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Chapter 8
Inspection and Evaluation of Common Steel Superstructures

Section 8

Inspection and Evaluation of Common Steel Superstructures

Topic 8.1 Fatigue and Fracture in Steel Bridges

8.1.1 Introduction

Since the 1960's, many steel bridges have developed fatigue induced cracks. Although these localized failures have been extensive, only a few U.S. bridges have actually collapsed as a result of steel fatigue fractures.

The first collapse was the Silver Bridge over the Ohio River at Point Pleasant, West Virginia on December 15, 1967. This structure was an eyebar chain suspension bridge with a 213 m (700-foot) main span that collapsed without warning and forty-six people died (see Figure 8.1.1). The collapse was due to stress corrosion and corrosion fatigue that allowed a minute crack, formed during casting of an eye-bar, to grow. The two contributing factors, over the years continued to weaken the eye-bar. Stress corrosion cracking is the formation of brittle cracks in a normally sound material through the simultaneous action of a tensile stress and a corrosive environment. Corrosion fatigue occurs as a result of the combined action of a cyclic stress and a corrosive environment. The bridge's eye-bars were linked together in pairs like a chain. A huge pin passed through the eye and linked each piece to the next. The heat-treated carbon steel eye-bar broke, placing undue stress on the other members of the bridge. The remaining steel frame buckled and fell due to the newly concentrated stresses.

Figure 8.1.1 Silver Bridge Collapse

The second collapse occurred on June 28, 1983, when a suspended two-girder span carrying I-95 across the Mianus River in Greenwich, Connecticut failed (see Figure 8.1.2).

Figure 8.1.2 Mianus River Bridge Failure

The above catastrophes were a result of fatigue cracking to the point of failure of a fracture critical member. For bridge inspectors, understanding the causes of the common member failure modes is of utmost importance. This understanding permits the inspector to use more time evaluating the trouble areas of a bridge and less time on the others.

When inspecting steel bridges, the inspector must be able to identify a fracture critical member by sight or based on previous reports and drawings. The National Bridge Inspection Standards (NBIS) require that all fracture critical members on a bridge be identified, and the inspection procedures listed prior to an inspection.

Fracture Critical Member

A fracture critical member (FCM) is a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse. Bridges that contain fracture critical members are fracture critical bridges.

Fatigue

Fatigue is the tendency of a member to fail at a stress level below its yield stress when subject to cyclical loading.

Fatigue is the primary cause of failure in fracture critical members. Describing the process by which a member fails when subjected to fatigue is called failure mechanics.

Reviewing Member Forces

For a bridge member to be classified as fracture critical, it must meet two criterions. The first criterion deals with the forces in the member. Members that are in tension or members that have fibers or elements that are in tension meet the first criterion. The five types of member forces are discussed in Topic P.2.3 and include:

- Axial tension – Acts along the longitudinal axis of a member and tends to "pull" the member apart
- Axial compression – Acts along the longitudinal axis of a member and tends to "push" the member together
- Shear – Equal but opposite transverse forces which tend to slide one section of a member past an adjacent section producing diagonal tension oriented 45 degrees to the longitudinal axis
- Flexure – Caused by moment commonly developed by the transverse loading of a member producing both tension and compression
- Torsion – Results from externally applied moments that tend to twist or rotate the member about its longitudinal axis producing diagonal tension present on all surfaces of the member

Redundancy

The second criterion for a bridge member to be classified as fracture critical is that its failure must cause a total or partial collapse of the structure. Therefore, recognition and identification of a bridge's degree of redundancy is crucial.

Redundancy is defined as a structural condition where there are more elements of support than are necessary for stability.

Redundancy means that should a member or element fail, the load previously carried by the failed member will be redistributed to other members or elements. These other members have the capacity to temporarily carry additional load, and collapse of the structure may be avoided. On nonredundant structures, the redistribution of load may cause additional members to also fail, resulting in a partial or total collapse of the structure.

There are three basic types of redundancy in bridge design:

- Load path redundancy
- Structural redundancy
- Internal redundancy

Load Path Redundancy

Bridge designs that have three or more main load-carrying members or load paths between supports are considered load path redundant. If one member were to fail, the bridge load would likely be redistributed to the other members, and bridge failure may not occur. An example of load path redundancy is a multi-girder bridge (see Figure 8.1.3).

Some agencies require that a bridge have four or more main load carrying members to be considered load path redundant. Definitive determination of load path redundancy requires structural analysis with members eliminated in turn to determine resulting stresses in the remaining members.

Figure 8.1.3 Load Path Redundant Multi-Girder Bridge

Structural Redundancy

Bridge designs which provide continuity of load path from span to span are referred to as structurally redundant. Continuous span arrangements consisting of three or more spans are considered structurally redundant (see Figure 8.1.4). In the event of an interior member failure, loading from that span can be redistributed to the adjacent spans, and bridge failure may not occur.

Figure 8.1.4 Structurally Redundant Continuous Span Bridge

The degree of structural redundancy can be determined through computer programs which model element failure. Some continuous truss bridges have structural redundancy, but this can only be determined through structural analysis.

Continuous spans are structurally redundant except for the end spans, where the development of a fracture would effectively cause two hinges, one at the abutment and one at the fracture itself. This situation would lead to structural instability.

Internal Redundancy

Internal or member redundancy exists when a bridge member contains three or more elements that are mechanically fastened together so that multiple independent load paths are formed. Failure of one member element might not cause total failure of the member. Examples of internally redundant members are shown in Figures 8.1.5 and 8.1.6.

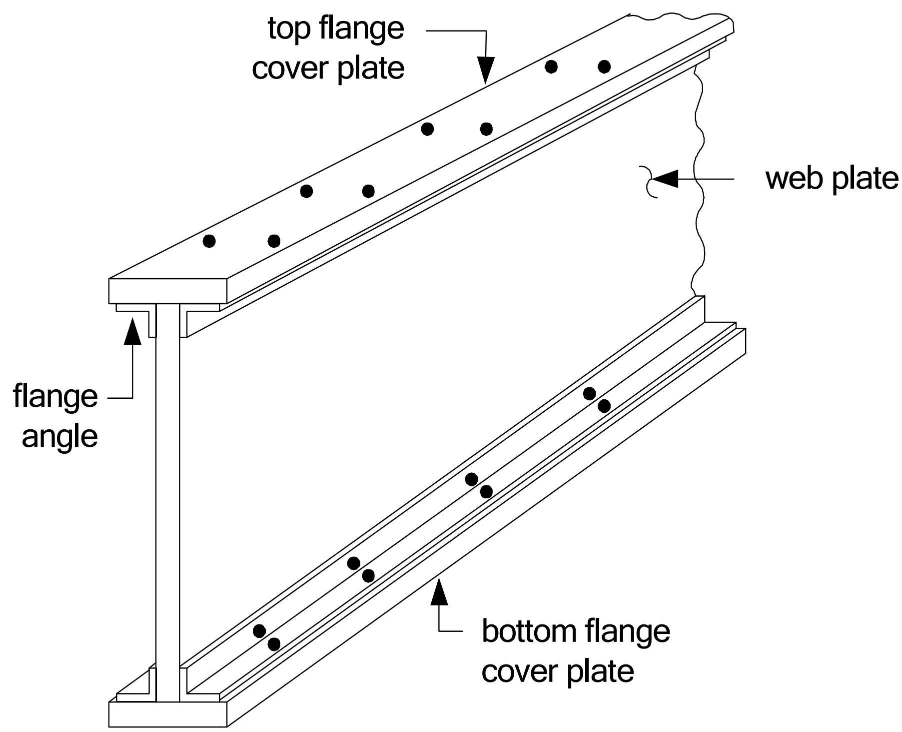


Figure 8.1.5 Internally Redundant Riveted I-Beam

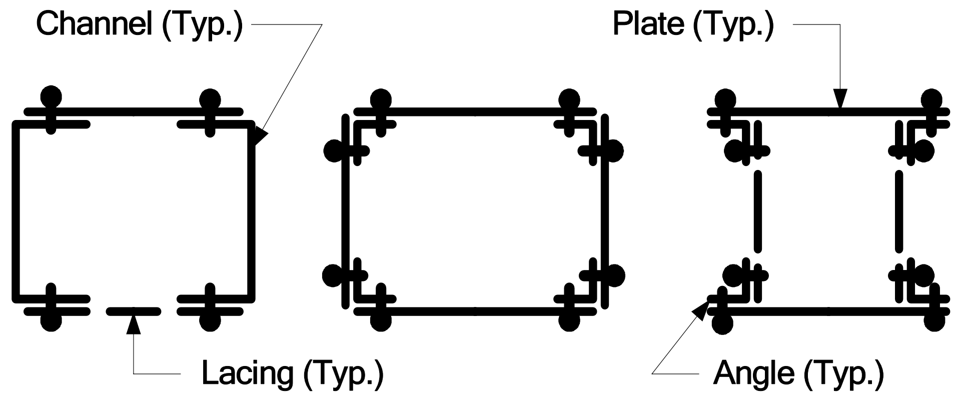


Figure 8.1.6 Internally Redundant Riveted Box Shapes

Internal redundancy of a member can be decreased or eliminated by repairs that involve welding. The welds provide paths for cracks to travel from one element to another (see Figure 8.1.7).

Figure 8.1.7 Patch Plate on Girder Web along Flange Angle

Non-redundant Configuration

Bridge inspectors are concerned primarily with load path redundancy. The inspector should neglect structural and internal redundancy and classify all bridges with less than three load paths as nonredundant (see Figure 8.1.8). Nonredundant bridge configurations in tension contain fracture critical members.

AASHTO Standard Specifications for Highway Bridges, 17th Edition, Section 10.3.1 states that main load carrying components subjected to tensile stresses may be considered nonredundant load path members if failure of a single element could cause collapse.

AASHTO LRFD Bridge Design Specifications, 3rd Edition, with 2005 Interim Revisions, Section 1.2, defines multiple load path structures as structures capable of supporting specified loads following loss of a main load-carrying component or connection. If a structure cannot support the specified loads following loss of a main load carrying member, the consequence is "collapse" as defined in the *AASHTO LRFD Specifications.* Section 1.2 defines collapse as a major change in geometry of the bridge rendering it unfit for use.

AASHTO LRFD Specifications, Section 1.3.4, discusses redundancy. Main elements and components whose failure is expected to cause collapse of the bridge are designated as failure-critical and the associated structural system is considered nonredundant. Failure-critical members in tension may be designated as fracture-critical. Those elements and components whose failure is not expected to cause collapse of the bridge are nonfailure-critical and the associated structural system is considered redundant.

Figure 8.1.8 Nonredundant Two-Girder

8.1.2 Failure Mechanics

Failure mechanics involves describing the process by which a member fails when subjected to fatigue.

The fatigue failure process of a member consists of three stages:

- Crack initiation
- Crack propagation
- Fracture

Crack Initiation

Cracks most commonly initiate from points of stress concentrations in structural details. The most critical conditions for crack initiation at structural details are those combining:

- High stress concentrations due to flaws
- High stress concentrations due to connection details
- High stress concentrations due to out-of-plane distortions

Crack Propagation

Once a fatigue crack has initiated, applied cyclic stresses cause propagation, or growth, of a crack across the section of the member until it reaches a critical size, at which time the member may fracture.

