder bridge where the deck overhangs enough to require the use of cantilever brackets. The area of concern is the connection between the floor beam and cantilever bracket (called out by the dashed box). Figure 55 shows a zoomed-in view of this area. To make the flanges of the floor beam and cantilever continuous for transferring moment, a tie plate spans over the girder flange between the two. If there is any kind of positive attachment between the tie plate and girder flange (i.e., bolts, tack weld, corrosion, or friction), displacement compatibility between the girder's top flange and the tie plate forces the tie plate to bend out of plane as shown in the right-hand portion of Figure 55. The displacement compatibility arises because the girder is not composite with the concrete deck, yet the stringers spanning from one floor beam to the next are composite with the deck. The stringers then restrain the top flanges of the floor beam and cantilever bracket from moving in the longitudinal direction of the girder. As the girder bends under moving loads, the girder flange becomes slightly displaced from the floor beam/cantilever bracket flanges. The out-of-plane flexing of the tie plate causes cracks to form, ultimately leading the tie plate to fracture.

The most appropriate strategy to address for tie plate cracking depends on the specific bridge detailing. In the case of the Lehigh Canal Bridge, temporary tack welds that held the tie plate in position for riveting spawned crack initiation sites under the secondary stresses.[(5)] For this bridge, the temporary tack welds were removed, and redesigned tie plates were installed to eliminate the positive attachment of the tie plate to the girder. Other possible retrofits would be to first remove any positive attachments (i.e., bolts or rivets) between the tie plate and the girder and verify the reduction of stress ranges. Thicker tie plates could be swapped for the old ones, or doubler plates could be added atop the old tie plates to increase the section modulus and provide internal member redundancy, thus reducing the stress ranges.

The final retrofit possibility is shown in Figure 56. A spacer plate is added beneath the tie plate but does not span across the girder flange. This creates a gap between the girder flange and tie plate, thus allowing free rotation of the girder without imparting out-of-plane bending in the tie plate. However, this may cause more flexing in the copes on the floor beam and cantilever bracket, which could lead to another fatigue cracking site. This depends on the particular bridge detailing. Field instrumentation should be used to verify stress range reductions.

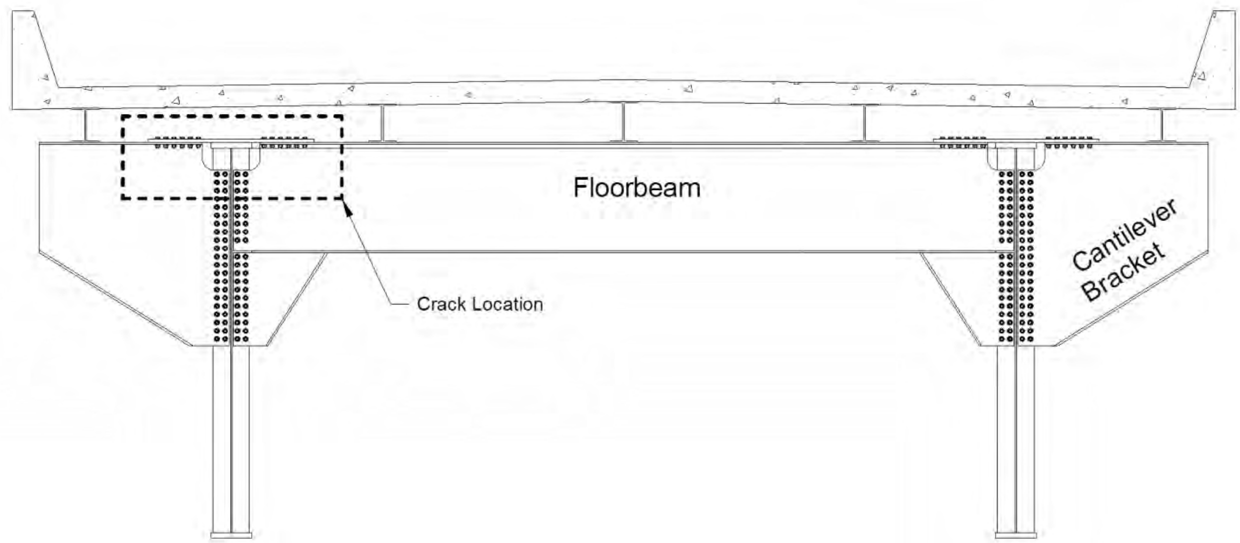


Figure 54. Illustration. Typical cross-section of a two-girder bridge with cantilever bracket outriggers.

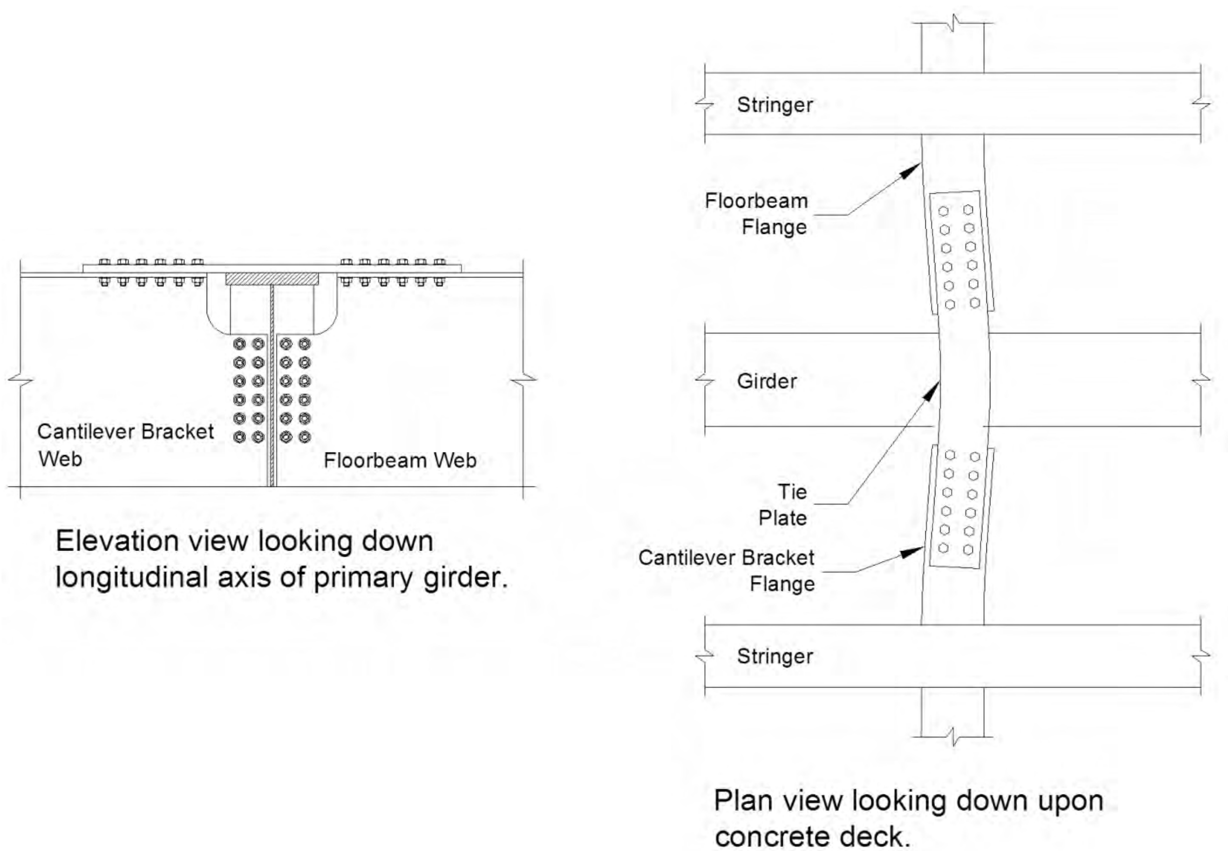


Figure 55. Illustration. Zoomed-in view of tie plate detail (left) and deformation mode that causes cracking (right).

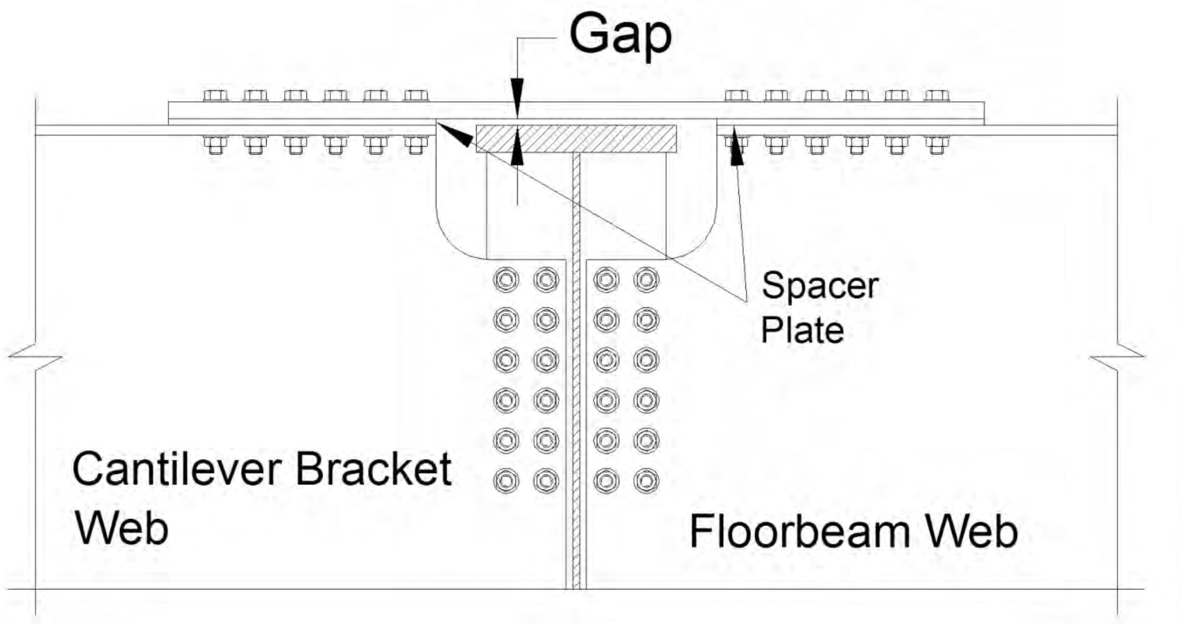


Figure 56. Illustration. Retrofit of tie-plate cracking through addition of spacer plates.

PROTOTYPE VERIFICATION OF SECONDARY STRESS REPAIR

Retrofitting web-gap fatigue is not an exact science, and not all retrofits will work in all situations. Therefore, verification of a particular retrofit must be performed before specifying every detail in an entire bridge be retrofitted. Analytical modeling (i.e., finite element analysis) can be used to compare various retrofit strategies and to determine the effectiveness of different retrofits. However, the possibility of modeling assumptions (i.e. connection stiffness assumptions, boundary conditions, etc.) would still make field stress-range verifications prudent. The easiest and simplest plan would be to instrument an uncracked detail, perform a retrofit based on recommendations from this manual, and verify the reduction of stress ranges and/or out-of-plane displacement at locations of potential cracking. When specifying welded retrofits, ensure that the weld retrofit itself has adequate fatigue resistance so as not to create a new problem. AASHTO should be consulted for detail categorization where possible. The engineer should choose whether to conduct an infinite or finite life analysis.(1) The instrumentation can be used to compare the stress ranges before and after the retrofit, at which point a cost/benefit analysis can be done to determine the best retrofit strategy for the entire bridge.

The instrumentation may show that the retrofit is not effective, and more modifications must be performed (as described for the Girard Point Bridge). Each bridge behaves differently so verification is always necessary. Typical field instrumentation could include strain gauges to measure stresses directly and/or displacement measurement devices such as linear variable differential transformer (LVDTs) or dial gauges. Displacement measurements may be preferred because of their ease and low costs, but they cannot be used in all situations.

In all fatigue retrofit verification, the absolute magnitude of the stresses is not the only criteria to consider. The reduction in the stress range value from before retrofitting is usually the critical measurement for determining the effectiveness of a retrofit. Once the reduction in stress range is attained from field verification, the fatigue life of the retrofit can be determined from crack growth laws. Since crack growth laws indicate that fatigue life is proportional to the cube of the stress range, halving the stress ranges produces an eight times improvement in life. However, each situation in calculating remaining life will have its own caveats. For instance, consider a butt-weld with fabrication-related discontinuities that has cracked after 5 years of service life under a nominal stress range of 15 ksi. If that crack was merely gouged out and weld repaired, the fatigue clock can be reset to zero. Since nothing was done to reduce the stress range, it could be expected that the repair butt weld will not crack for five more years of service. A critical assumption in this calculation is the histogram of loading remains similar between the first and second five year cycles. However, if the same cracked butt-weld is weld repaired and coverplated to reduce the stress range at the weld to 7.5 ksi, the repair weld would be expected to crack 40 years after the repair. Note in this case, the coverplates would likely be bolted, so the addition of holes will create a new fatigue detail that will have to be checked, and may govern the remaining life. Another important caveat to remember is if an uncracked detail is preemptively retrofitted to reduce the stress range, you must consider the consumed fatigue life before making a new life prediction. This can be illustrated by expanding upon the hypothetical situation above, however considering a similar butt-weld located in a nominal stress range of 10 ksi. After five years of service, this butt-weld is preemptively retrofitted based on cracking observed in the weld under a 15 ksi stress range. Based on the fatigue life provided by the first weld, the expected life of the second weld is estimated to be 5 years times $(15/10)^3 = 16.8$ years without retrofitting. So, approximately 30% of the life has been consumed when the retrofit is enacted. To determine the remaining life after the retrofit, you must first calculate the total possible life (5 year times $(15/5)^3 = 135$ years) and apply reduction to account for the previous 30% damage. In this case, the remaining life equals 135 years times 70%, or 94.5 years.

The rest of this section will be devoted to explaining the use of strain gauges and displacement devices for experimental validation of the recommended retrofits.

Strain Measurements

Strain gauges are a very effective way to measure stress ranges on steel bridges. They work under the principle that the change in foil grid resistance is proportional to the strain in the grid. The only drawback to using a strain gauge is an external conditioner will be required to complete the Whetstone bridge circuit to measure the resistance change within the gauge, and such equipment may not be readily accessible. In addition, a certain level of skill, knowledge, and practice are required to glue/weld a strain gauge to a steel bridge (particularly in extreme environments and tight, congested areas). Strain gauges can either be glued or spot welded to steel bridges, but weldable gauges are better suited for cold climates because some adhesives do not work well in cold conditions. Gauges are offered in a variety of formats, but simple, uniaxial grid patterns with 3-6 mm (0.125-0.25 inch) gauge lengths are all that is needed for verification purposes. Depending on the data collection system, the gauges can be read in a variety of ways, but common setups use 120 ohm or 350 ohm gauges in a quarter bridge circuit. Strain readings should also be collected at high enough rates to capture strain peaks. In the case of moving truck loads, the minimum sampling rate should be twice the natural frequency of the bridge to capture the vibration of the bridge (because of the Nyquist Criterion), but the question is what naturally frequency to use. The sampling rate should be selected based on previous experience, but collection rates of 50-100Hz are commonly used. The number of strain gauges applied should yield enough experimental evidence to make a confident decision, but at minimum, two are recommended within web gaps as shown in Figure 57. The strain gauge layout presented in Figure 57 places the gauges at the boundaries of the web gap to ensure there will be measurable strains despite single or double curvature deformation of the web gap. However, if feasible, it would be best to use four strain gauges back-to-back as shown in Figure 57 to directly measure membrane and bending stresses through the girder web. For stiffening and softening retrofits, this may be all that is required. For diaphragm removal and bolt loosening techniques, more gauges should be placed on other bridge members to determine the change in load paths to ensure a fatigue problem is not pushed to another portion of the bridge. For instance, one strain gauge affixed to the centroid of each member of a crossframe to determine the axial loads within the crossframe. However, potential bending moments will not be captured. Figure 58 shows how multiple strain gauges can be applied to a WT member of a cross frame to attain axial and bending loads. For diaphragms, three gauges should be placed vertically along the diaphragm: two at the extreme fibers and one at the centroid so that axial and bending loads can be determined. Figure 59 shows how strain gauges can be applied near the free edges of the retrofit specified in Figure 51. However, care should be taken when directly comparing these strain readings directly to S-N curves. The AASHTO S-N curves were calibrated based on nominal stresses, and the strain readings recorded by these gauges are local strain readings (the strain includes components of the nominal stress *and* the geometric stress concentration factor). Previous research has found local strain readings next to weld toes correlated well to the AASHTO Category C curve.[47] When a strain gauge is placed near the edge of a hole (as in Figure 59), the strain gauge is also reading the local strain; however, the detail itself does not have residual stresses from welding, and can therefore, be categorized as a Category A detail. This is the fatigue category for base metal. It would be appropriate to use the Category A when analyzing base metal hot-spot strain readings. However, to use the Category C resistance when analyzing local strains next to weld toes is more ambiguous. All of the fatigue categories have different CAFLs, but measuring the hot-spot strain collapses all these to approximately Category C. There is limited experimental data to suggest the Category C CAFL is appropriate for local stress analysis of welds. Therefore, a finite life calculation should be pursued as a conservative approach.