Fracture

Once a crack has initiated and propagated to a critical size, the member fractures. Fracture of a member is the separation of the member into two parts. The fracture of a critical member may cause a total or partial bridge collapse.

Fatigue Life

The number of cycles required to initiate a fatigue crack is the fatigue-crack-initiation life. The number of cycles required to propagate a fatigue crack to a critical size is called the fatigue-crack-propagation life. The total fatigue life is the sum of the initiation and propagation lives.

Bridge engineers use estimations of total fatigue life in predicting the performance of steel bridge members.

Types of Fractures

It is common to classify fractures into two failure modes: brittle fracture and ductile fracture.

- Brittle Fracture - Occurs with no warning and without prior plastic deformation (see Figure 8.1.9).
- Ductile Fracture - Generally proceeded by local plastic deformation of the net uncracked section. This plastic deformation results in distortion of the member, providing some visual warning of the impending failure (see Figure 8.1.10).

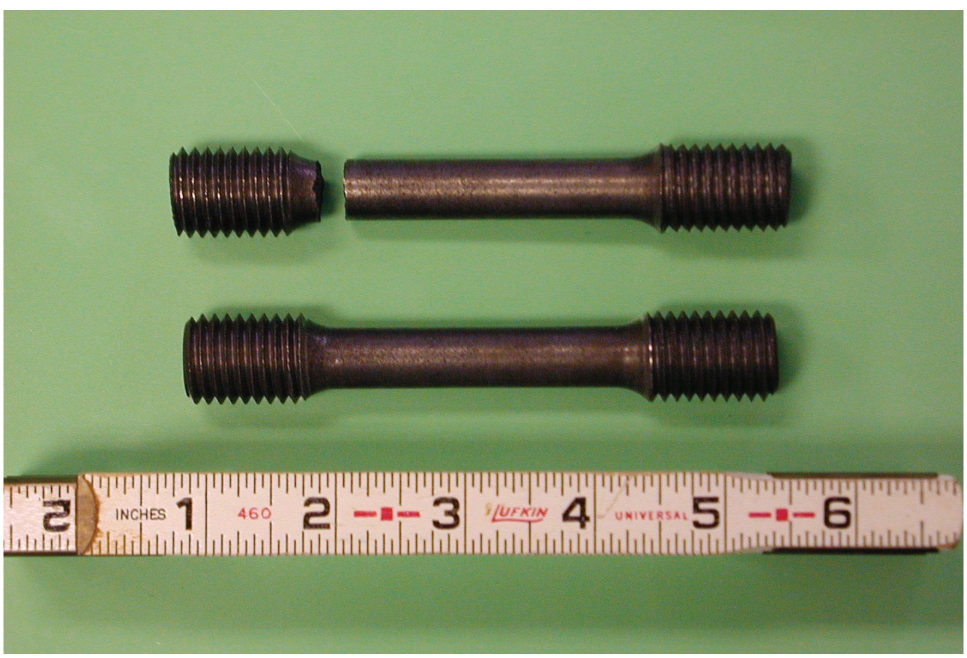


Figure 8.1.9 Brittle Fracture of Cast Iron Specimen

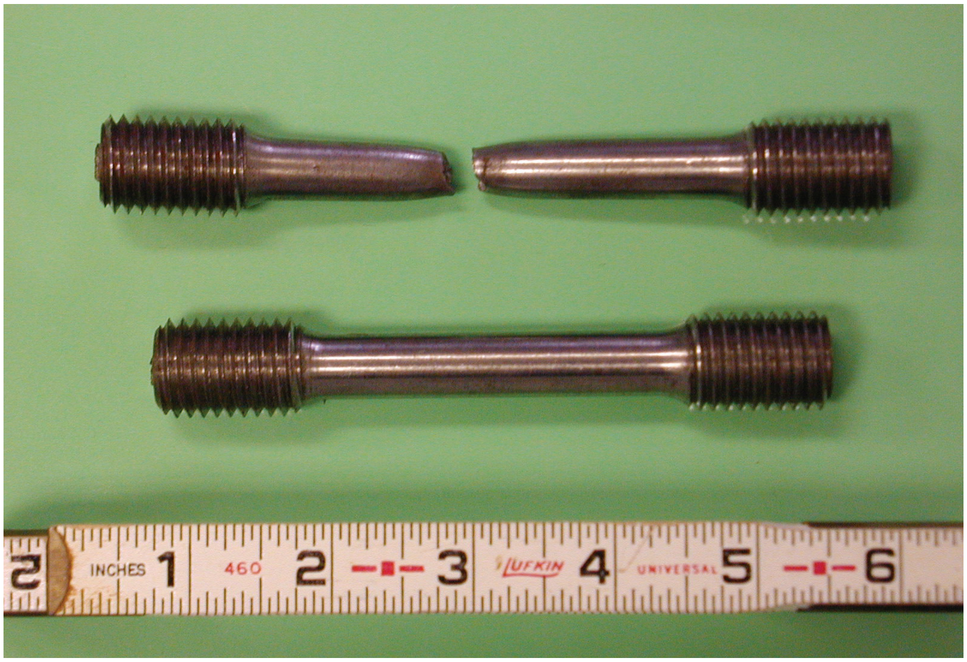


Figure 8.1.10 Ductile Fracture of Cold Rolled Steel

Factors that Determine Fracture Behavior

The transition between a brittle and ductile type of fracture is greatly affected by:

Ambient temperature – Each steel type has a transition temperature below which it becomes brittle.

Loading rate – Truck loading will normally stress the member at an intermediate loading rate which will not create a high energy level. Variations in the speed at which the truck crosses the bridge do not significantly alter the rate of loading. Rapid loading of a steel member, as would occur from a truck collision or an explosion, can create sufficient energy to cause a member to fail in brittle fracture.

Degree of constraint – Thick welded plates or complex joints can produce a high degree of constraint that will limit the steel's ability to deform plastically. Thinner plates are less prone to fracture, given the same conditions, than are thicker plates.

The risk of a brittle fracture in fatigue prone details is greatly increased when the fracture behavior factors include:

- Cold temperature
- Rapid loading
- High constraint

Conversely, some plastic deformation leads to a ductile fracture when the fracture behavior factors are:

- Warm ambient temperature
- Normal truck loading rates
- Low constraint

The transition is a matter of degree. In either case, when it occurs, the fracture of a critical member is sudden and catastrophic.

Fracture Toughness

The fracture toughness is a measure of the material's resistance to crack extension. Fracture toughness can be defined as the ability of a material to resist crack propagation while under load. Fracture toughness is dependent upon the chemical composition of the material. Steel has greater fracture toughness than iron. Fracture toughness generally depends on temperature, environment, loading rate, the composition of the material, together with geometric effects such as constraint. In general, thick welded members made of steel with low toughness are more likely to fracture on cold days.

An impact test that is used to determine the fracture toughness of a steel specimen or coupon is called the Charpy V-notch test (see Figure 8.1.11). This test measures the amount of energy absorbed by a test specimen prior to failure. The Charpy V-notch test requirements vary depending on the type of steel, type of construction, whether welded or mechanically fastened, and the applicable minimum service temperature.

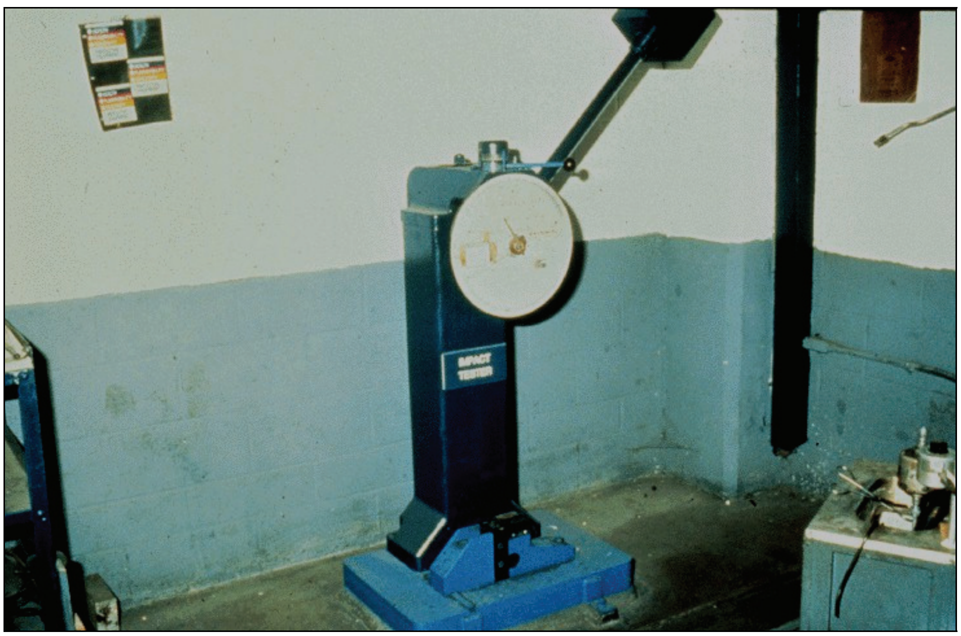


Figure 8.1.11 Charpy V-notch Testing Machine

8.1.3 Factors Affecting Fatigue Crack Initiation

Most critical conditions for fatigue crack initiation are those which involve a combination of flaws and stress concentrations. Girders, stringers, floorbeams, diaphragms, bracing, truss members, hangers, and other members must be structurally connected. Bridge structures, particularly those that are welded, cannot be fabricated without details that cause some level of stress concentrations. Good detailing can reduce the number and severity of these stress concentrations in connections.

Welds

Welds are the connections of metal parts formed by heating the surfaces to a plastic (or fluid) state and allowing the parts to flow together and join with or without the addition of filler metal. The term base metal refers to the metal parts that are to be joined. Filler metal, or weld metal, is the additional molten metal generally used in the formation of welds. The complete assembly is referred to as a weldment. Conditions of stress concentration are often found in weldments and can be prone to crack initiation.

The four common types of welds found on bridges are groove welds, fillet welds, plug welds, and tack welds.

Groove Welds – Groove welds, which are sometimes referred to as butt welds, are used when the members to be connected are lined up edge to edge or are in the same plane (see Figure 8.1.12). Full penetration groove welds extend through the entire thickness of the piece being joined, while partial penetration groove welds do not. Weld reinforcement is the added filler metal that causes the throat dimension to be greater than the thickness of the base metal. This reinforcement is sometimes ground flush with the base metal to qualify the joint for a better fatigue strength category (see Topic 8.1.5 for descriptions of AASHTO Fatigue Categories).

Fillet welds – Fillet welds connect members that either overlap each other or are joined edge to face of plate, as in plate girder assembly of web and flange plates (see Figure 8.1.13). Fillet welds are the most common type of weld because large tolerances in fabrication are allowable when members are lapped over each other instead of fitted together as in groove welds.