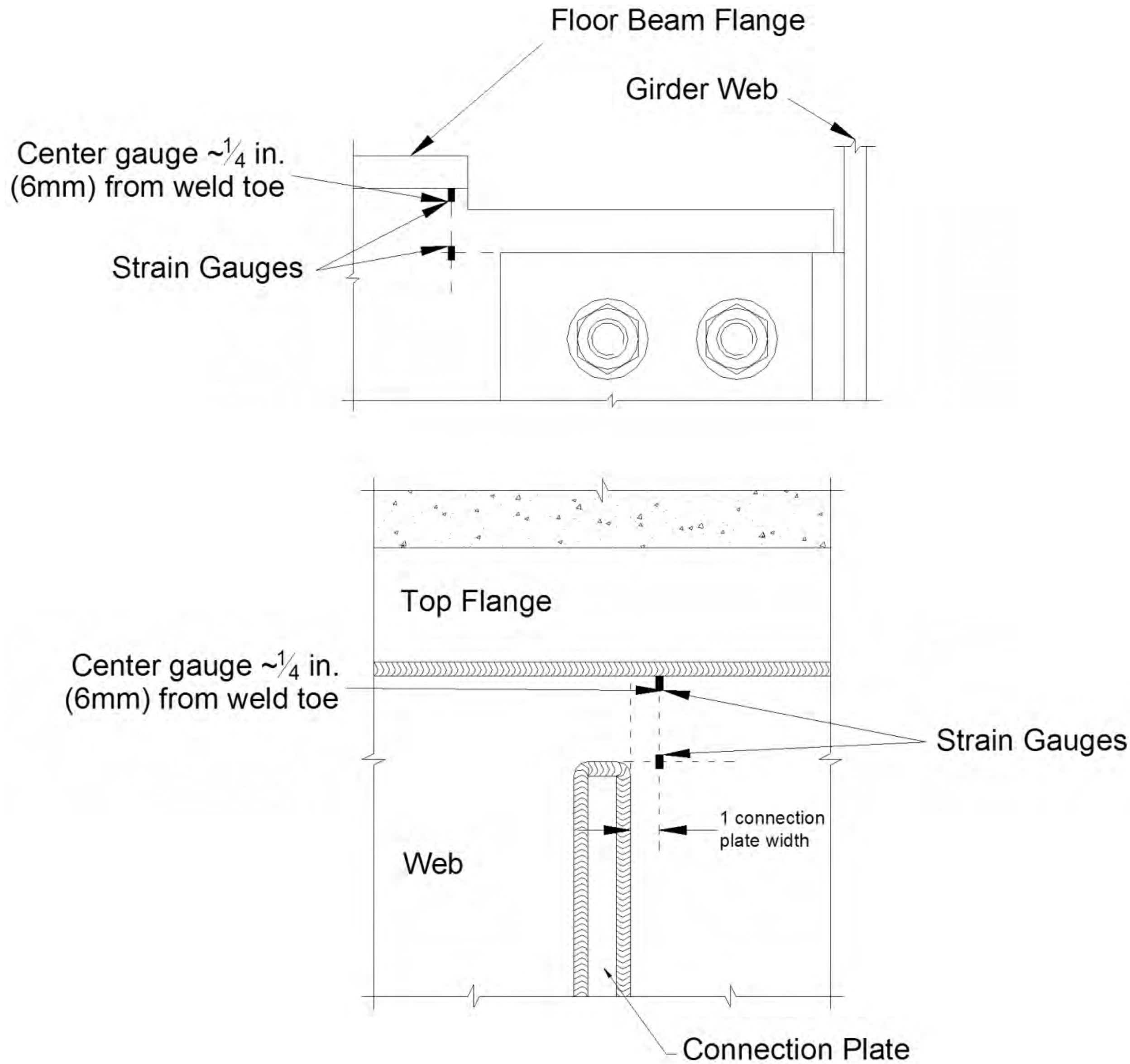


Figure 57. Illustration. Recommended strain gauge placement for retrofit verification. Bottom: strain gauge placement for out-of-plane distortion. Top: strain gauge placement for floor beam/tie girder connection or beam cope cracking.

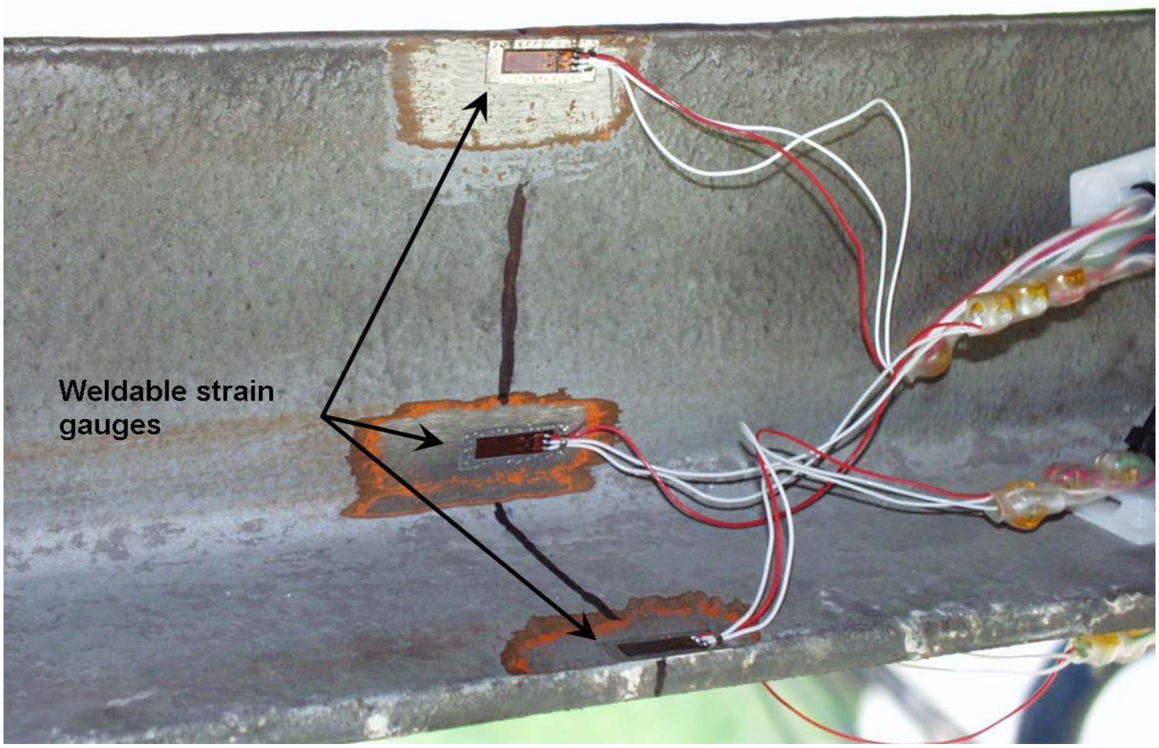


Figure 58. Photo. Weldable strain gauges applied to wind bracing tee member of I-535 Bridge in Duluth, Minnesota.

Figure 59. Photo. Weldable strain gauges applied near free edge of retrofit on Girard Point Bridge, Philadelphia, Pennsylvania. Taken from Mahmoud and Connor.[88]

Displacement Measurements

Measuring displacements is quicker and cost-effective in many instances rather than the measurement of strains, hence it becomes a preferable validation method. The theory behind measuring displacements is fatigue cracking occurs when the bridge is elastic, so there is a linear relationship between displacements and stresses, or ranges thereof. The problem is the measurement of displacements is really only applicable for use on stiffening retrofits. For example, welding the connection plate to the girder flange should completely lock up all relative motion between the connection plate and the girder flange. This provides a global reference for the perfect situation to remove the crack driving force. Bolting the connection plate to the girder flange can only partially restrain the relative movement, so measuring displacements before and after this retrofit gives a relative reference for the retrofit effectiveness. An example would be to measure the relative displacement between a connection plate and a girder flange, as shown in Figure 60 and Figure 61. If the displacements were reduced by a factor of 3 from before and after the retrofit, then the retrofitted detail would have approximately 27 times the life of the original detail. This cannot be said for softening techniques because their intent is to increase the compliance of the system, hence displacements will increase. There is no well-known relation between the increase of displacement to the decrease of stress because there is no defined reference point as with the stiffening retrofits and it would require engineering judgment.

Displacement measurement can use either dial gauges or LVDTs. Using LVDTs will require some signal conditioning and recording equipment, but the results will be more accurate than a dial gauge. However, the dial gauge is a quick, efficient means to gather preliminary data. The easiest technique is to use a strong, stiff magnetic base (to ensure minimal vibration) attached to the girder flange and the dial gauge or LVDT bearing on the connection plate as shown in Figure 61.

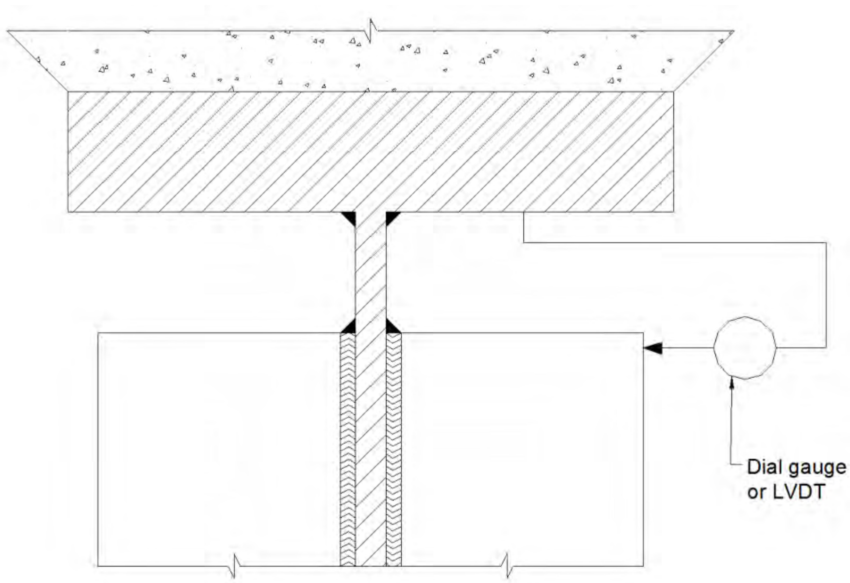


Figure 60. Illustration. Recommended web-gap displacement instrumentation.

Figure 61. Photo. Magnetic base and LVDT to measure the relative displacement between the girder and connection plate. Photo courtesy of Iowa Department of Transportation.

CHAPTER 5. LOAD-INDUCED FATIGUE CRACK REPAIR

COVERPLATES

Coverplates are used to increase the moment of inertia of a cross-section at select locations along a beam where moments are high. Coverplates welded to beam flanges are susceptible to fatigue cracking. Cracking occurs at the toe of the fillet weld of the coverplate termination as shown in Figure 62. Cracking of these details was first noted during the AASHTO Road Test between 1958-1960. The most noted coverplate cracking problem was found in the I-95 Connecticut Turnpike Bridge (Yellow Mill Pond) in the early 1970's. Prior to retrofitting the Yellow Mill Pond structure, an experimental study was first conducted to determine the ideal retrofit procedure. Grinding, hammer peening, and GTA-remelt were experimentally investigated as viable retrofitting procedures.(52) Grinding was found to be ineffective in improving the fatigue life of the as-welded and cracked details, and was not recommended as a retrofit procedure. Hammer peening was found to be more effective than grinding. In fact, hammer peening was able to provide a one category improvement (from E' to E) of uncracked details with surface crack depths up to 0.125 in. (3 mm). The GTA-remelt was found to be the most effective retrofit with the ability to repair cracks with depths up to 0.188 in. (5 mm). However, the GTA-remelt process requires more operator skill to perform than hammer peening, making it less attractive. Based on the findings of the experimental investigation, 14 coverplates on the Yellow Mill Pond Bridge were hammer peened and 11 were GTA-remelted as a retrofit procedure without closing down the road to traffic. This bridge was decommissioned between 1997 and 1998 because of inadequate vehicle capacity. Six of the retrofitted girders (i.e. those that had no surface cracks prior to retrofitting) were sent to Lehigh University for further evaluation. No cracks were observed in the retrofits after 20 years of service and an estimated 56 million truck passings.(89) The retrofitted beams (peened and GTA-remelt) were cycled to failure and all but one of the beams attained Category C resistance, the original untreated coverplate was a Category E' detail.

Another repair option would be to use bolted splice plates. This technique was experimentally studied at Purdue University.(90) Test beams were precracked and then retrofitted with bolted splice plates. The bolted splices were designed assuming the beam flange had been completely severed (though they were not) and the splice plates needed to develop the full flange force. The connection was also designed to be slip-critical and therefore the splice plates had to be extended beyond the coverplate termination so the splice could be developed at the crack. All of the beams tested using this retrofit attained Category B resistance after the repair, with failure defined as a severed flange. It is recommended to place a drilled hole in the web above the crack in conjunction with the splice plates. The hole should be 1.0 in. (25.4 mm) in diameter and drilled above the k-line into the web, over the crack, so if the flange eventually severs, the crack cannot grow further into the web (see Figure 63). This research also looked at using a partial splice plate (only using the upper splice plate next to the web) in conjunction with peening the weld toe of the coverplate. The data associated with repair had more scatter with repaired detail fatigue strengths ranging from Category C to better than Category B. The only real advantage of this repair over the fully bolted splice plate would be the extra clearance beneath the bottom flange if this was a constraint.

The repair shown in Figure 63 depicts a lower flange under positive moment, but coverplate cracking can occur in negative moment regions in the top flange. The repair is the same, but becomes more difficult because the concrete deck must first be excavated.

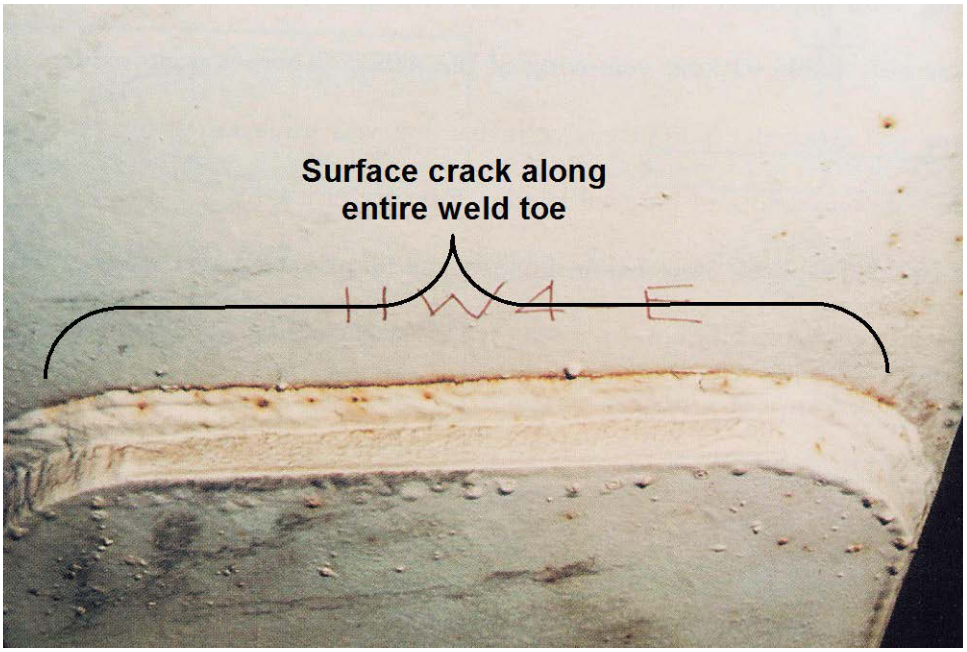


Figure 62. Photo. Cracked coverplate termination of Yellow Mill Pond of the I-95 Connecticut Turnpike.