Plug welds - Plug and slot welds pass through holes in one member to another, with weld metal filling the holes and joining the members together (see Figure 8.1.14). Plug welds have sometimes been used to fill misplaced holes. These repairs are very likely to contain flaws and microcracks that can result in the initiation of fatigue cracking. Plug welds are no longer permitted by AASHTO for bridge construction.

Tack welds - Tack welds are small welds commonly used to temporarily hold pieces in position during fabrication or construction (see Figure 8.1.15). They are often made carelessly, without proper procedures or preheating, and can be a fatigue-prone detail.

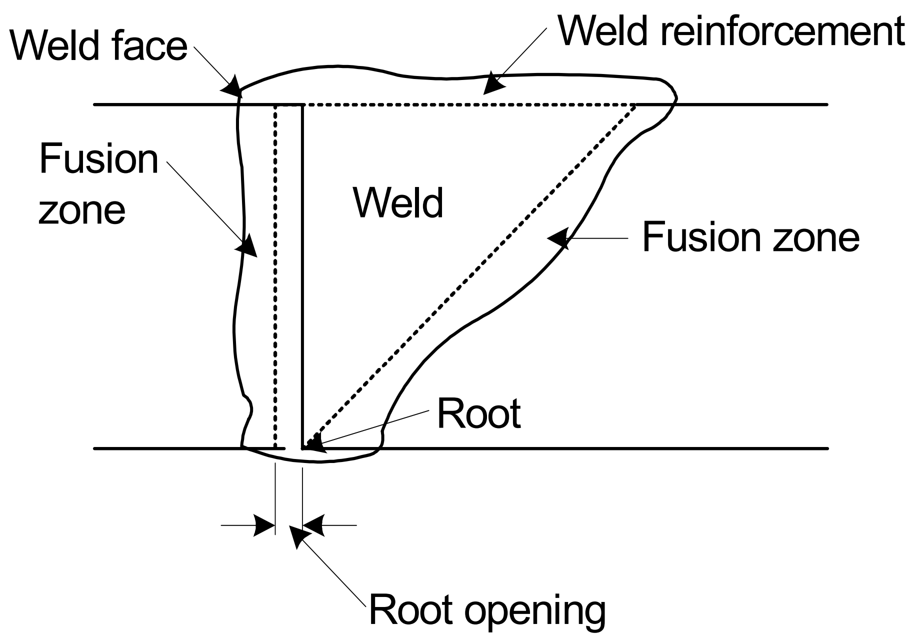


Figure 8.1.12 Groove Weld Nomenclature

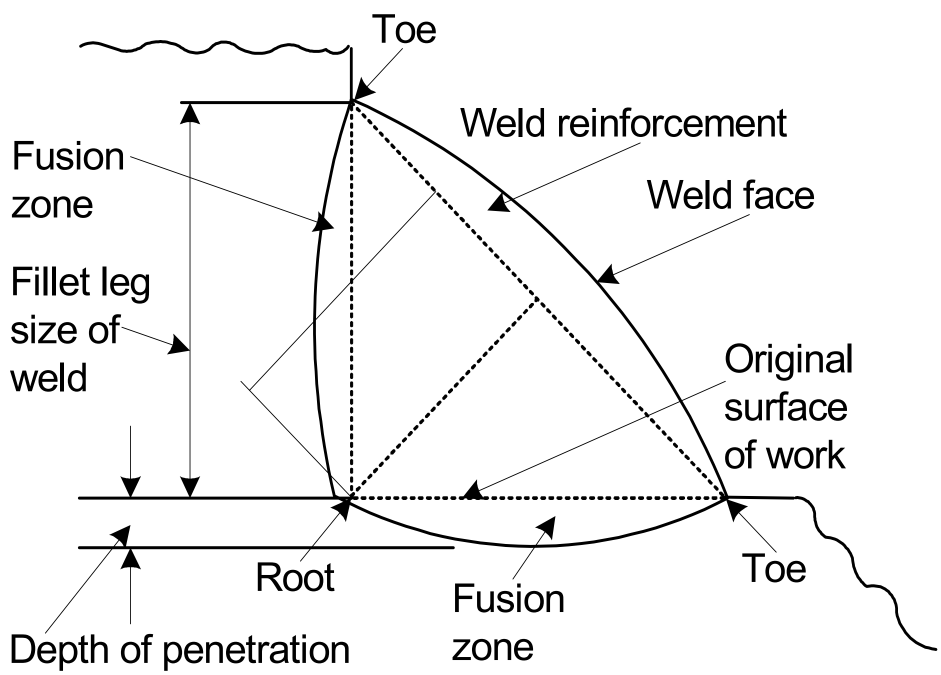


Figure 8.1.13 Fillet Weld Nomenclature

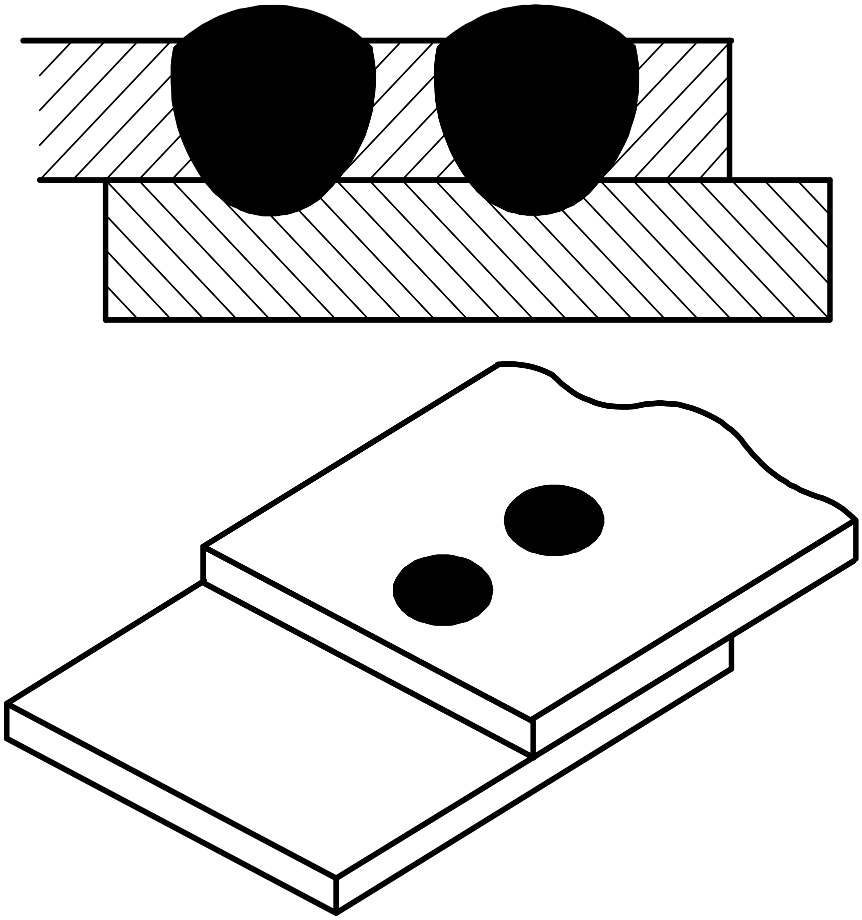


Figure 8.1.14 Plug Weld Schematic

Figure 8.1.15 Tack Weld

Both plug and tack welds are smaller than fillet and groove welds but they can be the source of serious problems to bridges.

The joint geometry is also used to describe the weld. Some common weld joints include (see Figure 8.1.16):

- Butt
- Lap
- Tee
- Edge
- Corner

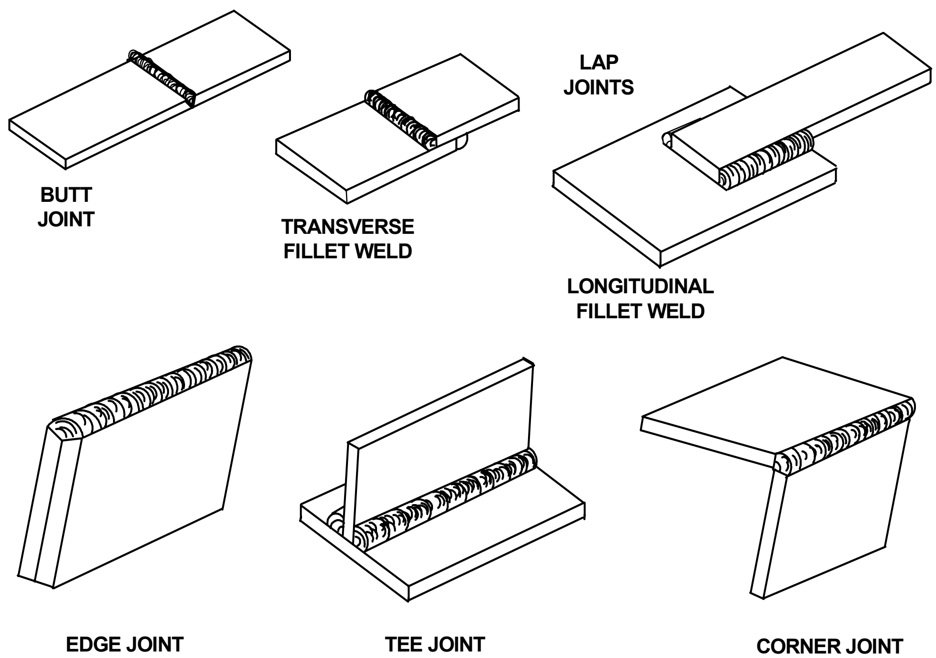


Figure 8.1.16 Types of Welded Joints

All welding processes result in high built-in residual tension stresses, which are at or near the yield point in the weldment and in the base metal adjacent to it. Load-induced stress concentrations also often occur at welded bridge connections, where these residual tensile stresses are high. This combination of stress concentration and high residual tensile stress is conducive to fatigue crack initiation. Such cracks typically begin either at the weld periphery, such as at the toe of a fillet weld, where there typically can be sharp discontinuities, or else at an internal discontinuity such as a slag inclusion or porosity. In the initial stages of fatigue crack growth, much of the fatigue life is expended by the time a crack has propagated out of the high residual tensile stress zone.

Bridge structures, particularly those that are welded, can contain flaws whose size and distribution depend upon the:

- Quality of weld and base material
- Fabrication methods
- Erection techniques

- In-service conditions

Flaws vary in size from very small undetectable nonmetallic inclusions to large inherent weld cracks.

Material Flaws

Material flaws may exist in different forms:

- External flaws (e.g., surface laps)
- Internal flaws (e.g., nonmetallic inclusions, laminations and "rolled-in" plate defects (see Figures 8.1.17 & 8.1.18)).

Figure 8.1.17 Centerline Crack in Steel Slab

The centerline crack in Figure 8.1.17 may have resulted from a shrinkage cavity like that shown in Figure 8.1.18 which was not forged and melded completely in the hot rolling process.

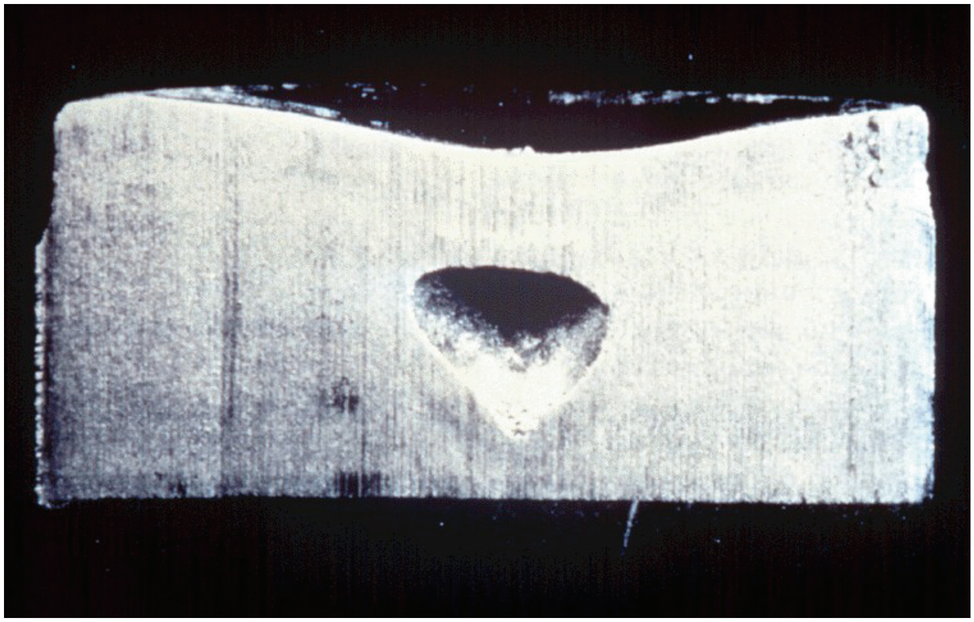


Figure 8.1.18 Shrinkage Cavity in Steel Billet

Fabrication Flaws

Fabrication can introduce a variety of visible and non-visible flaws. Typical non-visible weld flaws include:

Incomplete Penetration – Incomplete penetration occurs when the weld metal fails to penetrate the root of a joint or fails to fuse completely with the root face of the base metal (see Figure 8.1.19). Incomplete penetration is not permitted to any degree. Incomplete penetration welds cause a local stress riser at the root of a weld and can reduce the load carrying capacity of the member. A stress riser is a detail that causes stress concentration.

Lack of fusion – Lack of fusion is a condition in which boundaries of unfused metal exist either between base metal and weld metal or between adjacent layers of weld metal (see Figure 8.1.20). Lack of fusion is generally a result of poor welding techniques, can seriously reduce the load carrying capacity of the member, and could be a point of crack initiation at a lower stress.

Slag inclusions – Slag inclusion occurs when nonmetallic matter is inadvertently trapped between the weld metal and the base metal (see Figure 8.1.21). Slag from the welding rod shield may be forced into the weld metal by the arc during the welding operation. If large, irregular inclusions or lengthy lines of inclusions are present, crack initiation at a lower stress could begin and the strength of the weld may be considerably reduced. However, small isolated globe shaped inclusions do not seriously affect the strength of a weld, but can be a point of crack initiation.

Porosity – Porosity is the presence of cavities in the weld metal caused by entrapped gas and takes the form of small spherical cavities, either scattered throughout the weld or clustered in local regions (see Figure 8.1.22). It is tolerated if the amount does not exceed specified quantities relative to weld size. Sometimes, porosity is visible on the surface of the weld.

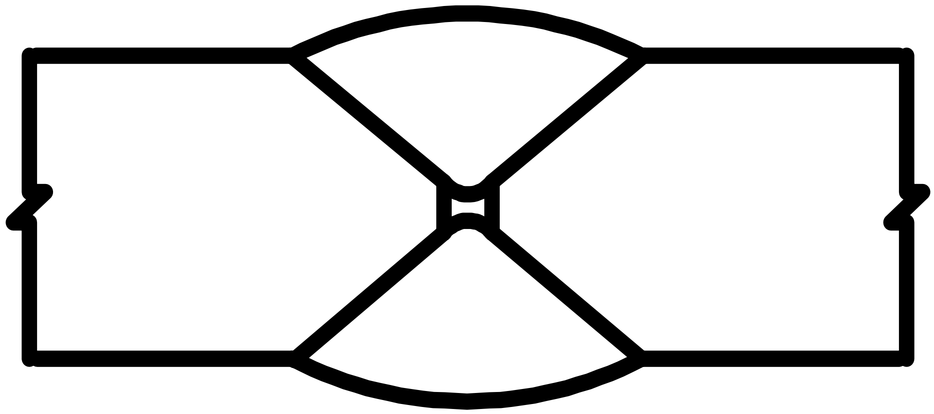


Figure 8.1.19 Incomplete Penetration of a Double V-Groove Weld

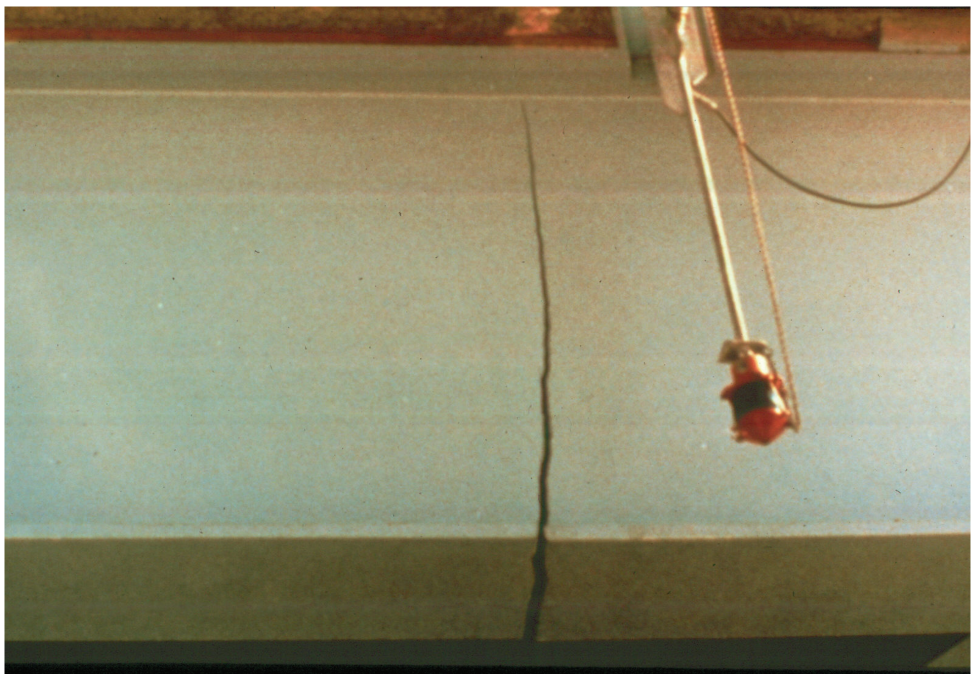


Figure 8.1.20 Crack Initiation from Lack of Fusion in Heat Affected Zone of Electroslag Groove Weld of a Butt Joint

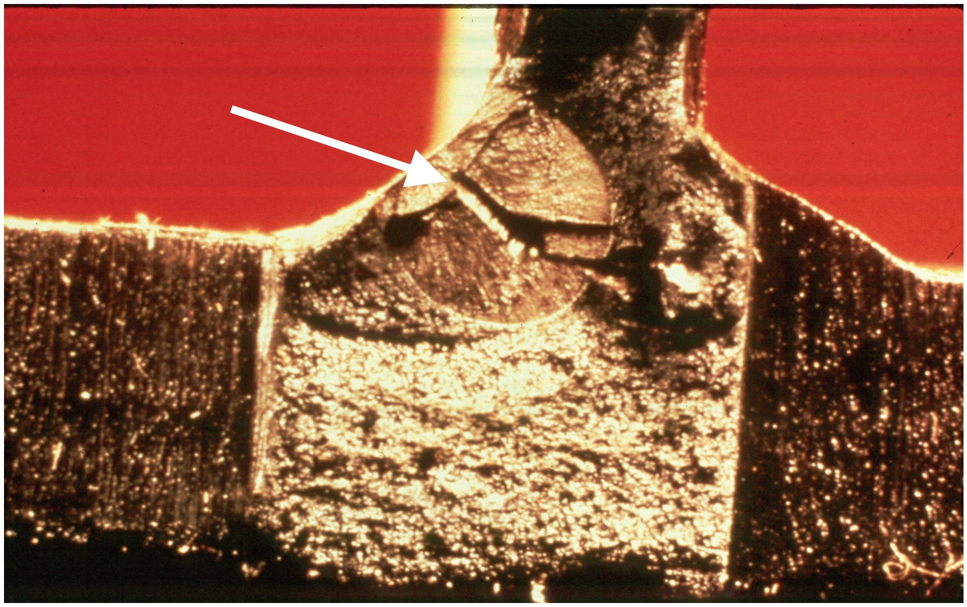


Figure 8.1.21 Web to Flange Crack due to Fillet Weld Slag Inclusion

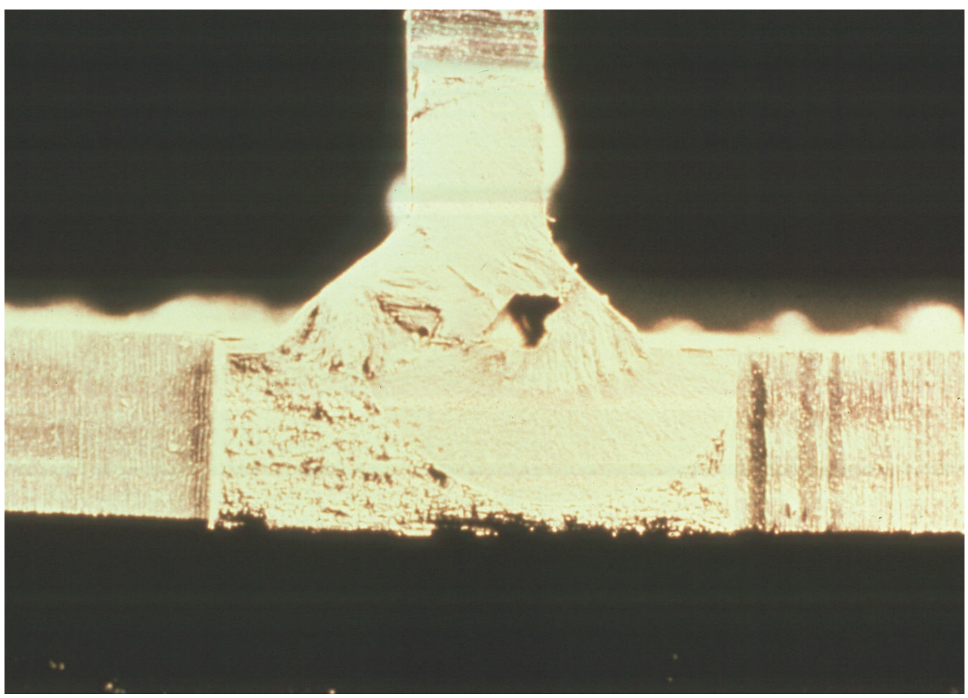


Figure 8.1.22 Crack Initiation from Porosity in Longitudinal Web-to-Flange Fillet Weld of Plate Girder

Plug welds are sometimes found in bridge members. In most cases, they were made to fill mislocated bolt holes. Such welds are highly restrained and often will contain incomplete penetration, lack of fusion, slag inclusions, and porosity. There have been many instances where a crack and fracture have occurred because of a plug weld (see Figure 8.1.23).