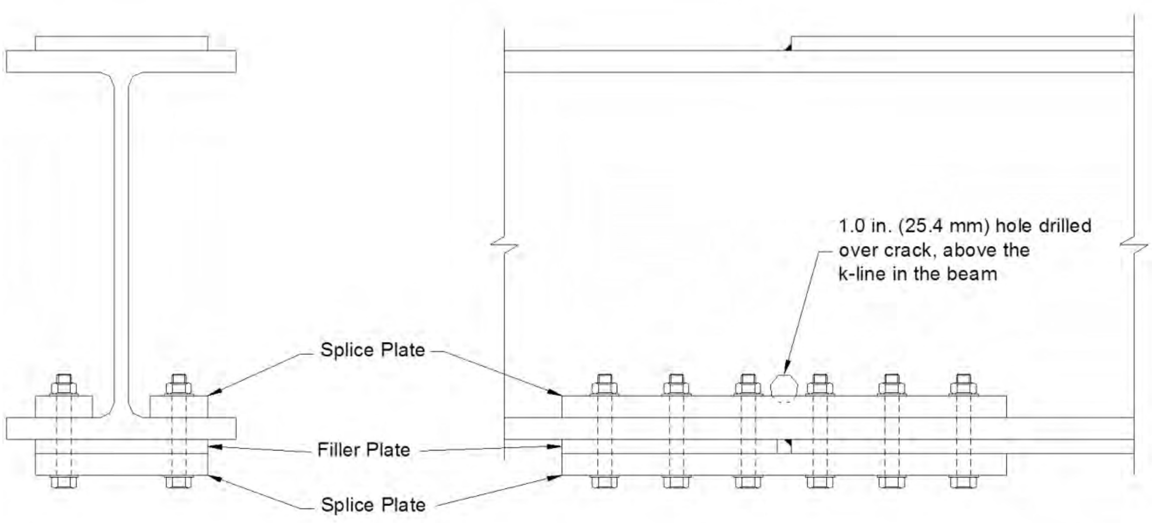


Figure 63. Illustration. Detailing of splice plate retrofit for cracked coverplate details.

EYEBARS AND HANGERS

The problem with eyebars is they usually come hand in hand with bridges of low redundancy; therefore, most eyebars can be classified as FCMs. Inherent flaws present from the fabrication process of the eyebars can serve as an initiation sites for fatigue cracks, which can grow to a critical length, leading to fracture. One of the most famous structural bridge failures occurred when the Point Pleasant Bridge (a.k.a. Silver Bridge) collapsed in the late 1960's due to a fatigue crack in an eyebar.[24] The fractured eyebar can be seen in Figure 64.

Unfortunately, there are no repair methods that can be recommended for cracked eyebars and hangers, because they are FCMs. The only thing that can be done is to fabricate new members with more improved fabrication techniques and higher material toughness, or add redundancy. As a temporary repair, post-tensioning tendons or bars should be placed on the structure, parallel to the eyebar to offer a secondary load path if the eyebar were to fracture before it could be replaced.

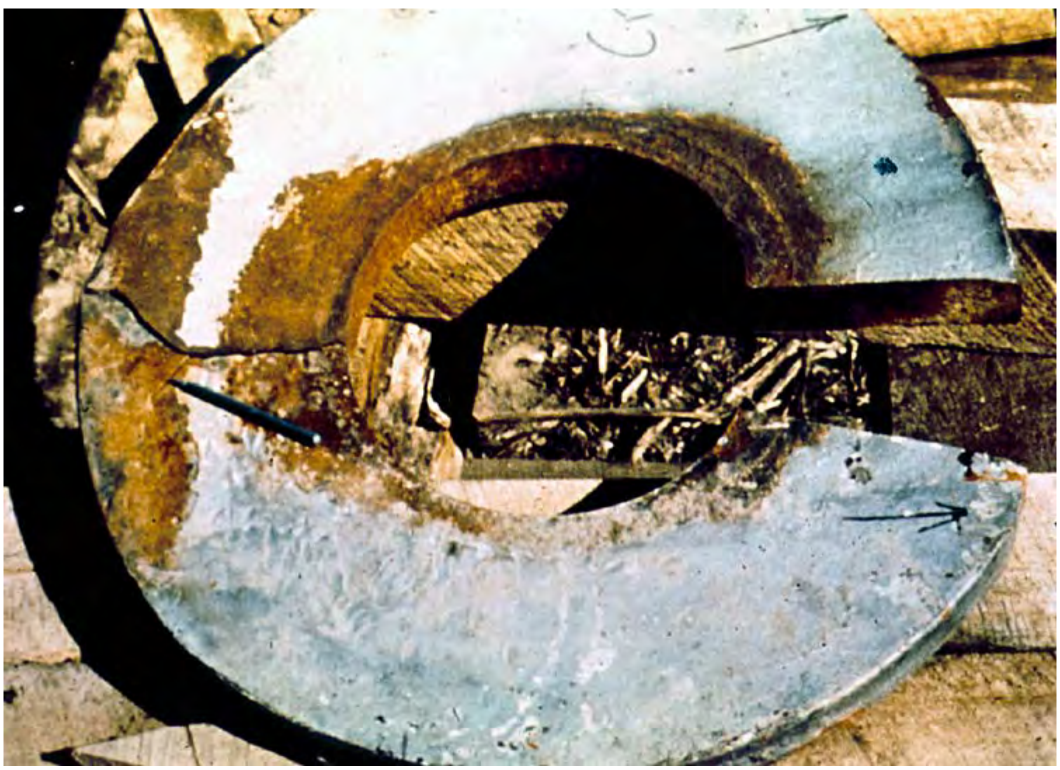


Figure 64. Photo. Fractured eye bar from Point Pleasant Bridge.

TEMPORARY TACK WELDS

Miscellaneous tack welds are commonly found on older structures. The purpose of these welds was to temporarily hold elements in place to aid in bolting, riveting, or drilling operations, as seen in Figure 65. These tack welds have been found to crack, and many people believe they are fatigue problem. The welds do crack from fatigue, but the question that needs to be asked is will it affect the rest of the bridge. Usually cracked (those which have delaminated from the base metal) and uncracked tack welds are benign. However, if a crack is found to have propagated into the base metal, then a repair on the base metal will have to be made. Tack welds which *are* fatigue sensitive are those aligned parallel to the primary stress, which are classified as a Category E detail.(91) Cracks can form at the end of the weld, and the crack will then propagate quickly in the base metal perpendicular to the primary stress. However, tack welds that have cracked through their throat are usually benign and can be evaluated as Category C details (assuming they are less than 2.0 in. (50.8 mm) long in the direction of primary stress). Temporary tack welds can be removed with either a disc or a burr grinder. For tack welds on FCMs, the AASHTO/AWS fracture control plan specifies the removal of tack welds should also remove some base metal to ensure the heat affect zone does not remain in the base metal.(2)

Figure 65. Photo. Tack weld used to temporarily hold a connection angle in place prior to riveting.

CONNECTION ANGLES

Various bridges have experienced cracking of angles used to connect various beams or stringers, which are assumed “simply-supported", therefore only designed for shear resistance. The problem is the angles possess a certain amount of flexural rigidity and cracks initiate due to incompatibility from differential girder displacements. An example of a cracked connection angle is shown in Figure 66. If the flexural rigidity could be decreased, the stresses from bending would also be decreased. The connections crack because the angles are too thick, and the cracking problem could be alleviated if the angle thickness were reduced. There is a published equation that was derived to aid in detailing the connection angle thickness such that cracking will not occur, and is outlined in Figure 67.[9] Where t = angle thickness, g = bolt gauge as shown in Figure 68, and L = the distance between girders. Other research has also suggested that the maximum bending stress range in the angle be less than 24 ksi (165 MPa) (i.e. the angles are Category A) for fatigue cracking not to occur in the angle.[92] This maximum allowable bending stress range was measured directly with a strain gauge placed on the connection angle as shown in Figure 69.

Figure 66. Photo. Cracked connection angle. Photo courtesy of John Fisher.

$$t \le 12\left({g^2}/{L} \right)$$

Figure 67. Equation. Determination of proper angle thickness to avoid connection angle cracking.

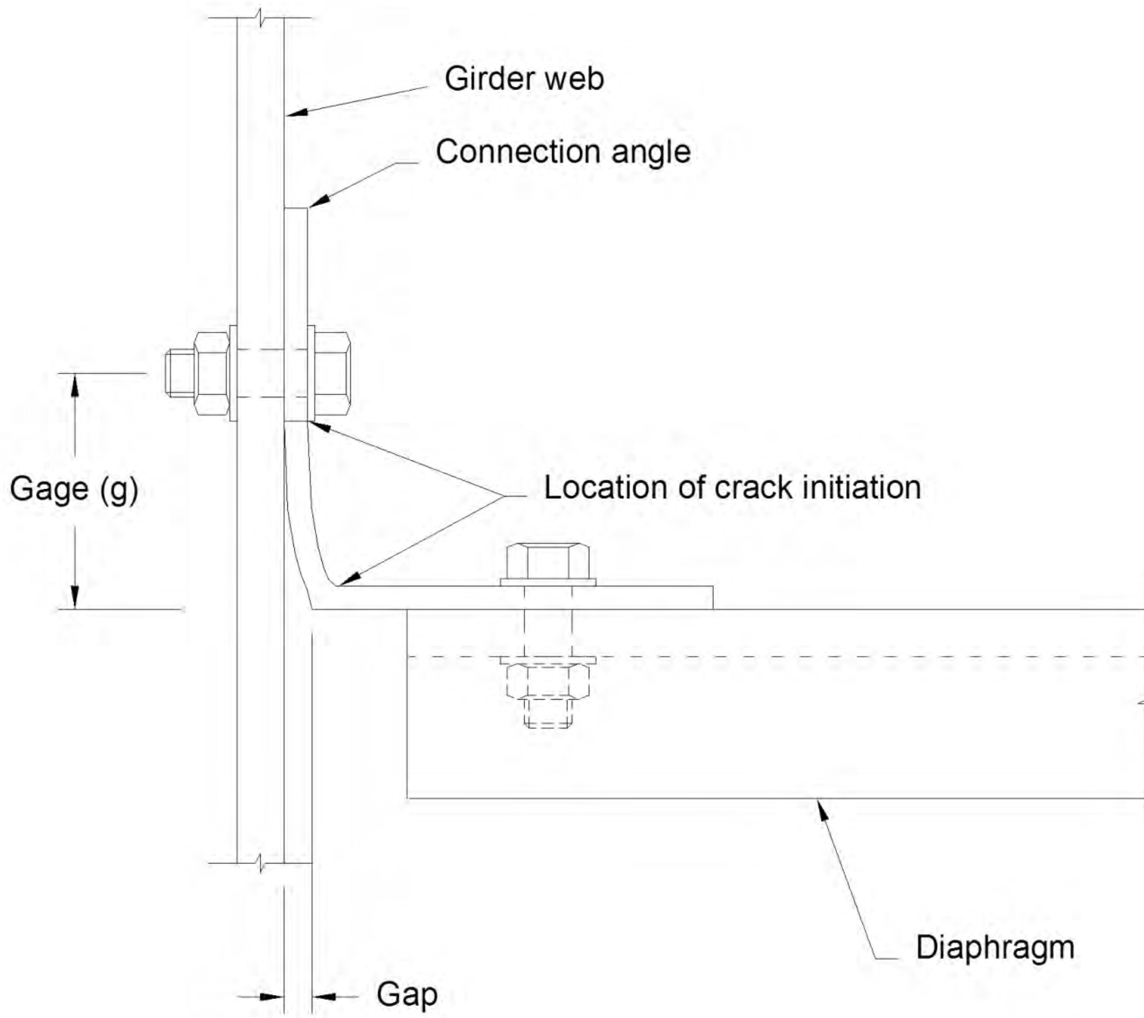


Figure 68. Illustration. Deformation mode of connection angles due to displacement compatibility.

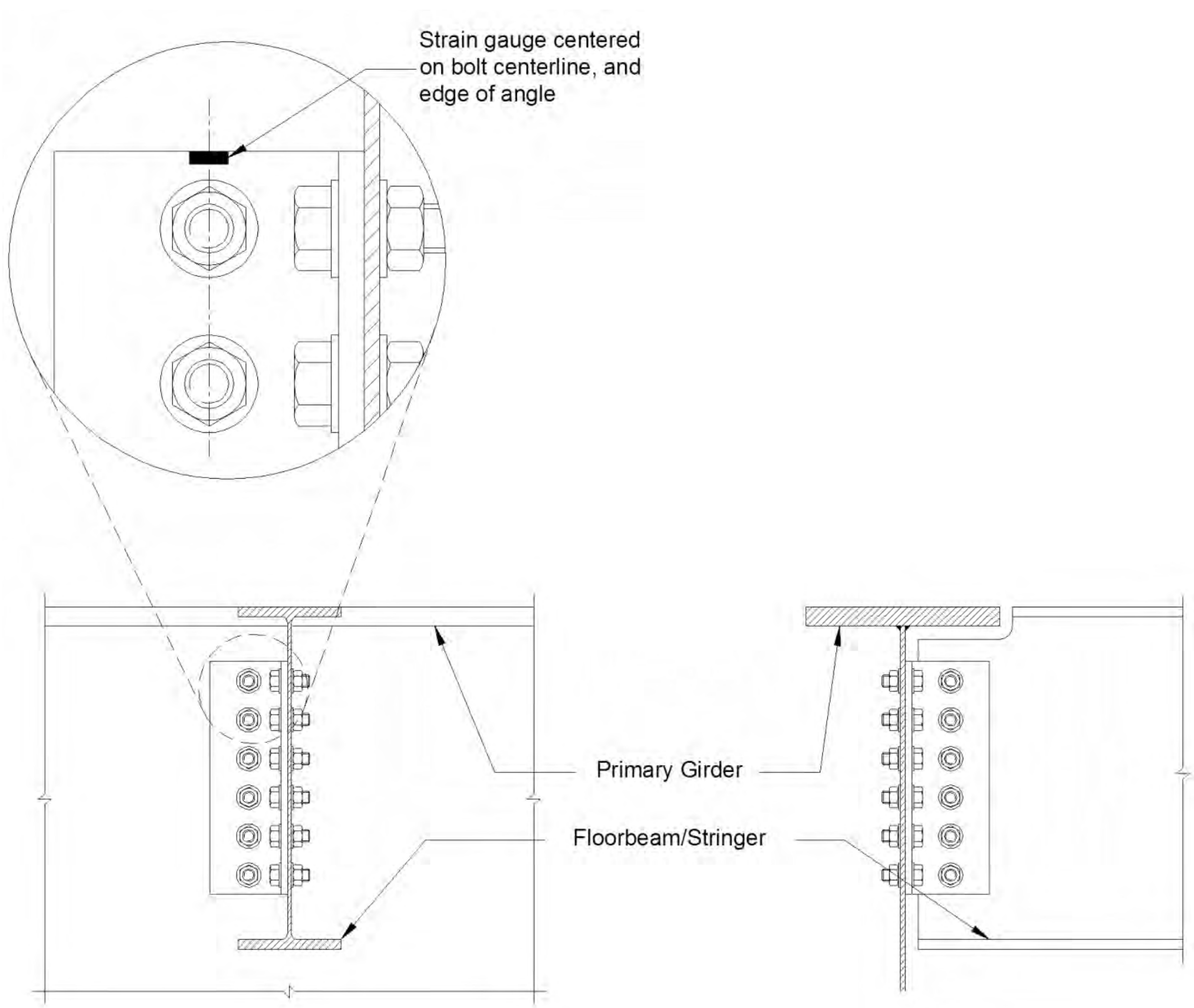


Figure 69. Illustration. Strain gauge location to measure allowable stress range in connection angle.