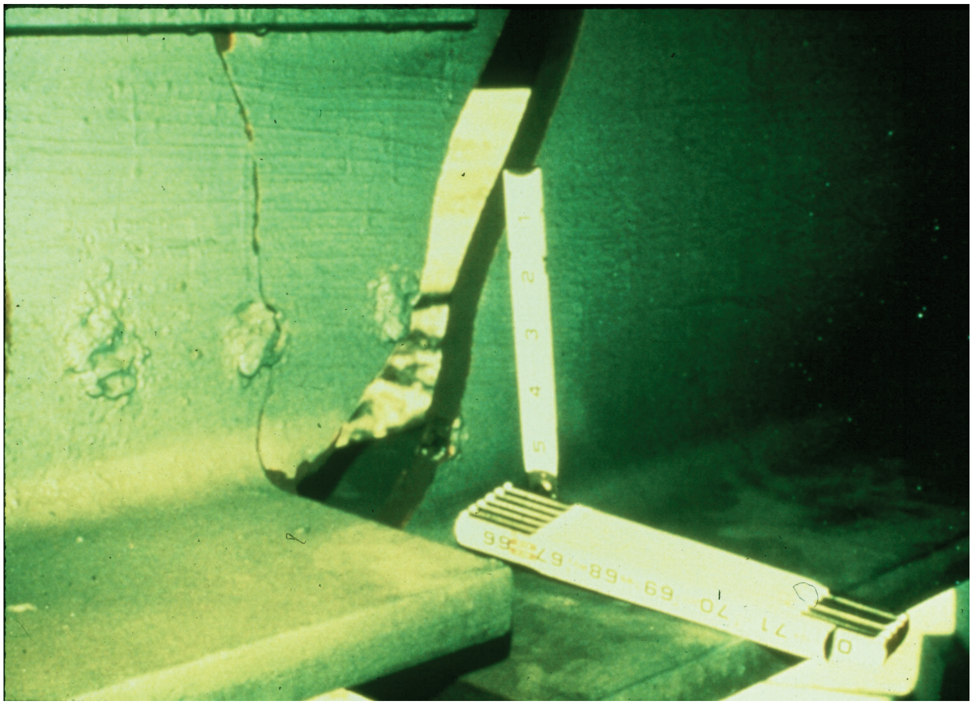


Figure 8.1.23 Crack Resulting from Plug Welded Holes

Visible weld flaws include:

Undercut – The condition in which a local reduction in a section of base metal occurs alongside the weld deposit. This may happen either on the surface of the base metal at the toe of the weld, or in the fusion face of multiple pass welds due to overheating. This groove creates a mechanical notch, which is a stress riser (see Figure 8.1.24). When an undercut is controlled within the limits of specifications and does not constitute a sharp or deep notch, it is not seen as a serious defect.

Overlap – Overlap is a weld flaw at the toe of a weld in which the weld metal overflows onto the surface of the base metal without fusing to it due to insufficient heat (see Figure 8.1.25). This condition may exist intermittently or continuously along the weld joint. Discontinuity at the toe of a weld acts as a stress riser and reduces the fatigue strength of the member.

Bolt and Rivet Holes – Holes of any kind in the base metal create a stress riser. Punched holes for rivets, without reaming, contain gouges that can initiate a crack. Burrs generated during the drilling process are additional risers and should be removed. However, smooth, round holes are not significant stress risers.

Beam Coping – When flange/web copings do not have the proper radius as per AASHTO specifications a stress riser is created (see Figure 8.1.26).

Flame Cuts – Flame cutting, although fast, creates large surface discontinuities that are stress risers (see Figure 8.1.27). The surface of flame cut plates in tension should be ground smooth in the direction of the tensile stress.

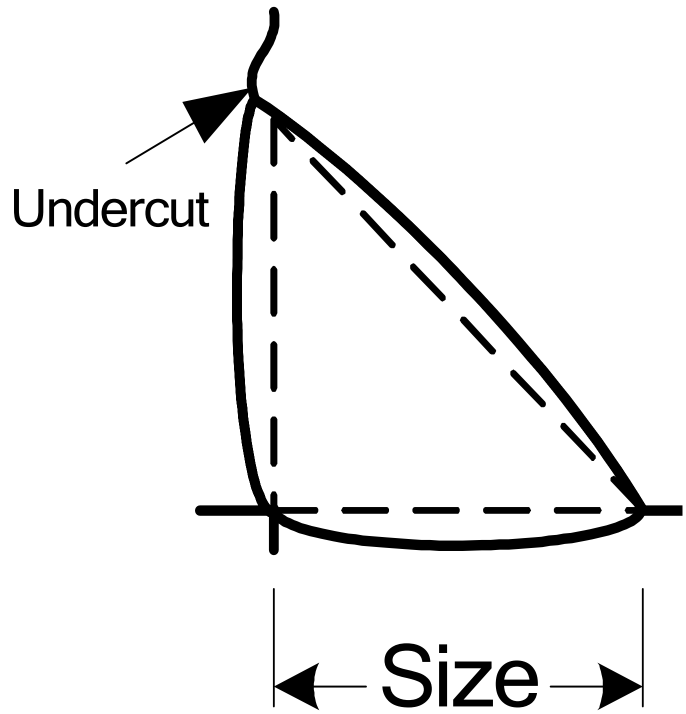


Figure 8.1.24 Undercut of a Fillet Weld

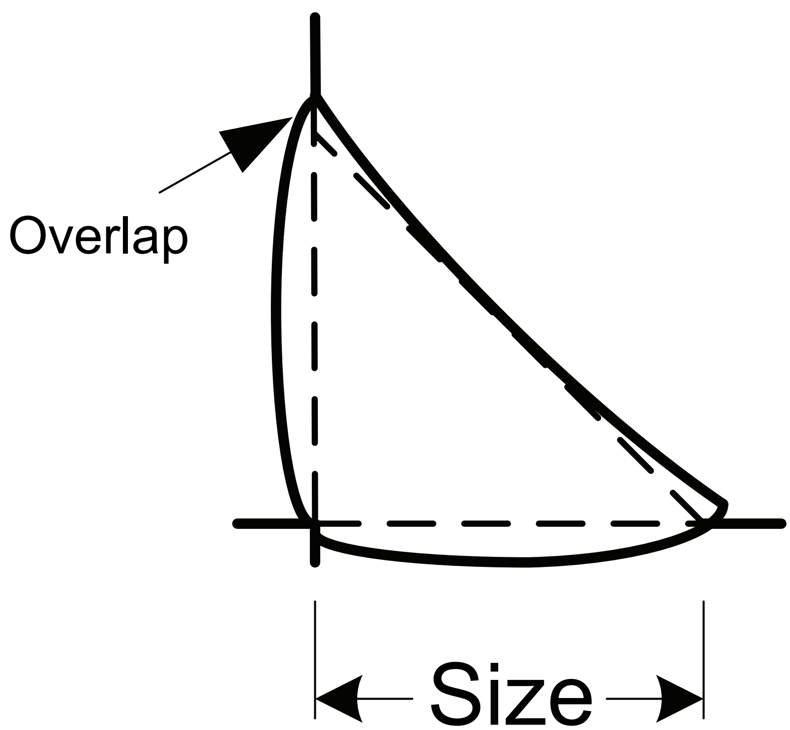


Figure 8.1.25 Overlap of a Fillet Weld

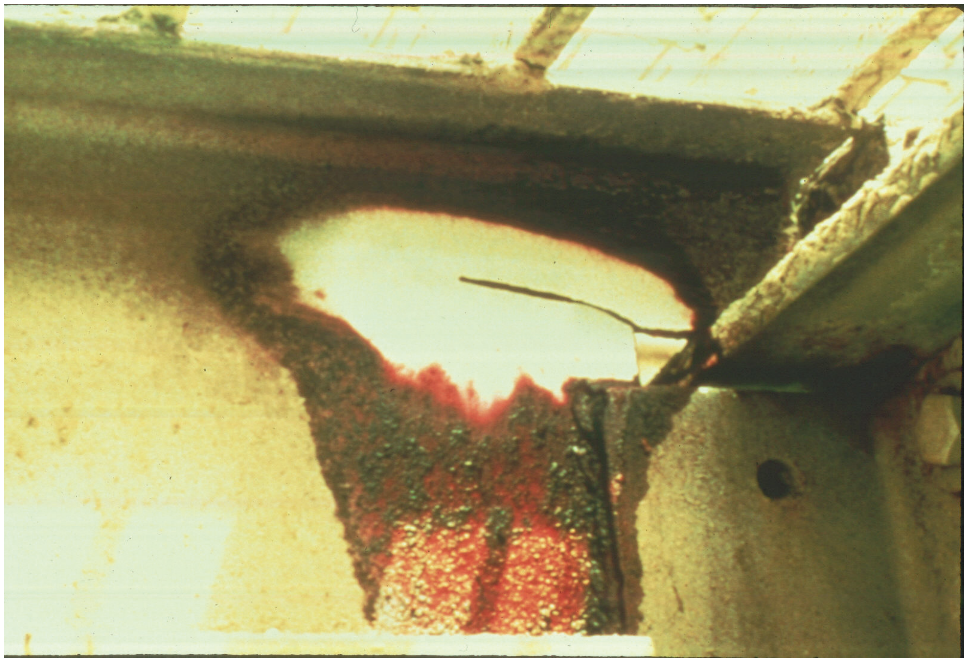


Figure 8.1.26 Crack Initiation at Coped Web in Stringer to Floorbeam

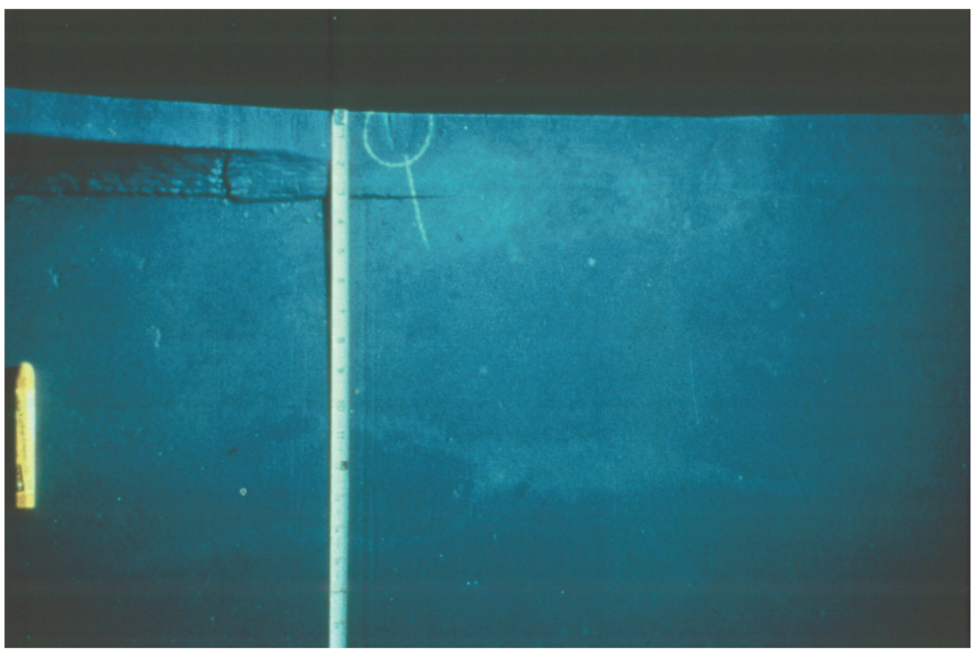


Figure 8.1.27 Insufficiently Ground Flame Cut of Gusset Plate for Arch to Tie Girder Connection

Transportation and Erection Flaws

Careless handling during transportation and erection may leave the following flaws along the edges of members:

Nicks, Notches, and Indentations – Beam handling devices such as lifting tongs develop intense pressure at the point of contact and can cause measurable indentations and gouges.