WEB GUSSET PLATES

Longitudinal web gusset plates have also been a source of fatigue cracks in the past. Bridges built with deep girders require extensive bracing elements for the bottom flange to resist wind loads. Wind bracing is typically a crossframe type structure in the horizontal plane of the bridge, connecting into the girder web via gusset, or shelf plates as shown in Figure 70 and Figure 71. Two types of cracks can form from these details when they are located in tension areas as shown in Figure 70. The first type of crack can grow in the web of the girder from the intersecting welds between the gusset/web weld and gusset/connection plate weld. Either this crack initiates from gross weld root defects, or constraint induced fracture cracks from intersecting welds. The second type of cracking has been known to grow out of the termination of the gusset plate to web weld, which is a Category E detail.

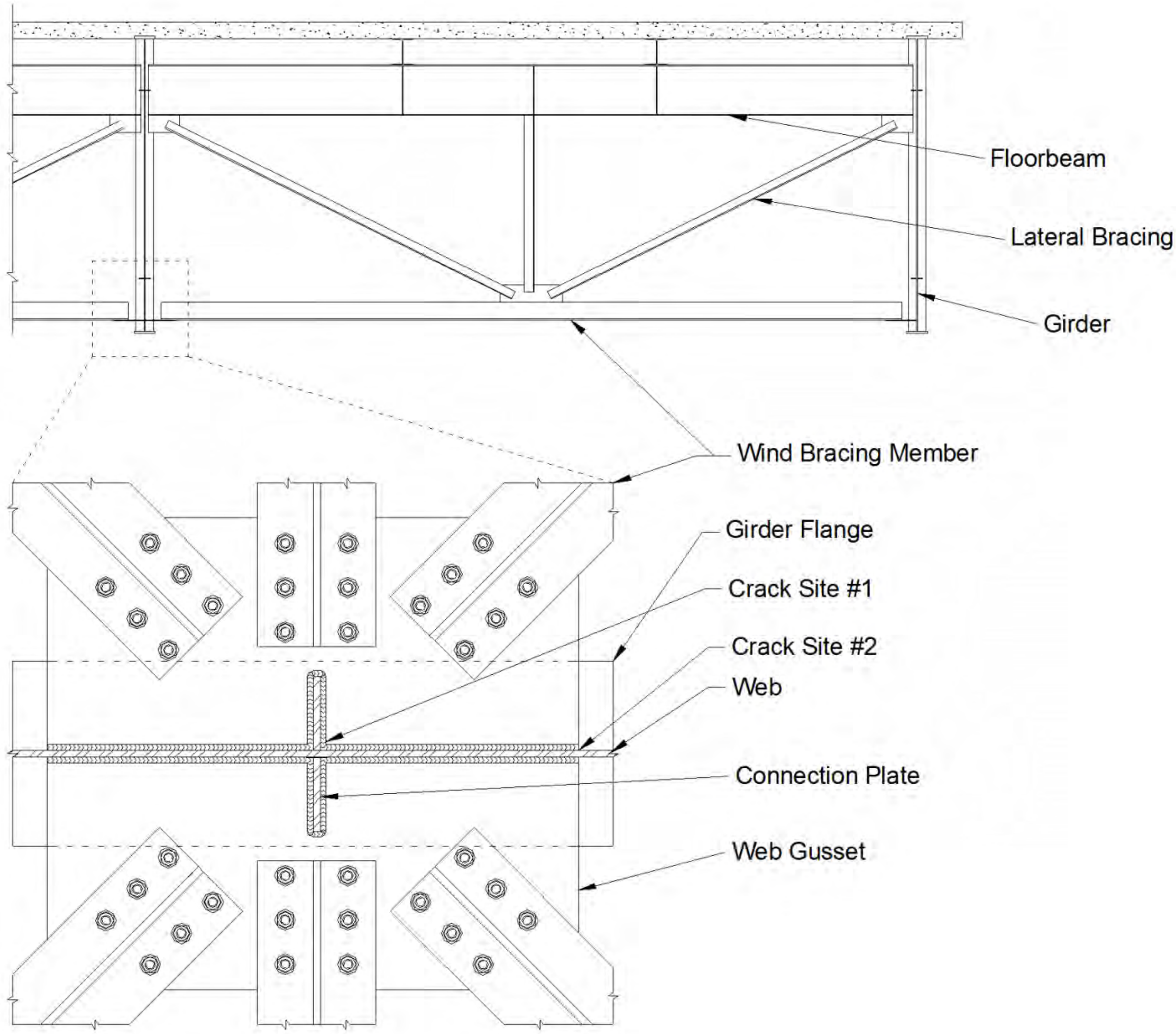


Figure 70. Illustration. Typical cross-section of a deep girder bridge (top) and plan view of the web gusset detail (bottom).

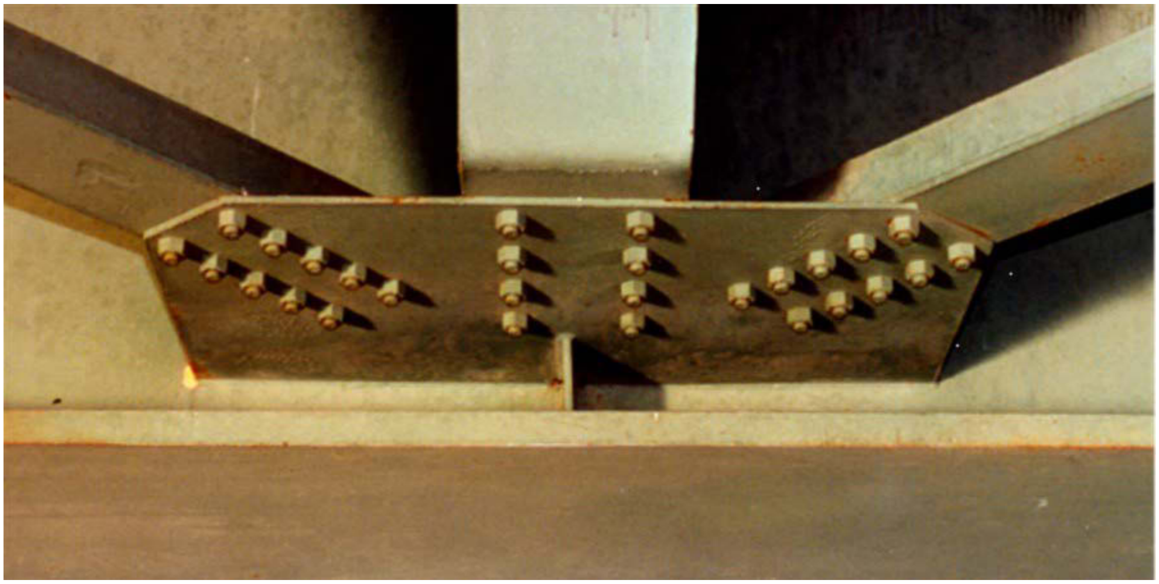


Figure 71. Photo. Web gusset with bracing members attached.

Crack Site #1 Retrofits

Gusset plates usually intersect transverse stiffeners, and in some cases, the two are connected via a continuous weld along the girder web, wrapping around the stiffener, and back out along the web. This creates an intersecting weld at the point where the girder web, gusset plate, and stiffener all come together. There are always lack of fusion defects at the root of intersecting welds that serve as crack initiation sites. Figure 72 shows a core extracted from an intersecting weld condition at a gusset plate, showing the severity of the root defect. Intersecting welds also create a triaxiality condition where the material is not allowed to yield, thus the hydrostatic state of stress increases the likelihood of fracture. It is important to note that in some cases no defects were found at the cracked detail as in the case of the Hoan Bridge. The cracked details were brittle in nature and there were no sign of fatigue crack initiation or growth.[19]

The retrofit of these details involves complete removal of the intersecting welds. This is completed by coring a hole tangent to the girder web and connection plate through the gusset plate, then hand grinding away the remaining material in the corner. Not only will coring remove weld root defects, it will also increase the unsupported length of the web in the detail and reduce the weld constraint to an acceptable level. A clear distance between welds of 0.25 inch (6.4 mm) has been shown sufficient in reducing the triaxiality condition in these details, and in the 2010 LRFD Bridge Design Specifications it was recommended for new designs to make the minimum distance 1.0 inch (25.4 mm).[1, 93] Though in practice, this distance should be increased with much larger holes to capture all the weld root defects. Figure 73 shows the detailing of this retrofit and a picture of a retrofitted detail is shown in Figure 74.

Figure 72. Photo. Core taken from intersecting welds between gusset plate, girder web, and connection plate. View of top of the gusset (left) and bottom of the gusset (right). Cores courtesy of Wiss, Janney, Elstner Associates.

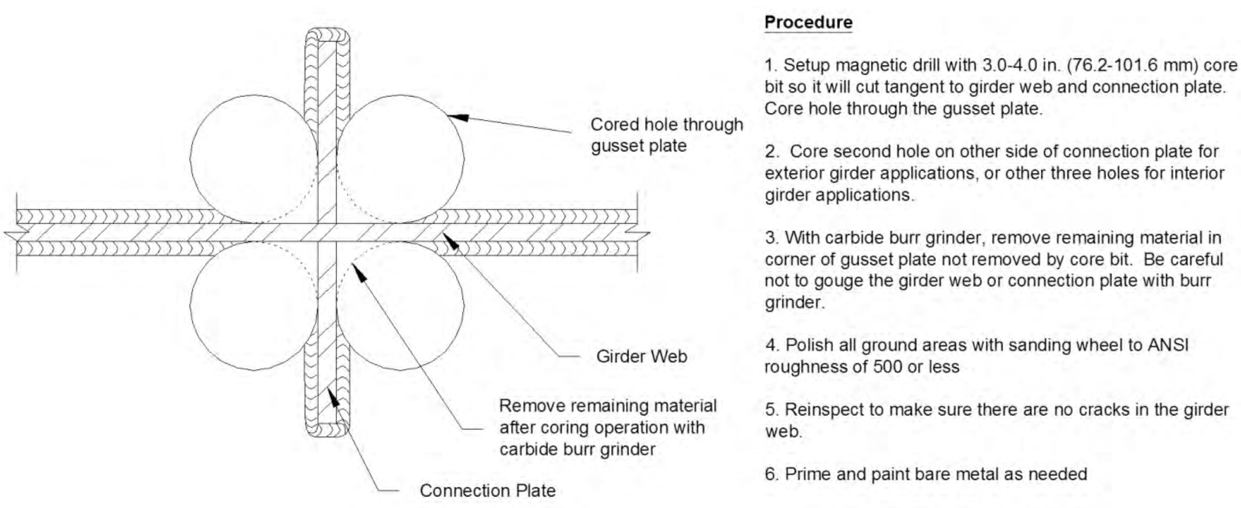


Figure 73. Illustration. Retrofit detail for gusset plates with intersecting welds.

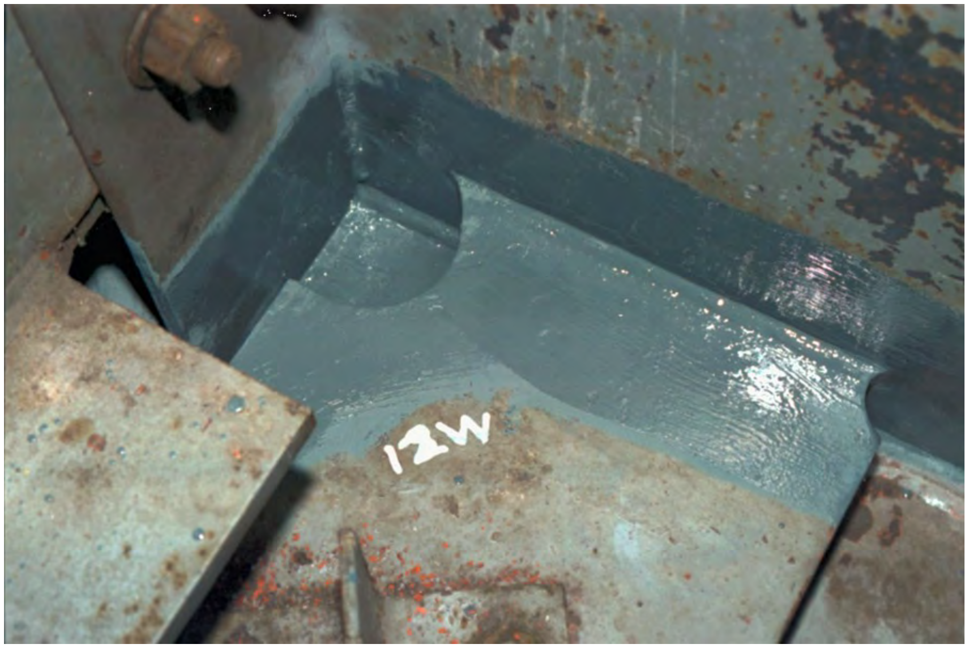


Figure 74. Photo. Completed retrofit of gusset plate weld intersection. Photo courtesy of Wiss, Janney, Elstner Associates.

Web Gaps of Gusset Plates

Instead of completely welding the gusset plate to the stiffener, sometimes the gusset plates were detailed with large cut-outs so the gusset plate never touches the stiffener. When the gusset plate is cut to fit around the connection plate, a small web gap is formed. Cracking from this detail is really a secondary stress phenomena from the alternating loads applied on the gusset plate from the bracing members. The bracing members are setup in a truss configuration, so any load in this truss decomposes into to alternating loads at each gusset plate as shown in Figure 75. This causes the gusset plate to rotate, but the stiffener plate is constrained from rotation since it is usually welded to the girder compression flange. The web gap is then forced into a highly stressed bent configuration, which can lead to cracking in the girder web. The retrofit of such a detail would entail providing a positive connection between the gusset and connection plates or lengthening the gap to reduce stress ranges. One way to stiffen the web gap is to use bolted angle sections as illustrated in Figure 76. All detailing procedures would be the same as outlined in Chapter 3 for stiffening or softening retrofits of web gap fatigue.

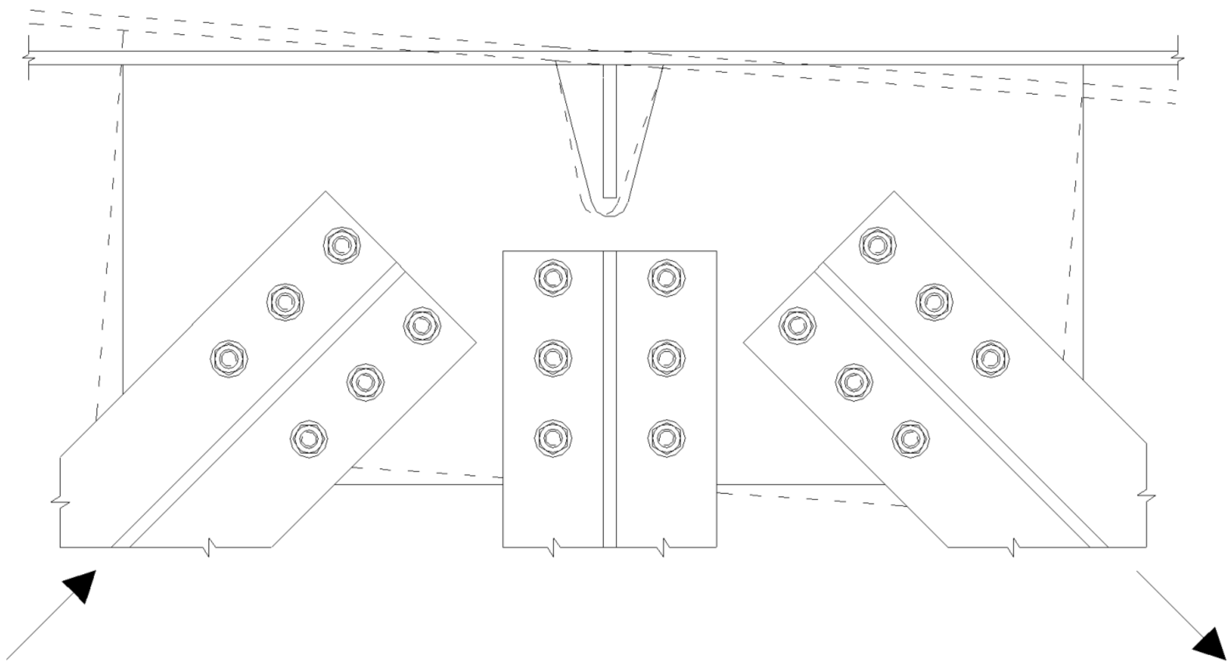


Figure 75. Illustration. Alternating loads from the bracing members cause the deformation of the web gap in shelf plate details.

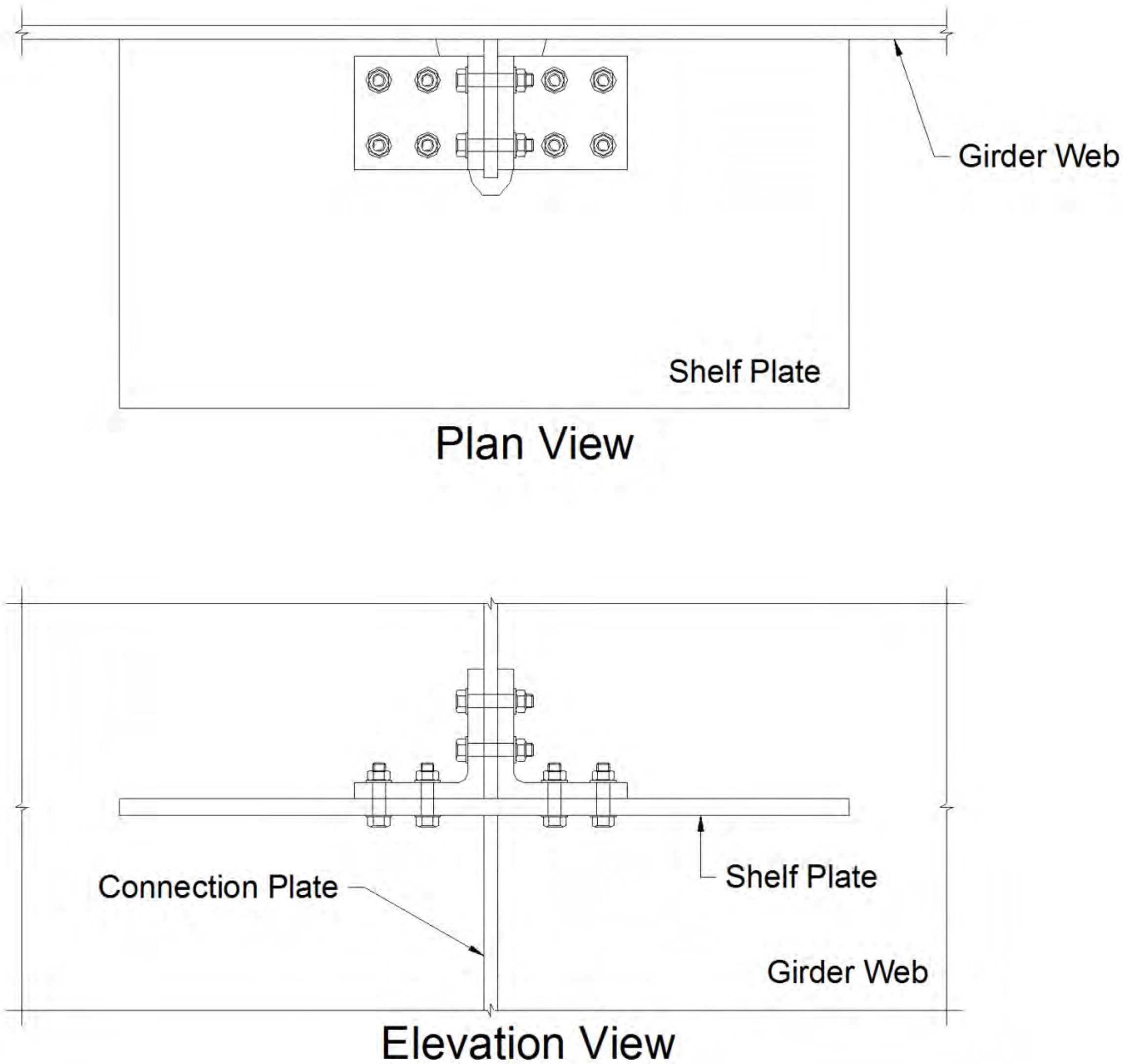


Figure 76. Illustration. Schematic of shelf plate web gap retrofit providing positive connection between the shelf and connection plates.

Crack Site #2 Retrofits

Cracks have been observed to emanate from the ends of the gusset plates and grow into the girder web. This is typically caused from poor workmanship where run-off tabs were not used, or not properly ground smooth after fabrication. If this type of cracking is occurring, impact treatments of the weld termination should be a sufficient retrofit to extend the life of uncracked details. Fillet welded gussets plates have been experimentally tested where hammer peening and GTA-remelt was explored as a retrofit strategy and were found to be effective considering the cracks were within the limits of the repair technique as outlined in Chapter 3.[65] An alternative retrofit would be to grind away the weld termination as shown in Figure 78. The termination typically contains many internal discontinuities where cracks can initiate, especially for full-penetration groove welds if a run-off tab was not used. Grinding this area away removes the defects.

If cracks originating from the weld termination extend into the web, then stop holes are required to isolate the crack tips as outlined in Chapter 3. If the crack severs the majority of the web, then stop hole should still be used, but doubler plates will be needed to restore the strength/stiffness of the girder.

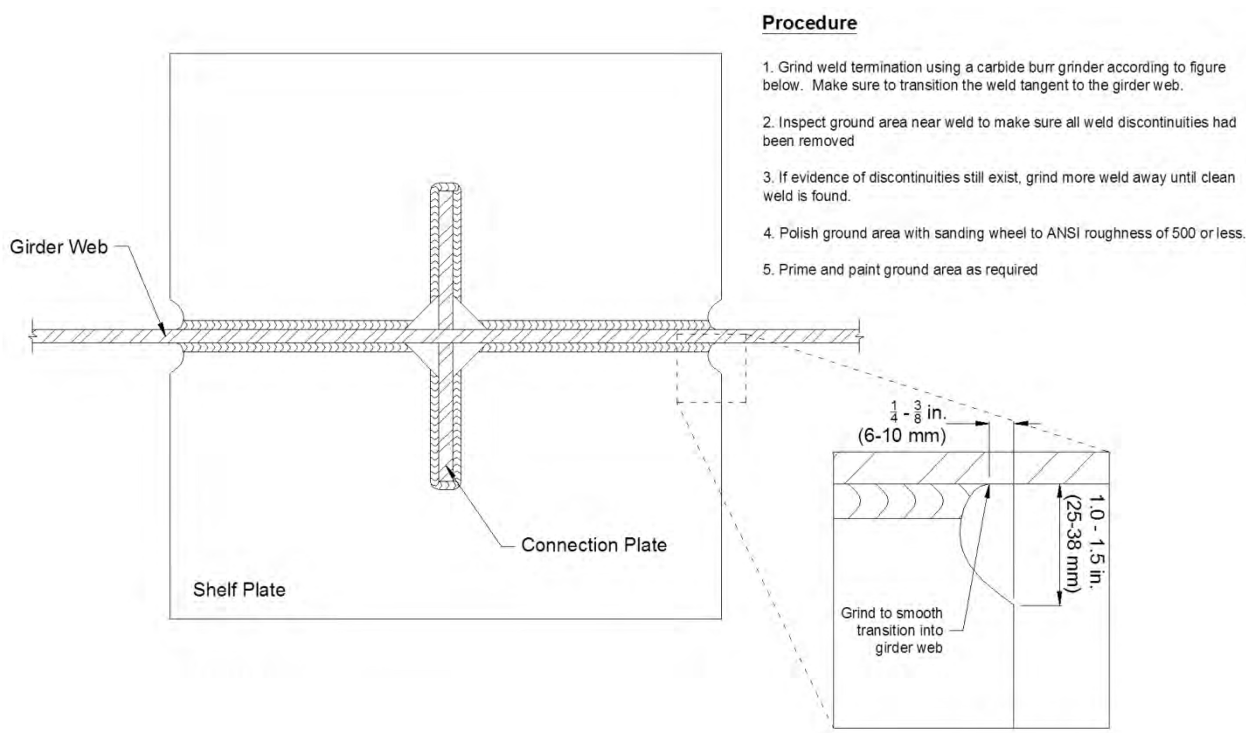


Figure 77. Illustration. Detailing of crack site #2 retrofit.

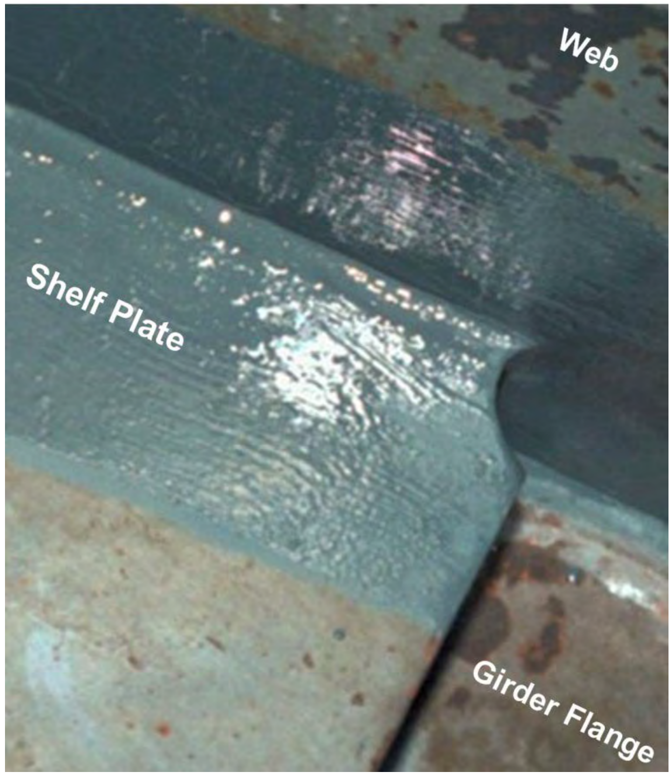


Figure 78. Photo. Picture of web gusset retrofit for crack site #2. Photo courtesy of Wiss, Janney, Elstner Associates.

LONGITUDINAL STIFFENERS

Longitudinal stiffeners have been prone to two different types of cracking. First are fatigue cracks, which grow in the butt weld between two adjoining longitudinal stiffeners. Figure 79 shows a typical butt weld crack in a longitudinal stiffener. In the early years of welding, the weld between a longitudinal stiffener and the girder web was carefully completed because it was thought to be critical, but the same care was not given to the butt weld splices of longitudinal stiffeners. Therefore, the weld quality was poor and internal defects served as crack initiation sites. Cracks in the longitudinal stiffeners themselves are benign, but because it is welded to the girder web, the crack can eventually grow into the girder web. Note that longitudinal stiffeners are used in compression regions of the girder (hence, cracks probably will not grow), except in circumstances when they were used in the tension zone of the girder as an architectural feature.

The means of retrofitting a cracked longitudinal butt weld would be to remove the path between the crack tip and the girder web. This can be accomplished by drilling a large diameter (diameter ≅ 50% of longitudinal width) through the longitudinal butt weld, and tangent to the girder web. Figure 80 shows the details of this retrofit and Figure 81 shows a picture of retrofitted longitudinal stiffener. As an alternative for girders with one-sided longitudinal stiffeners, an easier retrofit would be to core a hole 3.0-4.0 in. (76.2-101.6 mm) in diameter into the girder web adjacent to the butt weld adjoining the two side of the longitudinal stiffener. This alternative retrofit is schematically shown in Figure 82 showing how the hole breaks any continuity between the butt weld and girder web. The core penetrates through the girder web and partially through the longitudinal stiffener. The core can remain in place if corrosion protection of the bare steel surfaces is not a concern.

The second type of longitudinal stiffener cracking occurs at terminations of longitudinal stiffener welds, which is a Category E detail. Cracks propagate from the weld toe, into the girder web as shown in Figure 83. Hole drilling is the ideal retrofit for this cracking. This should only be needed as preventive maintenance in the vicinity of the girder inflection points as this is where most failures have been observed.

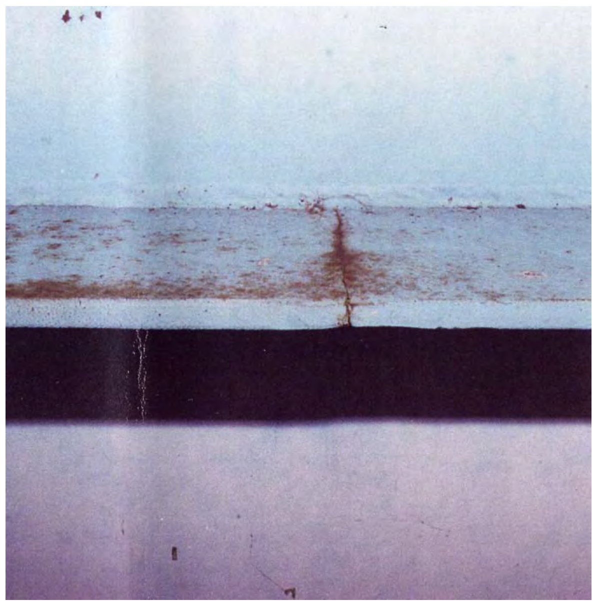


Figure 79. Photo. Cracked butt weld in longitudinal stiffener.

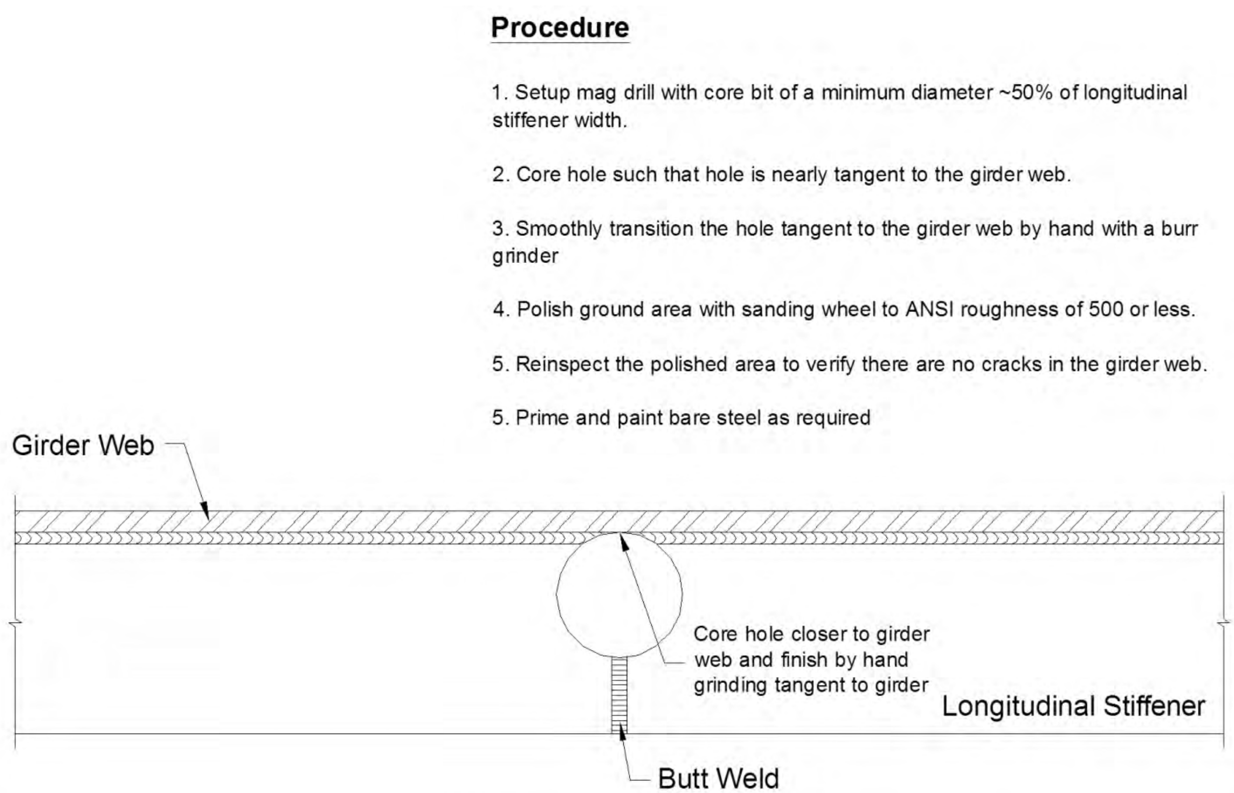


Figure 80. Illustration. Work plan of longitudinal butt weld retrofit.