Chain marks – When transporting steel beams, chains are commonly used to secure the beam to the truck or railroad car.

Out-of-Plane Bending Forces – Beams must be securely blocked to resist cyclic sidesway movement during truck or rail transport. There have been extreme cases where cracks have initiated in beams before they have been erected.

Tack Welds – Tack welds were very common in the mid 1950's and through the early 1960's and were used to hold members together during erection. When left in place and exposed to tensile stress, they are a potential crack initiation location.

In-Service Flaws

Once the structure is placed in service, some members may be prone to collision damage by errant vehicles which may nick, tear, and excessively stress the steel (see Figure 8.1.28). Improper heat straightening may damage the steel. Deep corrosion pits can develop in structures that are improperly detailed for corrosion control, poorly maintained, or left unpainted. Also, any indiscriminate welding of conduit supports, lighting attachments, and ladder brackets to steel members can cause stress risers in the base metal.

Figure 8.1.28 Severe Collision Damage on a Fascia Girder

In summary, bridges can contain significant flaws that can be the point of initiation of fatigue cracking and possibly result in fracture. Their presence must be presumed, and inspection must be diligent to identify them in order to permit timely corrective measures whenever appropriate.

8.1.4 Factors Affecting Fatigue Crack Propagation

Failures due to cracking develop as a result of cyclic loading and usually provide little evidence of plastic deformation. Hence, they are often difficult to see before serious distress develops in the member. Fatigue cracks generally require large magnitudes of cyclic stresses, corresponding to a high frequency of occurrence or to a long exposure time. Structural details have various amounts of resistance to fatigue cracks caused by these large magnitudes of cyclic stresses. The three major parameters affecting fatigue crack propagation life are:

- Stress range
- Number of cycles
- Type of details

Stress Range

The stress range is defined as the algebraic difference between the maximum stress and the minimum stress calculated at the detail under consideration. In other words, it is the value of the cyclic stress caused by a truck crossing the bridge (see Figure 8.1.29). The weight of the bridge produces a constant stress instead of a cyclic stress. Therefore, it does not affect the crack propagation life. Only stress ranges in tension or stress reversal can drive fatigue cracks to failure. Stress ranges in compression may cause cracks to grow to some extent at weldments where there are high residual tensile stresses. However, these "compression" cracks eventually arrest, and they do not induce fracture of the member.

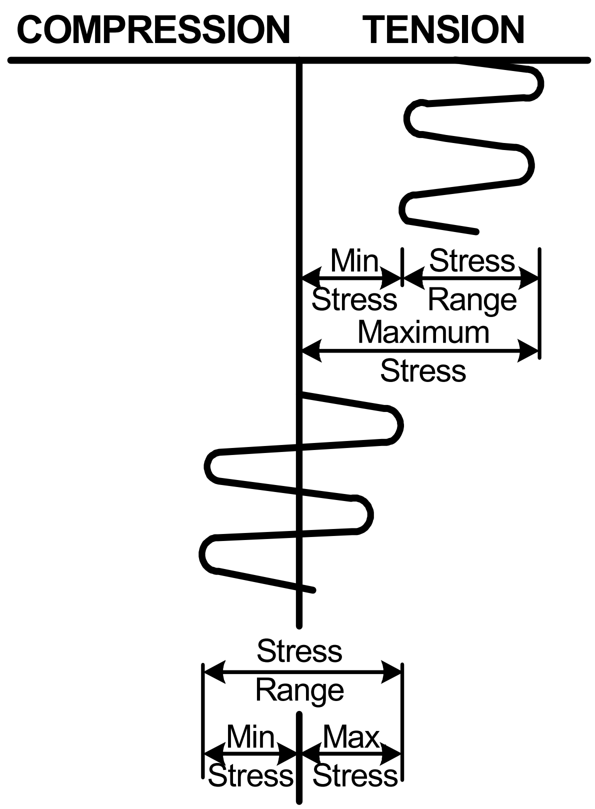


Figure 8.1.29 Applied Tensile and Compressive Stress Cycles

Number of Cycles

The number of stress cycles is proportional to the number of trucks that cross the bridge during its service life. The number of cycles per truck passage depends upon span arrangement, member orientation, structure type, and location relative to interior supports. In some bridges, stress cycles are induced by wind loading.

Types of Details

There are many details used in the connections of bridges. AASHTO has chosen some typical details, or Illustrative Examples (see Figure 8.1.30). These Illustrative Examples are used to help determine AASHTO Fatigue Categories (see Topic 8.1.5).

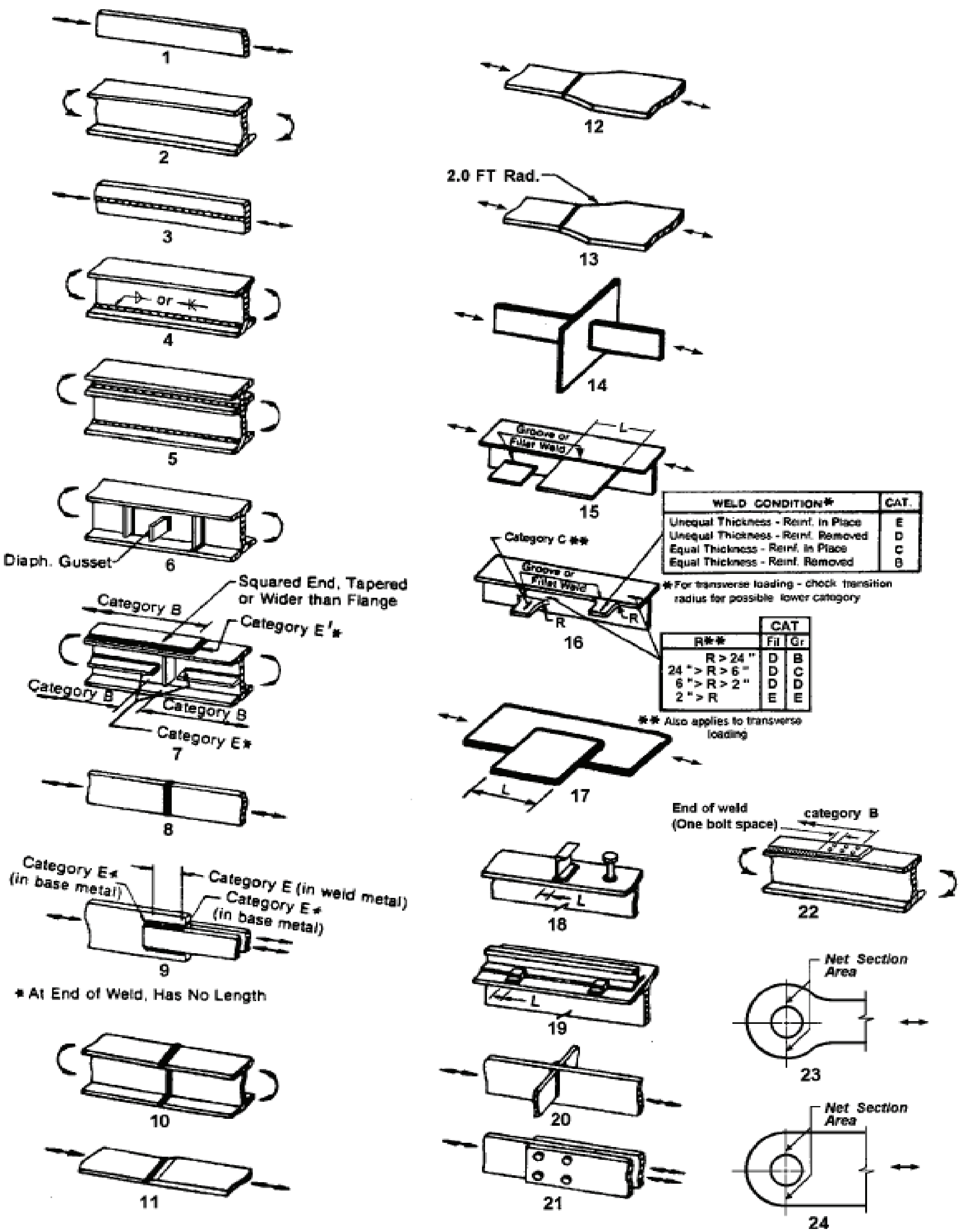


Figure 8.1.30 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Figure 6.6.1.2.3-1 - Illustrative Examples of Bridge Details

Various details will have different fatigue strengths associated with them. It is common practice among bridge engineers to group steel bridge structural details into a number of categories. By doing this, the bridge engineer can design against risk levels of fatigue failure of the various details (i.e., details of higher fatigue strength categories are allowed higher stress ranges than the lower category details).

Flange Crack Failure Process

A common location for initiation of a flange crack is at the end of a partial length cover plate welded longitudinally along its sides and transversely across the ends as it is attached to the tension flange of a rolled beam.

One or more cracks can initiate from microscopic flaws or defects at the weld toe of the transverse end weld (see Figure 8.1.31). Such cracking may then advance in three stages:

Figure 8.1.31 Part-Through Crack at a Cover Plated Flange

Stage 1

In the first stage, a part-through surface crack is only barely visible as a hairline on the bottom of the flange at the toe of weld. As stress is applied, the small cracks that have initiated join each other and begin to form a larger part-through surface crack (see Figure 8.1.32).

Figure 8.1.32 Part-Through Crack Growth at Cover Plate Welded to Flange

The crack front develops a thumbnail or half penny shape as it propagates in the thickness direction of the flange until reaching the inside surface. Once it breaks through the thickness of the flange, the shape rapidly changes into that of a three-ended crack.

Crack propagation begins at a very slow rate and gradually accelerates as the crack grows in size. Approximately 95% of the fatigue life is spent growing the Stage 1 part-through crack.

Stage 2

During the second stage, the crack then propagates with two fronts moving across the flange width and one front moving into the web until it reaches a critical size, at which time the member may fracture (see Figure 8.1.33).