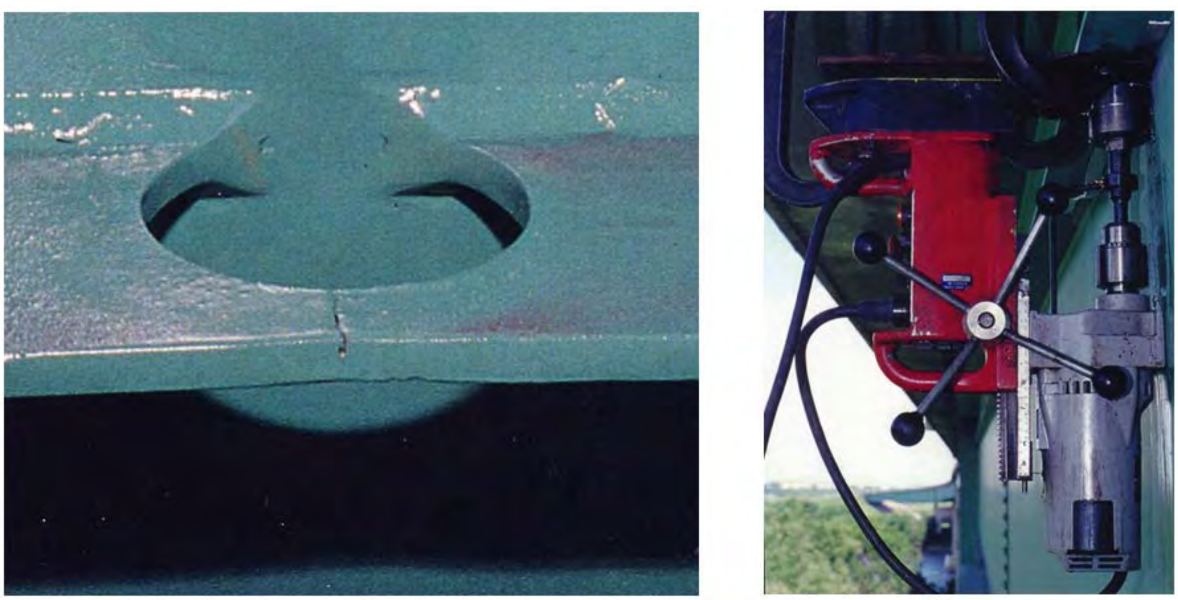


Figure 81. Photo. Completed core of longitudinal stiffener butt weld (left) and picture of magnetic-base drill coring out longitudinal butt weld (right). Photo courtesy of Wiss, Janney, Elstner Associates.

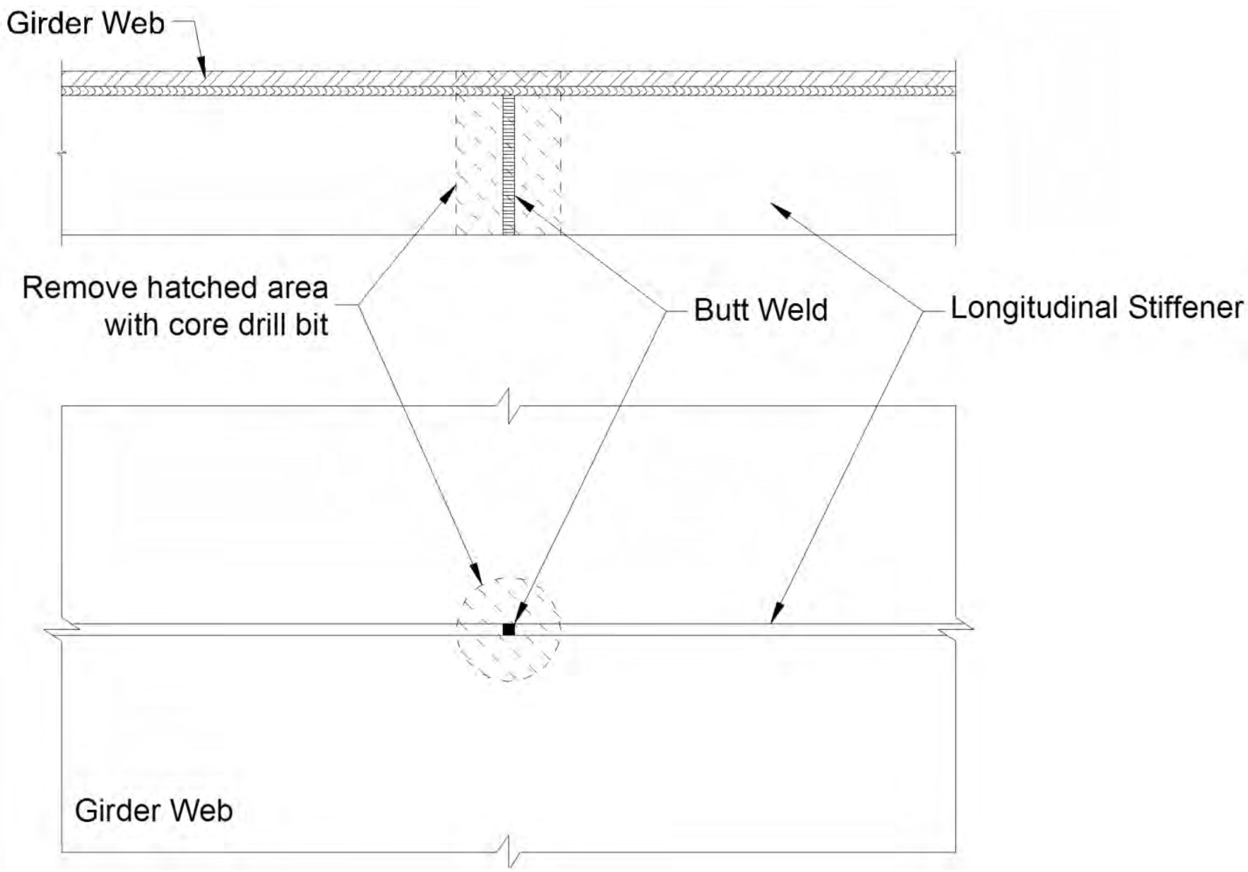


Figure 82. Illustration. Alternative detailing of longitudinal butt weld retrofit.

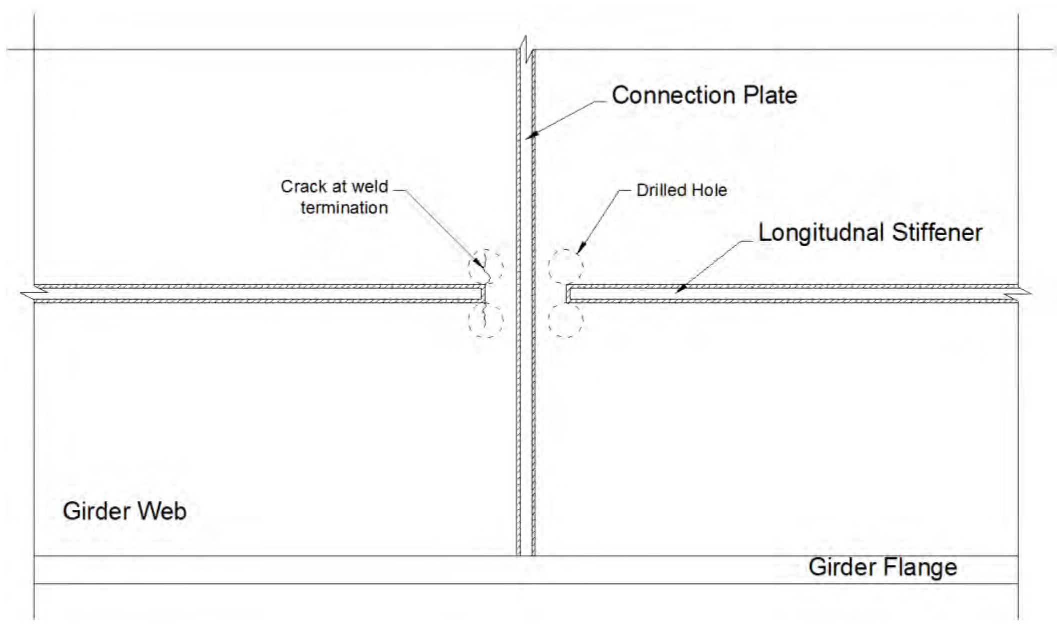


Figure 83. Illustration. Schematic of longitudinal weld termination crack and repair procedure.

COPED BEAM ENDS

Cracks have also been observed to grow from the ends of coped beams or beams with terminated flanges as shown in Figure 48 and Figure 84. This type of cracking can be evidence of either a stress riser from flame cut copes not ground smooth, or the presence of a bending moment at a connection designed as simply supported. The worst fatigue performance is noted with square cut copes (Figure 85, left) having less than Category E' resistance, but flame cut copes with a surface roughness of 250ST can attain Category D resistance.(66) For existing cracks, stop holes are effective in temporarily arresting cracks, but crack growth could be ceased by installing a tensioned bolt in the stop hole.(66) However, the stop hole with tensioned bolt will not prevent crack growth at other initiation sites along the cope. A more rigorous retrofit would require cutting back the cope slightly, giving a radius transition (or provide a larger radius is originally provided with one). Figure 85 shows the difference between good and bad detailing of a beam cope. The detailing of cutting in a better radius transition should use similar detailing as outlined in Figure 51, preferably with a radius greater than 2.0 in. (50.8 mm). A final alternative would be to remove bolts or rivets from the connection. This can only be done if strength can be preserved, but it reduces the connection stiffness thus lowering the stress range being transferred through the cope. However, this last retrofit would have to be experimentally verified case-by-case to ensure either cracking can be prevented or crack growth terminated.

Figure 84. Photo. Close-up view of a crack in a coped beam. Photo courtesy of John Fisher.

Figure 85. Photo. Bad coped beam detail with a sharp corner (left) and better coped detail with a smooth ground transition radius (right).

CHAPTER 6. DESIGN EXAMPLES

To fully understand and grasp the topics introduced in this manual, this chapter will be devoted to presenting design examples to show how different retrofit may be specified and how to determine the remaining fatigue life. Many of the measurements presented in this chapter are fictitious and may not fully represent actual behavior. In addition, the retrofits designed in each example may also not be the ideal retrofit, but rather the examples were created to condense many nuances of retrofit design to create a better understanding of the processes this manual presents.

EXAMPLE #1, DISTORTION-INDUCED FATIGUE AT CROSSFRAME DETAIL

A particular bridge owner experienced web gap fatigue in the negative moment region of a bridge after 25 years of service. The cracked bridge was a two span bridge with three continuous girders, laterally braced with cross-frames. An identical bridge existed in their bridge inventory, yet was five years younger and subjected to 80% of the ADTT of the cracked bridge. The owner wants to know if the uncracked bridge (shown in Figure 86) is also susceptible to fatigue cracking and hired a consultant to investigate.

The first thing the consultant did was apply instrumentation to common details in the uncracked bridge. Since the similar bridge experienced cracking in only the negative moment region, the negative moment region in the uncracked bridge became the focus. Shown in Figure 86 are two circles representing the web gaps to be instrumented. This location was ideal because the two points were symmetric and if the results showed cracking was likely, two retrofits were going to be tested.

Each web gap was instrumented with four strain gauges and one LVDT. The strain gauges were attached to the girder web as shown in Figure 87. The LVDT was used to measure the relative out-of-plane displacement between the top of the connection plate and girder flange, similarly shown in Figure 60 and Figure 61. At each web gap, two more strain gauges were adhered to the bottom of the girder flange, halfway between the girder web and flange tip, on each side of the web. Load tests were performed with heavily loaded sand trucks (type of truck is dependent up availability, but weight of 50 kips (222 kN) is typical). Slow crawl tests were performed by patterning the trucks to find the worst effect. On the north side of the bridge, there was 0.04 in. (1.02 mm) of relative displacement between the connection plate and girder flange, and was 0.033 in. (0.84 mm) on the south side. Since the gauged locations were symmetrically placed in the bridge plans, the maximum strain readings were also approximately the same. The maximum measured strains are summarized in Table 3. The strains measured in the flanges are inconsequential, for the rest of this analysis because the stress ranges are so low. However, the maximum local stress range measured at the girder web/flange weld was 20.1 ksi (138.6 MPa). This is a Category C detail since the local strain is at a weld toe, with a CAFL of 10 ksi (68.9 MPa), hence infinite life would not be expected. The gauges below these are measuring base metal stresses, which can be compared to a fatigue resistance of Category A. The CAFL for Category A is 24 ksi (165.5 MPa), which is higher than the stress ranges measured on both the north and south side of the bridge. Because of the history of cracking in the other bridge and measured stress ranges higher than the CAFL at the girder web/flange weld, cracking was likely and investigations of two retrofits began.

Table 3 Maximum Strains Recorded during Truck Tests

Gauge Location	Axial Strain Range (με)	Bending Strain Range (με)	Total Stress Range[a] (ksi / MPa)
North Flange[b] (Location 1 in Figure 87)	100	2	3.0 / 20.4
North Upper web gap (Location 2 in Figure 87)	143	550	20.1 / 138.6
North Lower web gap (Location 3 in Figure 87)	98	600	20.2 / 139.6
South Flange[b]	97	1	2.8 / 19.6
South Upper web gap	101	527	18.2 / 125.6
South Lower web gap	111	598	20.6 / 141.8

[a] – computed as addition of axial and bending components times modulus of steel
[b] – bending component of flange is across the flange width (i.e. lateral flange bending)

It was decided to use a large hole retrofit on the south side of the bridge. A 4.0 in. (101.6 mm) diameter hole was drilled into the girder web with the detailing shown in Figure 46. A stiffening retrofit was used on the north side of the bridge using a WT9x79, bolted to the connection plate and nailed to the girder flange. Because the bridge was highly skewed, the tee was designed to resist 30 kips (133.4 kN) of force. The nail manufacturer publishes a shear resistance of 0.8 kip/nail (3.6 kN/nail), therefore a minimum of 38 nails must be shot through the flange of the tee into the girder flange.

The instrumentation on the north side of the bridge was undisturbed with the tee installation and was reusable for the second load test. The instrumentation was removed as a consequence of coring the web for the large diameter retrofit on the south side. The new instrumentation plan called for five strain gauges to be installed. Three gauges would be installed tangent to the hole and two gauges would read the local strains near the web/connection plate weld, and flange/web weld as depicted in Figure 88. The same slow crawl load tests were conducted and the following were found:

- On the north side, the relative displacements between the connection plate and girder flange reduced to 0.034 in. (0.86 mm). The strain ranges reduced by 55%.

- On the south side, the worst strain range recorded by the five strain gauges on the hole perimeter was 327 $\mu\varepsilon$, or a 9.5 ksi (65.5 MPa) stress range.

Since no cracks have formed in this bridge yet, it is hard to determine a remaining life exactly. However, since an identical bridge, under heavier loading, cracked after 25 years of service, that life can be conservatively used in life calculations of the investigated bridge. The stiffening retrofit on the north side of the bridge only remedied the crack driving force but small cracks could still be present. The stiffening retrofit reduced the stress ranges by 55%. Since fatigue life is proportional to the cube of the stress range, the remaining life can be multiplied by $(1/0.55)^3=6$. The total fatigue life is 25 years, however, 20 have been consumed prior to retrofitting. Therefore, the remaining 5 year fatigue life of retrofitted details can be multiplied by 6, or fatigue cracking would not be expected for another 30 years after retrofitting. As for the large diameter hole retrofit, the maximum stress ranges around the hole are less than the Category C CAFL, so infinite life could be expected. In addition, if cracks were to form from the remaining steel left in the web gap, the large holes should intercept the cracks.

Because of the ease and cost of performing the large diameter retrofit, it was specified for the remainder of the web gaps in the negative moment region.

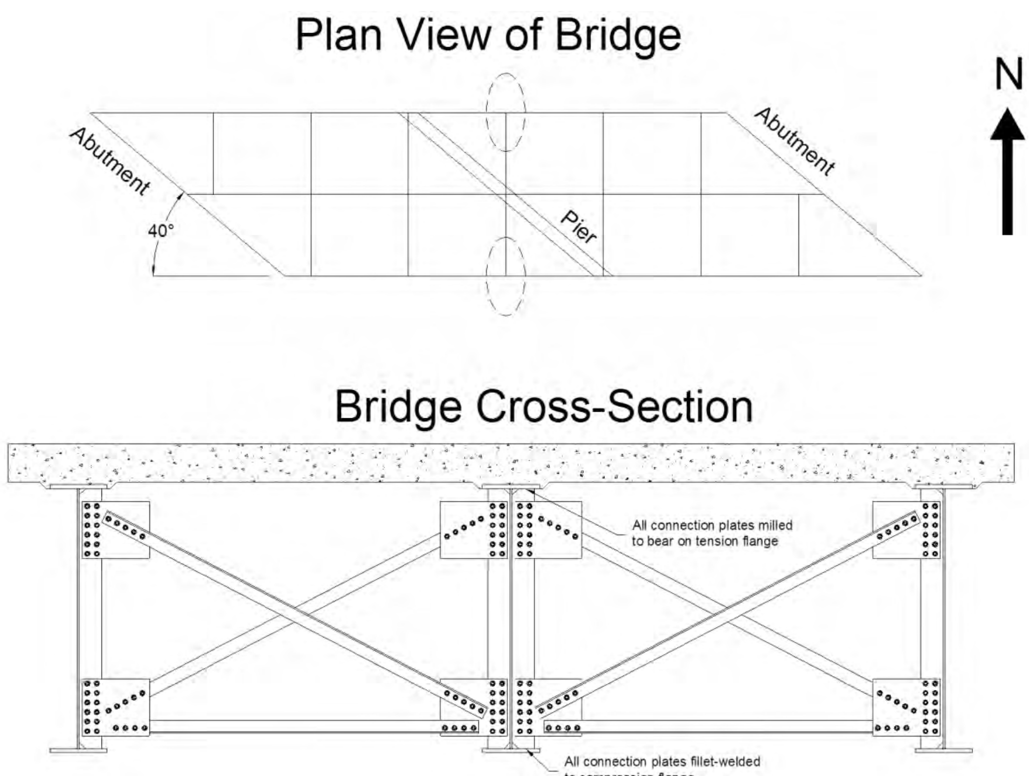


Figure 86. Illustration. Bridge plan for design example #1.

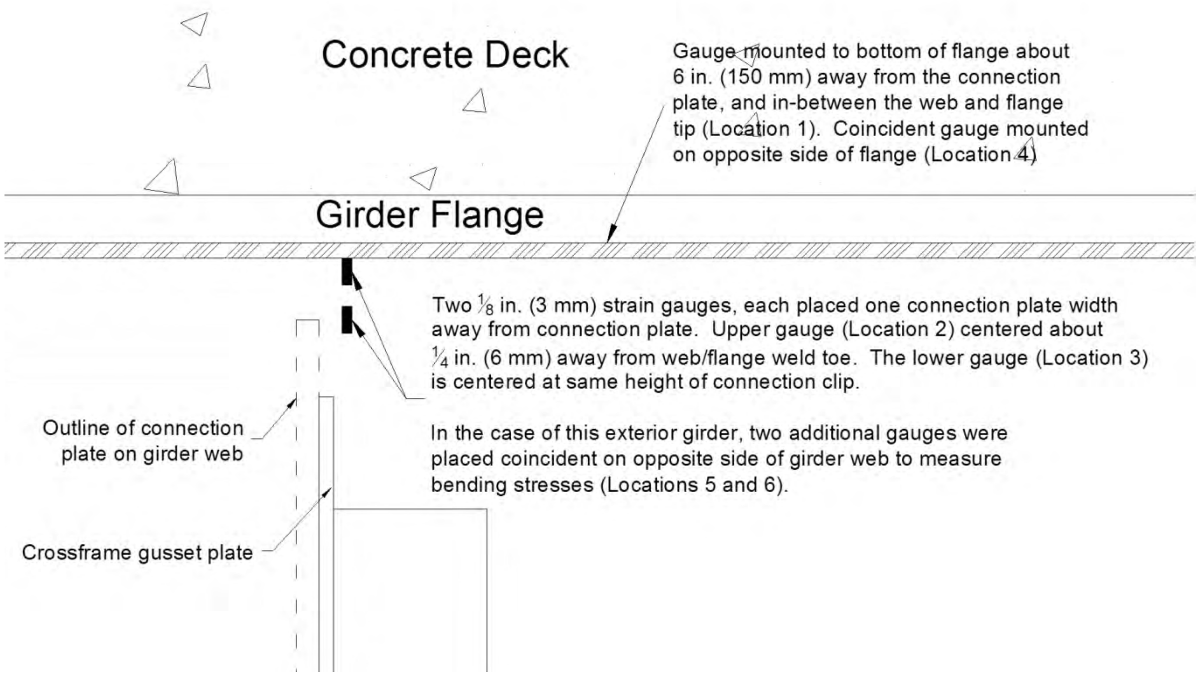


Figure 87. Illustration. Instrumentation plan for design example #1.

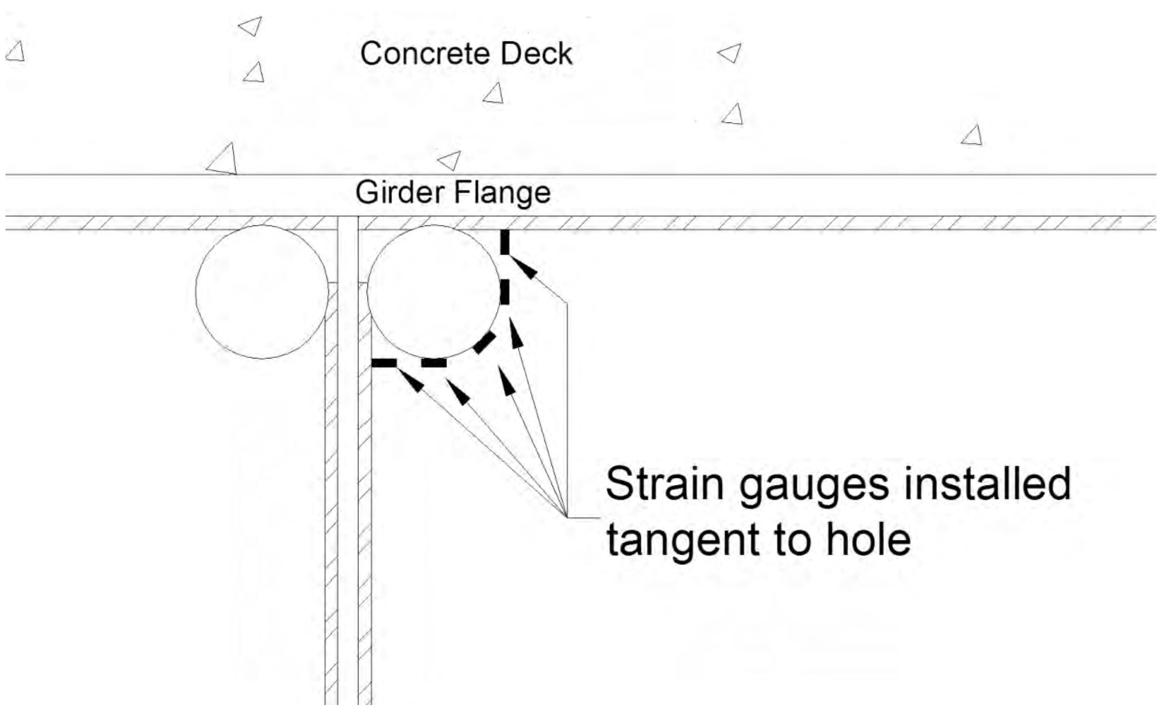


Figure 88. Illustration. Large diameter hole instrumentation for design example #1.

EXAMPLE #2, ANALYSIS OF A STEEL, BOX GIRDER PIER CAP

The following example is from a field study performed by the University of Minnesota on Mn/DOT bridge 69832, which carries I-35 through Duluth, MN.[12] This is a valuable example because it shows how system behavior knowledge can explain why cracks occur and why not all cracks must be repaired.

In situations where a concrete pier cannot be set directly beneath the reaction points from a girder, a steel pier cap is used to carry the loads out to a point where a concrete column can be erected. Ideal designs would just have the steel girders set directly upon the top of the steel girder. In the case of Bridge 69832, the bridge girders have to partially pass through the pier cap because of clearance issues over the railroad line the pier cap spans over, this is shown in Figure 89. Shown in Figure 90 is a cross-section of the pier cap showing how the girder only passes through half the pier cap. This detailing requires the pier cap web to be stiffened beneath the girder flange, ideally with a diaphragm that spans from each web of the pier cap. However, for inspection purposes, only two smaller stiffeners were specified so an inspector could pass beneath the opening under the girder flange. Fatigue cracking was observed at the weld attaching the stiffener to the girder flange within the box.

Such cracking is not intuitive and grillage models were created to better understand the global behavior of the structure. The modeling found there is a "soft-story" effect happening in the lower cell of the pier cap, at the cross-section where the girder passes through. As the girder rotates from vehicular traffic it imparts a torquing moment on the pier cap. The top cell of the pier cap coincidently rotates because of the displacement compatibility with girder web. However, the lower cell is not as stiff because the stiffeners are not full-width across the pier cap, and to exacerbate the problem, the stiffeners were not welded to the bottom flange (or tension flange) of the pier cap. This caused the lower cell to deform much more than the upper cell, leading to large axial loads in the stiffener, hence cracking along the weld atop the stiffener.

A variety of instrumentation was used to verify this behavior. Many strain gauges were placed on the gross section of the approaching girders and pier cap to understand how the load is distributed through the system. The unique instrumentation can be seen in Figure 92 where internally, four LVDTs (two in the top cell and two in the lower cell) to measure the shear deformation of each cell of the pier cap. The shear deformation can be measured by monitoring the diagonal deformation of the cell. Two more LVDTs measured the relative displacement between the stiffener and the bottom flange of the pier cap because the stiffener was not welded at this location.

There were two retrofit options that could be used. The first was a stiffening retrofit where a stiffened angle could be bolted on the pier cap exterior between the girder flange and pier cap web. This would promote more even shearing of the two internal cells of the pier cap to alleviate the axial stresses in the stiffeners. However, this retrofit is expensive to implement and may not work. The recommend retrofit option was to do nothing, and only monitor the cracks. The cracks are growing along the weld toe of the stiffener where it joins to the bottom of the girder flange. But the stiffener had clipped corners and the girder/stiffener weld does not intersect the pier cap web. This makes the crack benign because there is no continuity, and the crack cannot propagate into the pier cap web.

The lesson learned from Bridge 69832 is to truly understand why fatigue cracking is occurring; system behavior above and beyond design simplifications must be fully understood. In this bridge's aspect, torsional rotations of the pier cap caused high axial stresses in a stiffener detail, thus causing cracking. Fortunately, the cracking was self-contained within the particular detailing of this bridge, and no retrofit was needed other than continued monitoring.

Figure 89. Photo. Picture of MnDOT bridge #69832 in Duluth, Minnesota.

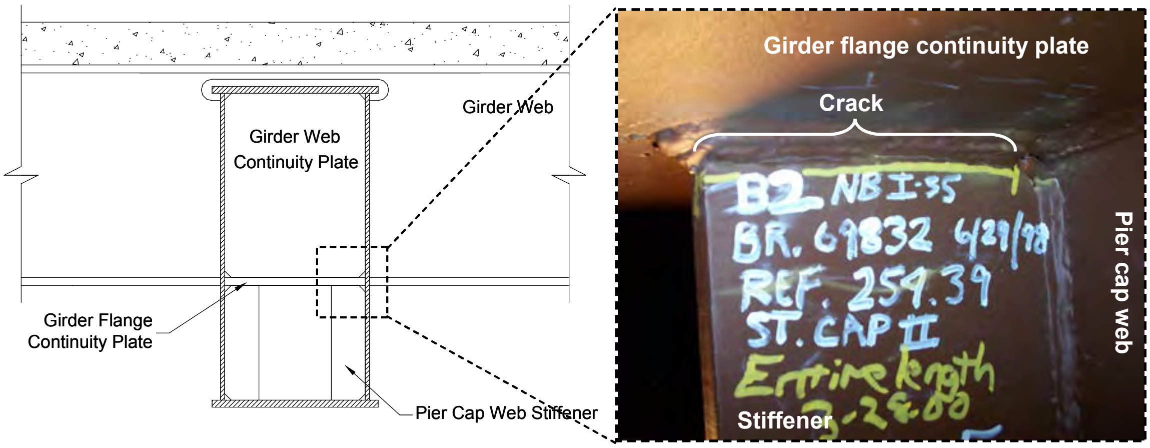


Figure 90. Illustration. Location of cracks in pier cap.

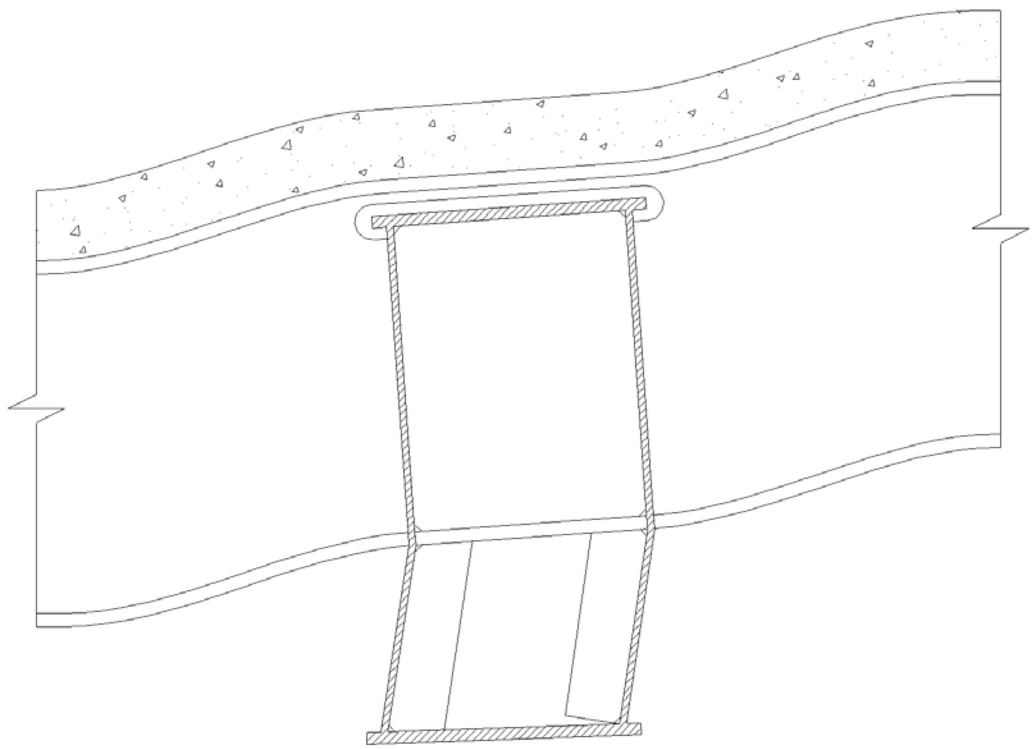


Figure 91. Illustration. Exaggerated deformation mode of pier cap.