The crack is readily visible as a through-the-thickness crack on both the top and bottom surfaces of the flange (see Figure 8.1.34).

Approximately 5% of the fatigue life is left for growing the Stage 2 through crack (see Figure 8.1.35).

Stage 3

When a crack propagates to a critical size, the member will fracture. Fracture is the separation of the member into two parts. When the member is fracture critical, the span, or a portion of it, would likely collapse.

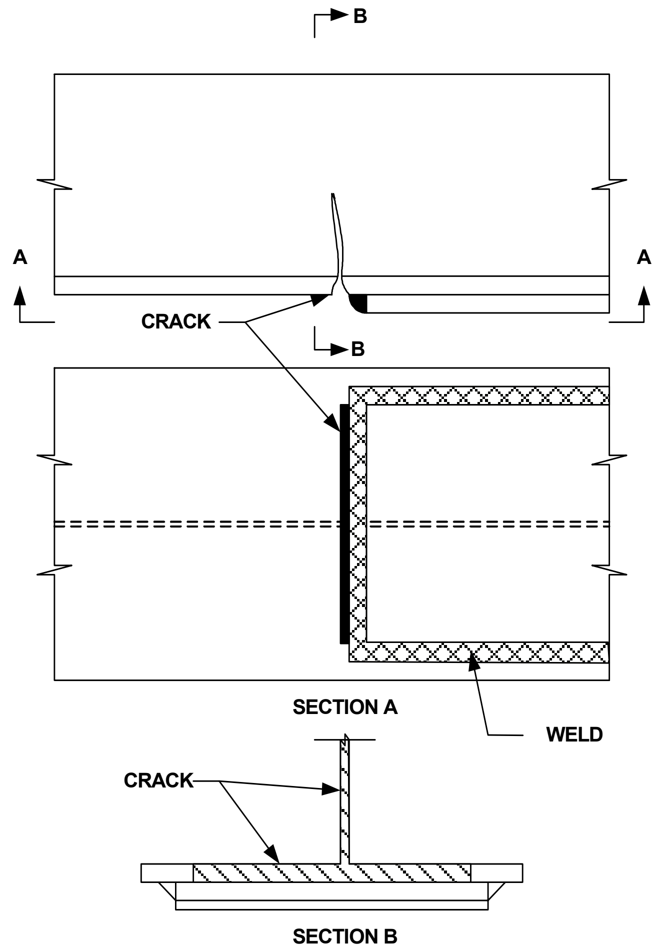


Figure 8.1.33 Through Crack Growth at Cover Plate Welded to Flange

Figure 8.1.34 Through Crack at a Cover Plated Flange

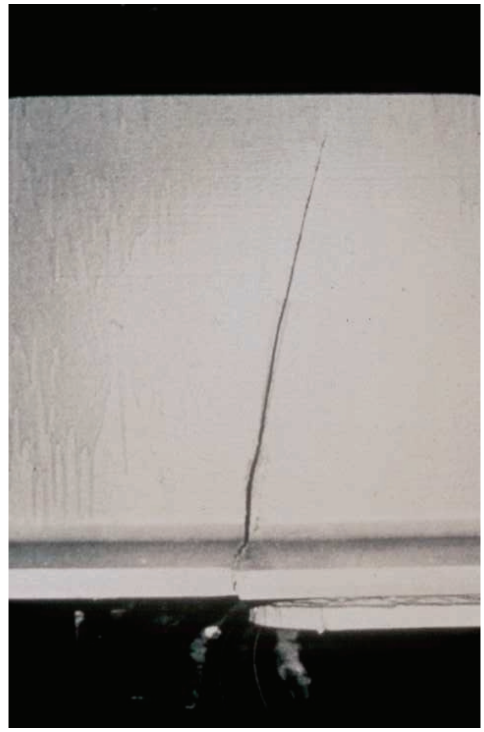


Figure 8.1.35 Through Crack has Propagated into the Web

Inspection

It is important the inspector realizes that cracks are only readily detectable visually as a through crack after most of the fatigue life of the detail is gone. Therefore, a structural engineer must be notified immediately whenever cracks are found in a flange.

When the fatigue life is finally used up, that is the fatigue crack has grown to a critical size and stress intensity, then fracture will occur. The brittle fracture surface will appear crystalline or uneven, and will often reveal a herringbone pattern oriented toward the point of fracture initiation (see Figure 8.1.36).

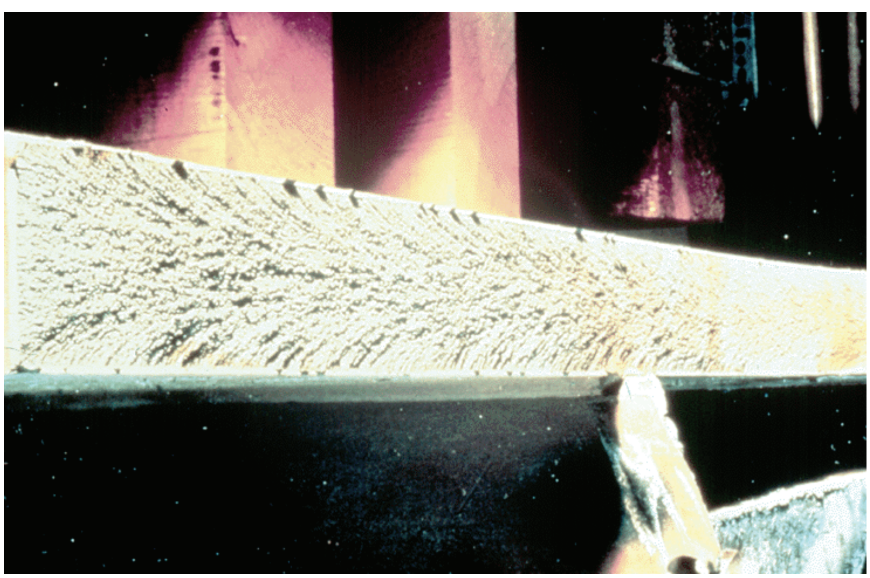


Figure 8.1.36 Brittle Fracture – Herringbone Pattern

Web Crack Failure Process

A common location for initiation of a web crack is at the weld toe of a transverse stiffener that is welded to the web of a beam. This type of crack grows in three stages (see Figure 8.1.37):

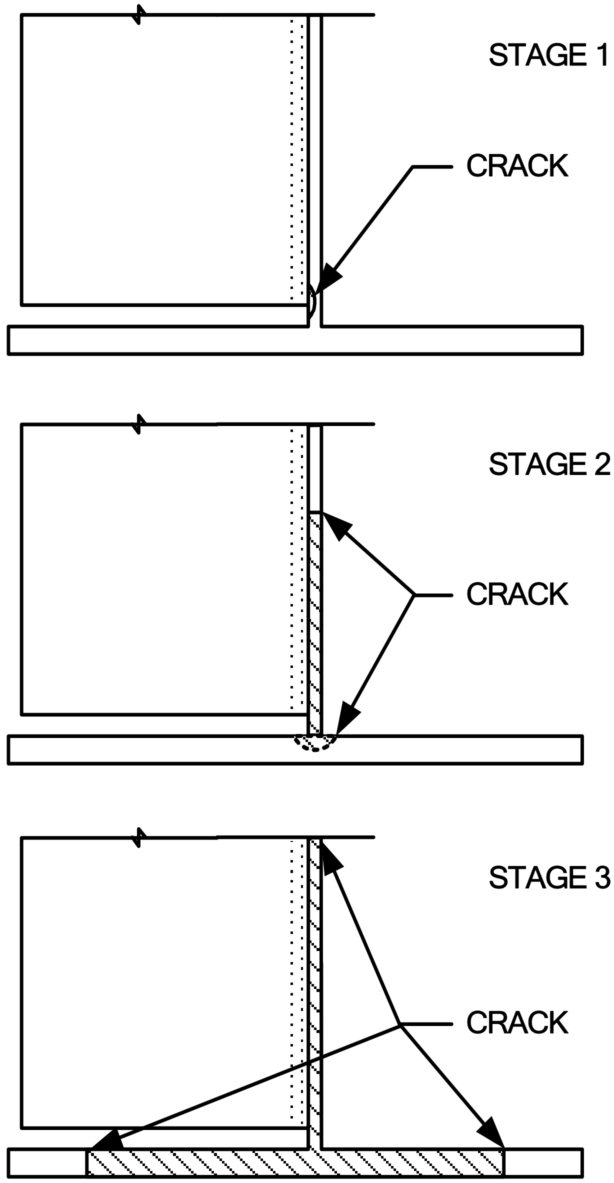


Figure 8.1.37 Crack Growth at Transverse Stiffener Welded to Web

Stage 1

A fatigue crack initiates at the weld toe near the end of the stiffener and propagates during the first stage as a part-through crack in the thickness direction of the web until it reaches the opposite face of the web.

A part-through stiffener crack is often just barely visible as a hairline along the toe of the weld (see Figure 8.1.38).

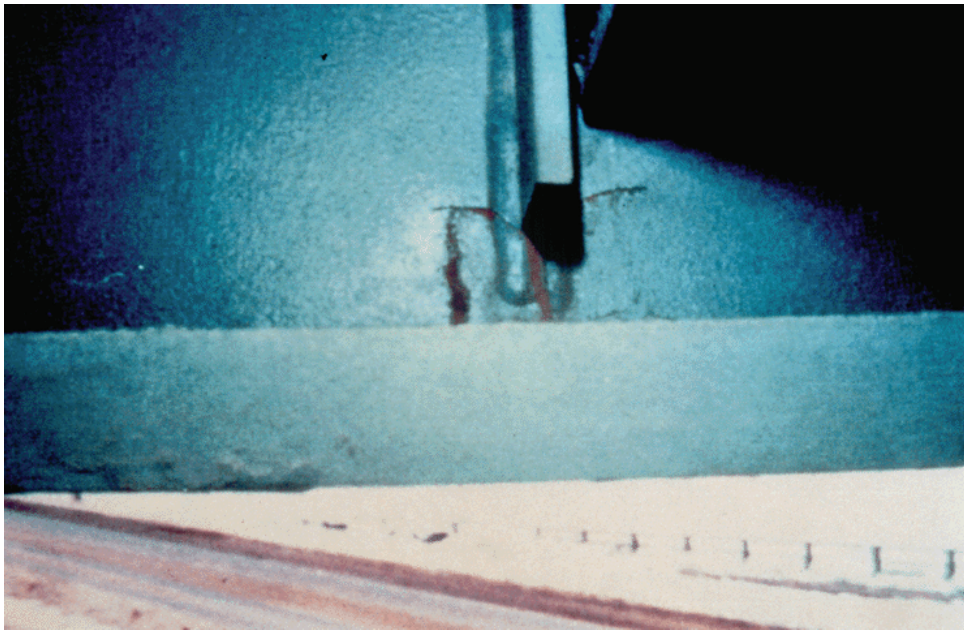


Figure 8.1.38 Part-Through Web Crack

This type of stiffener crack expends about 95% of its fatigue life propagating in Stage 1.