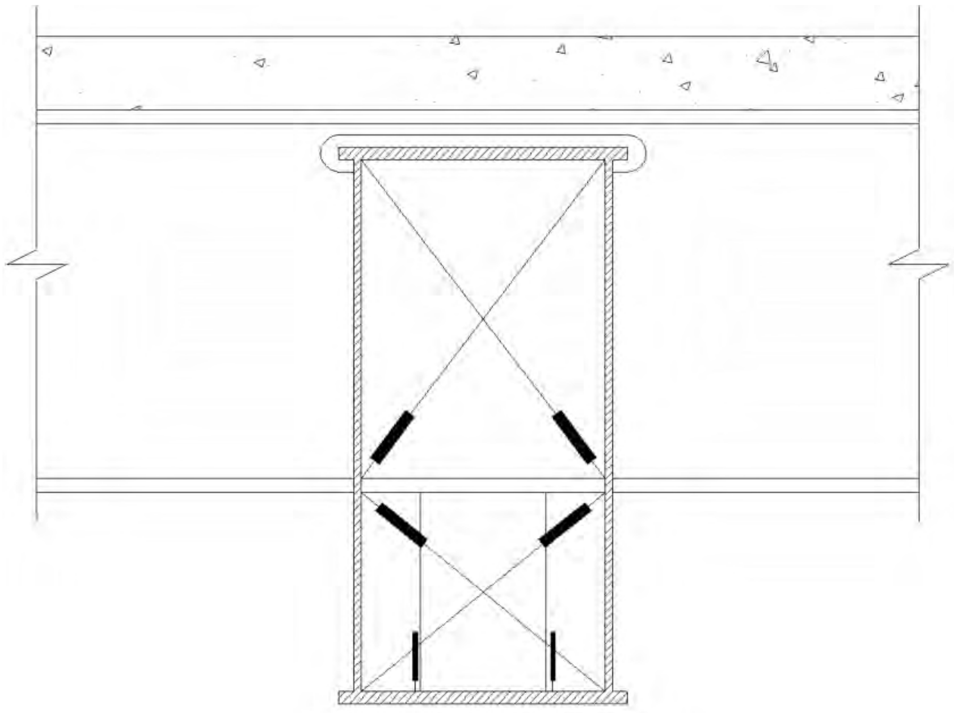


Figure 92. Illustration. Layout of LVDT instrumentation inside pier cap.

EXAMPLE #3, FLANGE CRACKING WITH COVER PLATED BEAM

A bridge is composed of eight, 98.5 ft. (30 m), simply-supported spans. In each direction of traffic, there are five coverplated W36x150 rolled beam sections. After 25 years of service, two types of fatigue cracks were found during an inspection interval.

Surface cracks were noted in the weld toe of a coverplate weld and were estimated to be 0.06 in. (1.6 mm) deep. Strain gauges verified that stress ranges in the unstiffened girder was 3.9 ksi (36.9 MPa) while the stress range in the coverplated portion was 2.3 ksi (15.9 MPa). In this case, hammer peeening the weld toes will close-up the surface cracks and make the coverplate terminations Category E details. Since the nominal stress range is 3.9 ksi (36.9 MPa), this is below the Category E CAFL, so no more cracking should be expected for the remainder of the bridge's life.

The second type of crack initiated at the coverplate termination, but at inspection, the crack was observed to have penetrated about halfway through the flange thickness. Since the depth of the crack is greater than what can be repaired using surface treatments, splice plates will have to be designed and installed. First assume the flange is completely severed and calculate the required area of the splice plates needed to restore the lost area. Assume the splice plate will be made from the same strength material as the girder.

$$A_f = (11.975in) \cdot (0.94in) = 11.25in^2$$

Since bolts will be used to attach the doubler plates, and extra area of plate needs to be added on to account for the net section loss due to the bolt holes in the girder flange. Assume two rows of 22mm (7/8 inch) bolts will be used so:

$$A_{plates} = (11.25in^2) + (2)(0.9375in)(0.94in) = 13in^2$$

Use two 4.0 x 0.75 in. (101.6 x 19.1 mm) plates above the flange and one 10.0 x 1.0 in. (254.0 x 25.4 mm) plate below the flange which gives a cross-sectional area of the plate to be 13.0 in.^{2} (8387 mm^2). Determine the number of A490 bolts for a slip critical connection to develop the strength of the doubler plates.

$$\#bolts = \frac{(11.25in^2)(50ksi)}{.25\pi(0.875^2)(21ksi)(2\ slip\ planes)} = 22.3\,bolts$$

To maintain a symmetric bolt pattern, two rows of 12 bolts will be required on each side of the crack. This splice plate detail will be able to transmit the flange force even if the flange is completely severed from continued fatigue crack growth. However, this retrofit does not prevent the crack from entering the web. To isolate the crack, a preventive stop hole (minimum of 1.0 in. (25.4 mm) diameter) is drilled into the girder web, as close as possible to the girder flange, so when the crack does penetrate into the web, it will be stopped. The final detail is outlined in Figure 93.

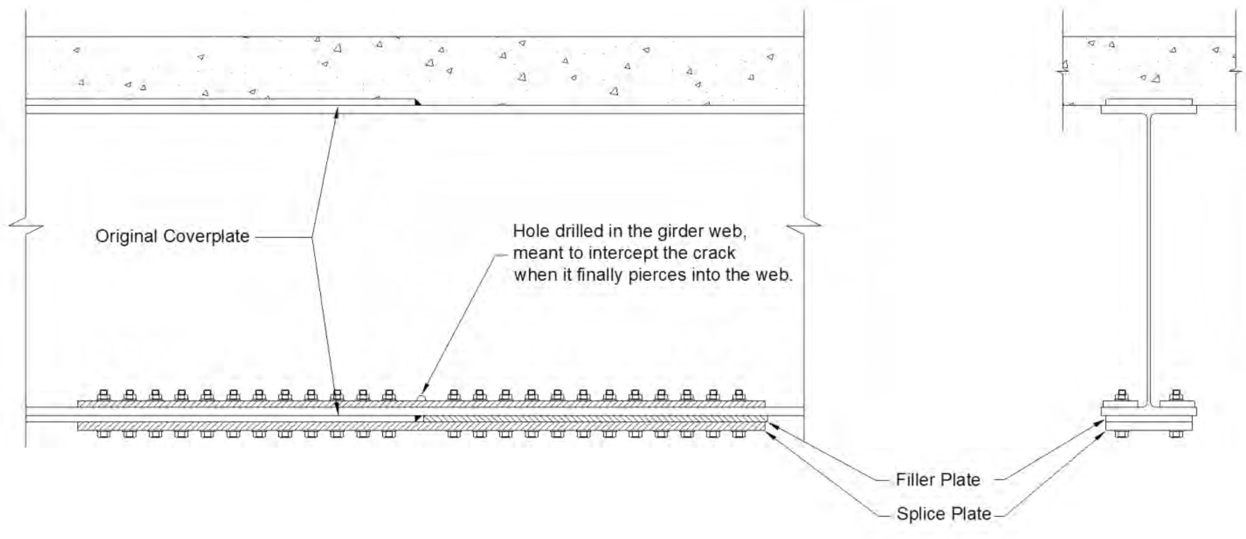


Figure 93. Illustration. Final detailing of splice plate repair.

EXAMPLE #4, POSITIVE MOMENT DISTORTION-INDUCED FATIGUE

Pictured in Figure 94 is a cracked girder in a multi-girder bridge. The girders are connected transversely across the bridge with bent plate diaphragms, which are bolted to the connection plates. The connection plates are not welded to the bottom tension flange of the girder. Though not common, cracks have been found in the positive moment region, growing from the termination of the connection plate/web weld. As shown in Figure 94 are two cracks, a typical horseshoe shaped crack growing to each side of the connection plate, and the second originated from the same spot, but is growing straight down towards the girder flange. There was great concern about the crack growing towards the flange and all crack tips were immediately specified to be drilled out. Drilling out the crack tip of the crack growing towards the girder flange was difficult because two connection plates are present on each side of the web. To properly drill out this crack tip, two 1.0 in. (25.4 mm) holes had to be drilled crossing each other at a 45 degree angle as shown in Figure 95. Depending on what is connected to the connection plates, it may be possible to cut back one of the plates to allow for a drill bit to remove the vertical crack head on. Both tips of the horseshoe crack were drilled out with 1.0 in. (25.4 mm) holes. This temporarily ceases crack growth because typically stop holes do not work by themselves as a retrofit for distortional-induced fatigue. This allows for long-term monitoring to determine the live load stress ranges and investigate if the connection plate could just be welded to the lower girder flange.

Welding the connection plate to the girder flange would make the girder flange a Category C detail, which has a CAFL of 10.0 ksi (68.9 MPa). To verify this retrofit, a simple instrumentation plan installed one strain gauge on the top side, of the lower girder flange, about 3.0 in. (76.2 mm) from the connection plate. The output from the strain gauges was monitored with a data collection system for a two week period, and cycles were counted with a rainflow counting algorithm. Knowing the CAFL is 10.0 ksi (68.9 MPa), the rainflow counter was setup up to neglect all stress ranges lower than 25-33% of the CAFL (2.5-3.5 ksi (17.2-24.1 MPa) in this example) to correlate with the full-scale variable amplitude testing.[39] The rainflow algorithm was set up to neglect all cycles less than 3.0 ksi (20.7 MPa), and to count and sort cycles into 1.0 ksi (6.9 MPa) bins. The benefit of collecting rainflow data is histograms can be constructed to determine the percent exceedance of the CAFL stress range. Table 4 shows the rainflow data collected from the strain gauge attached to the girder flange. Summing the percentage of total cycles from the stress range bins greater than the CAFL is only 0.0098%. Since the percent exceedance is less than 0.01%, infinite life could be expected from this welded retrofit. The connection plate was then welded to the girder flange with the procedure outlined in Figure 34. Welding the connection plate the girder flange eliminates all out-of-plane displacements, hence the crack arrest holes will be sufficient to cease further crack growth.

Table 4 Stress Ranges Collected Through Rainflow Counting for Design Example #4

Stress Range Bin (ksi)	Number of Cycles	Percentage of Total Cycles
1.0-2.0	-	-
2.0-3.0	-	-
3.0-4.0	48213	42.9071%
4.0-5.0	37759	33.6036%
5.0-6.0	17043	15.1674%
6.0-7.0	6270	5.5800%
7.0-8.0	2842	2.5292%
8.0-9.0	186	0.1655%
9.0-10.0	42	0.0374%
10.0-11.0	5	**0.0044%**
11.0-12.0	4	**0.0036%**
12.0-13.0	1	**0.0009%**
13.0-14.0	0	**0.0000%**
14.0-15.0	0	**0.0000%**
15.0-16.0	0	**0.0000%**
16.0-17.0	1	**0.0009%**
17.0-18.0	1	**0.00040%**
18.0-19.0	0	**0.00000%**
19.0-20.0	0	**0.00000%**

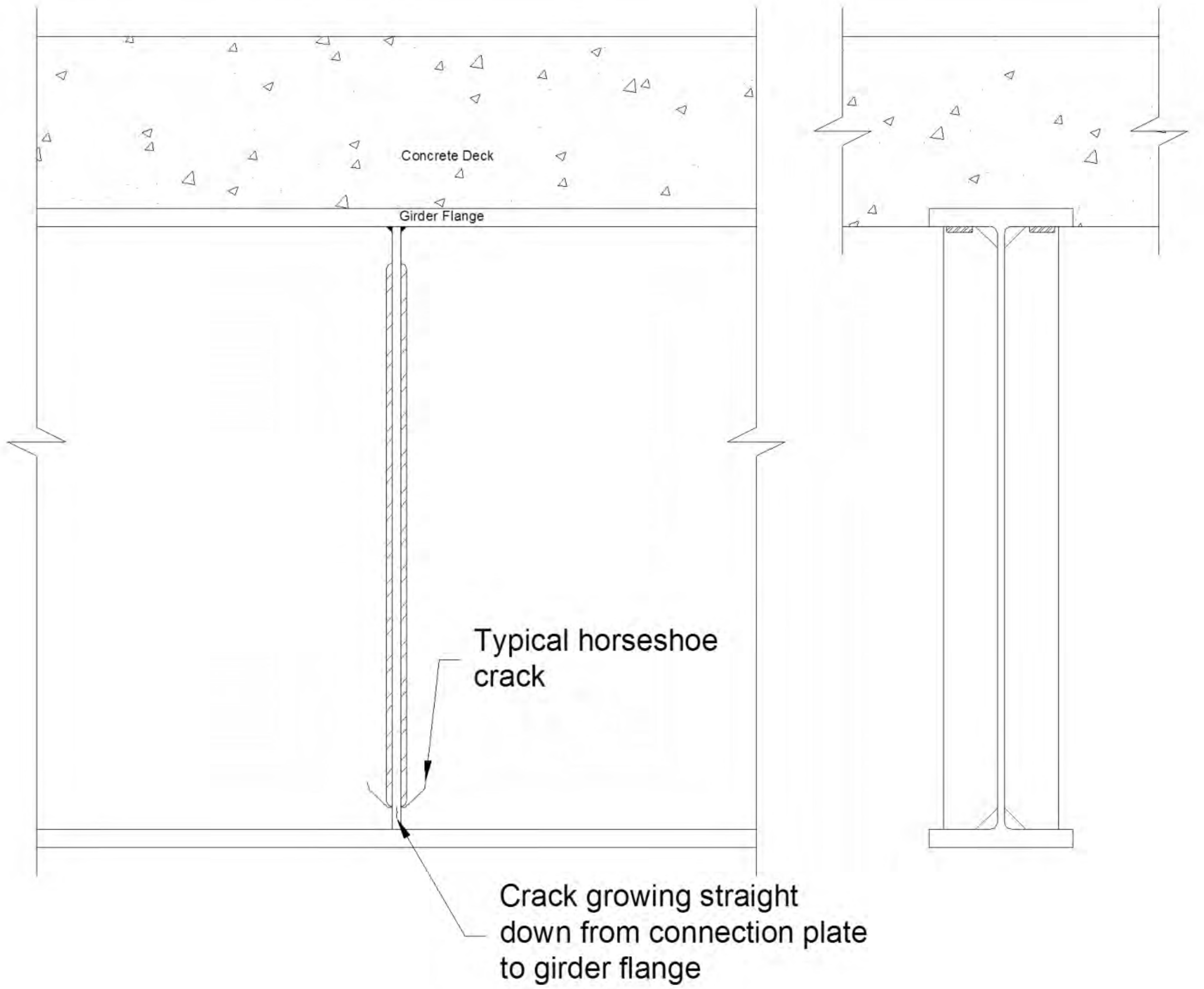


Figure 94. Illustration. Cracking in girder of design example #4

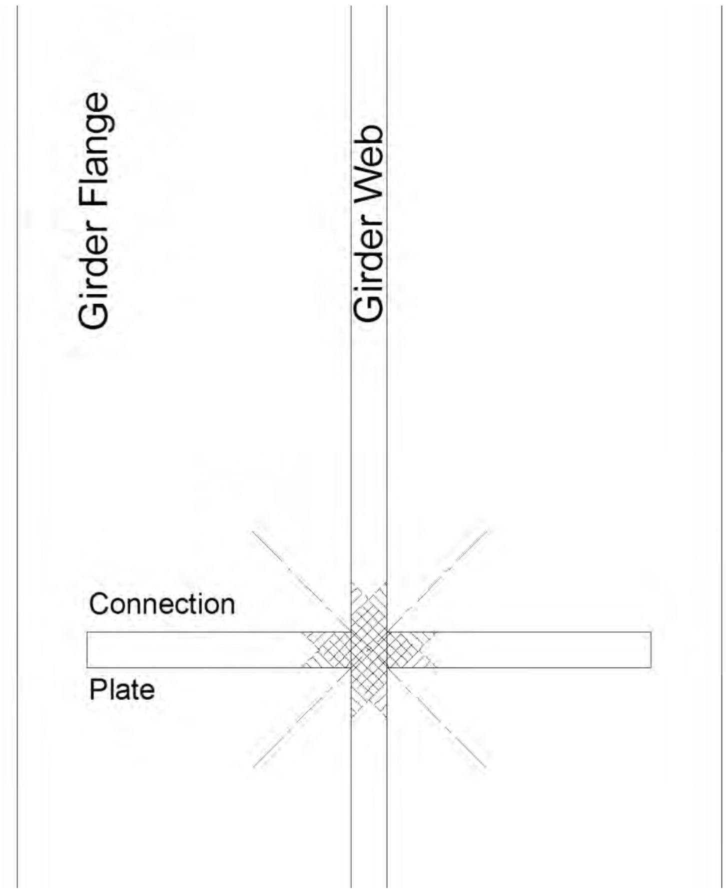


Figure 95. Illustration. Crack tip drilling plan of vertical crack of design example #4.

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