Stage 2

After it breaks through the web, the shape changes into a two-ended (or more) through crack which propagates up and down the web (see Figure 8.1.39). The through crack can be readily seen on both sides of the web. The stiffener crack expends about 5% of the fatigue life propagating in Stage 2.

Figure 8.1.39 Through Crack in the Web

Stage 3

Eventually, the lower crack front will reach the bottom flange, and the three-ended crack will then propagate with two fronts moving across the flange and one front moving farther up the web, until the member fractures (see Figure 8.1.40).

Figure 8.1.40 Through-Crack Ready to Propagate into the Flange

The through crack can usually readily be seen on both sides of the web and on both sides of the flange.

Any web cracks discovered should be brought to the immediate attention of a bridge engineer.

8.1.5 AASHTO Detail Categories for Load-Induced Fatigue

For purposes of designing bridges for fatigue caused by in-plane bending stress, the details are grouped into categories labeled A to E' (see Figure 8.1.41). The classification of details by category does not apply to details that crack due to out-of-plane distortion. Each letter represents a rating given to a detail that indicates its level of fatigue strength. The details assigned to the same category have about equally severe stress concentrations and comparable fatigue lives. The alphabetical classification by the severity of the stress concentration is a useful method of identifying fatigue strength.

When used in inspection, these fatigue categories serve as a reminder of which details are prone to fatigue cracking. The categories are defined as follows.

Category A

Includes “base metal” or plain material with rolled or cleaned surfaces, away from welded, riveted or bolted connections.

This condition has the best fatigue resistance.

It is not common practice to examine these base metal regions for fatigue cracks unless the regions are susceptible to distortion, because cracks usually develop at nearby connection details with lower fatigue strength categories.

Category B

Includes the following welded structural details and high strength bolted joints:

- Longitudinal continuous welds in built-up plates and shapes.
- Transverse full penetration groove welds with weld reinforcement ground smooth and weld soundness established by non-destructive testing (NDT).
- Groove welded attachments with a transition radius not less than 610 mm (24 inches).
- High strength bolted connections.

Category B'

Sub-category including details similar to those of Category B, but found to be more sensitive to fatigue:

- Longitudinal continuous welds in built-up plates and shapes not detailed in Category B.
- Transverse full penetration groove welds with reinforcement ground smooth to provide straight transition in width or thickness, slopes of transition not steeper than 1 to 2.5, and base metal A514 or A517.

Category C and C'

Includes transverse stiffeners, very short attachments, and transverse groove welds with reinforcement not removed.

- Base metal at welds connecting transverse stiffeners or vertical gusset plates to connection and gusset plates of girder webs or flanges.
- Transverse full penetration groove welds, weld reinforcement not removed, but with weld soundness established by NDT.
- Groove or fillet welded horizontal gusset or attachment, the length of which (in the direction of the main member) is less than 50 mm (2 inches).
- Groove welded attachments 150 to 610 mm (6 to 24 inches) in length with transition radius.
- Intersecting plates connected by fillet welds with the discontinuous plate not more than13 mm (½ inch) thick.
- Shear connectors.

Category D

Includes welded short attachments, welded connections with sharp transition curves, and riveted joints.

- Welded attachments with a short groove or fillet weld in the direction of the main member between 50 and 100 mm (2 and 4 inches) long but less than 12 times the plate thickness.

- Groove welded attachments with transition radius between 50 and 150 mm (2 and 6 inches).
- Groove welded attachments with unequal plate thickness, weld perpendicular to attachment, weld reinforcement removed, and a transition radius of at least 50 mm (2 inches).
- Longitudinally loaded fillet weld with length in the direction of stress of 50 to 100 mm (2 to 4 inches).
- Riveted connections, net section.

Category E and E'

Includes details that have the lowest fatigue strength in comparison to those in other categories. Generally, for welded details in this group with the same configurations, Category E' applies if the flange plate thickness exceeds 20 mm (0.8 inch) or if the attachment plate thickness is 25 mm (1 inch) or more.

- Ends of partial length cover plates on girder or beam flanges.
- Welded attachment, with groove or fillet weld in the direction of the main members, more than 100 mm (4 inches) or 12 times the plate thickness.
- Welded attachment with curved transition radius.
- Welded attachment with loads transverse to welds.
- Intermittent fillet welds
- Shear stress on the throat of a fillet weld (Formerly classified Category F)
- Deck plate at the connection to floorbeam web.

Of all the details, those in Categories E and E’ are the most susceptible to fatigue crack growth. These details should be closely examined at every inspection.

GENERAL CONDITION	SITUATION	DETAIL CATEGORY	ILLUSTRATIVE EXAMPLE, SEE FIGURE 6.6.1.2.3-1
Plain Members	Base metal:		1, 2
	• With rolled or cleaned surfaces; flame-cut edges with ANSI/AASHTO/AWS D1.5 (Section 3.2.2) smoothness of 1,000 µ - in. or less	A	
	• Of unpainted weathering steel, all grades, designed and detailed in accordance with FHWA (1989)	B	
	• At net section of eyebar heads and pin plates	E	
Builtup Members	Base metal and weld metal in components, without attachments, connected by:		3, 4, 5, 7
	• Continuous full-penetration groove welds with backing bars removed, or	B	
	• Continuous fillet welds parallel to the direction of applied stress	B	
	• Continuous full-penetration groove welds with backing bars in place, or	B'	
	• Continuous partial-penetration groove welds parallel to the direction of applied stress	B'	
	Base metal at ends of partial-length cover plates:		
	• With bolted slip-critical end connections	B	22
	• Narrower than the flange, with or without end welds, or wider than the flange with end welds		7
	• Flange thickness ≤ 0.8- in.	E	
	• Flange thickness > 0.8- in.	E'	
	• Wider than the flange without end welds	E'	

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Table 6.6.1.2.3-1 - Fatigue Categories of Bridge Details

GENERAL CONDITION	SITUATION	DETAIL CATEGORY	ILLUSTRATIVE EXAMPLE, SEE FIGURE 6.6.1.2.3-1
Groove-Welded Splice Connections with Weld Soundness Established by NDT and All Required Grinding in the Direction of the Applied Stresses	Base metal and weld metal at full-penetration groove-welded splices:		
	• Of plates of similar cross-sections with welds ground flush	B	8, 10
	• With 2.0 ft. radius transitions in width with welds ground flush	B	13
	• With transitions in width or thickness with welds ground to provide slopes no steeper than 1.0 to 2.5		11, 12
	◦ Grades 100/100W base metal	B'	
	◦ Other base metal grades	B	
	• With or without transitions having slopes no greater than 1.0 to 2.5 when weld reinforcement is not removed	C	8, 10, 11, 12
Longitudinally Loaded Groove-Welded Attachments	Base metal at details attached by full- or partial-penetration groove welds:		
	• When the detail length in the direction of applied stress is:		
	◦ Less than 2.0 in.	C	6, 15
	◦ Between 2.0 in. and 12 times the detail thickness, but less than 4.0 in.	D	15
	◦ Greater than either 12 times the detail thickness or 4.0 in.		
	▪ Detail thickness < 1.0- in.	E	15
	▪ Detail thickness ≥ 1.0- in.	E'	15
	• With a transition radius with the end welds ground smooth, regardless of detail length:		16
	◦ Transition radius ≥ 24.0 in.	B	
	◦ 24.0 in. > transition radius ≥ 6.0 in.	C	
	◦ 6.0 in. > transition radius ≥ 2.0 in.	D	
	◦ Transition radius < 2.0 in.	E	
	• With a transition radius with end welds not ground smooth	E	16

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Table 6.6.1.2.3-1 - Fatigue Categories of Bridge Details, continued

GENERAL CONDITION	SITUATION	DETAIL CATEGORY	ILLUSTRATIVE EXAMPLE, SEE FIGURE 6.6.1.2.3-1
Transversely Loaded Groove-Welded Attachments with Weld Soundness Established by NDT and All Required grinding Transverse to the Direction of Stress	Base metal at detail attached by full-penetration groove welds with a transition radius:		16
	• With equal plate thickness and weld reinforcement removed:		
	• Transition radius ≥ 24.0 in.	B	
	• 24.0 in. > transition radius ≥ 6.0 in.	C	
	• 6.0 in. > transition radius ≥ 2.0 in.	D	
	• Transition radius < 2.0 in.	E	
	• With equal plate thickness and weld reinforcement not removed:		
	• Transition radius ≥ 6.0 in.	C	
	• 6.0 in. > transition radius ≥ 2.0 in.	D	
	• Transition radius < 2.0 in.	E	
	• With unequal plate thickness and weld reinforcement removed:		
	• Transition radius ≥ 2.0 in.	D	
	• Transition radius < 2.0 in.	E	
	• For any transition radius with unequal plate thickness and weld reinforcement not removed	E	
Fillet-Welded Connections with Welds Normal to the Direction of Stress	Base metal: • At details other than transverse stiffener-to-flange or transverse stiffener-to-web connections	Lesser of C or Equation 6.6.1.2.5-3	14
	• At the toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	C'	6
Fillet-Welded Connections with Welds Normal and/or parallel to the Direction of Stress	Shear stress on the weld throat	E	9

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, 6.6.1.2.3-1 - Fatigue Categories of Bridge Details, continued

GENERAL CONDITION	SITUATION	DETAIL CATEGORY	ILLUSTRATIVE EXAMPLE, SEE FIGURE 6.6.1.2.3-1
Longitudinally Loaded Fillet-Welded Attachments	Base metal at details attached by fillet welds:		
	• When the detail length in the direction of applied stress is:		
	• Less than 2.0 in. or stud-type shear connectors	C	15, 17, 18, 20
	• Between 2.0 in. and 12 times the detail thickness, but less than 4.0 in.	D	15, 17
	• Greater than either 12 times the detail thickness or 4.0 in.		7, 9, 15, 17
	• Detail thickness < 1.0- in.	E	
	• Detail thickness ≥ 1.0- in.	E'	
	• With a transition radius with the end welds ground smooth, regardless of detail length		16
	• Transition radius ≥ 2.0 in.	D	
	• Transition radius < 2.0 in.	E	
	• With a transition radius with end welds not ground smooth	E	16
Transversely Loaded Fillet-Welded Attachments with Welds Parallel to the Direction of Primary Stress	Base metal at details attached by fillet welds:		16
	• With a transition radius with end welds ground smooth:		
	• Transition radius ≥ 2.0 in.	D	
	• Transition radius < 2.0 in.	E	
	• With a transition radius with end welds not ground smooth	E	
Mechanically Fastened Connections	Base metal:		21
	• At gross section of high-strength bolted slip-critical connections, except axially loaded joints in which out-of-plane bending is induced in connected materials	B	
	• At net section of high-strength bolted nonslip-critical connections	B	
	• At net sections of riveted connections	D	

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Table 6.6.1.2.3-1 - Fatigue Categories of Bridge Details, continued

GENERAL CONDITION	SITUATION	DETAIL CATEGORY	ILLUSTRATIVE EXAMPLE, SEE FIGURE 6.6.1.2.3-1
Eyebar or Pin Plates	Base metal at the net section of eyebar head, or pin plate	E	23, 24
	Base metal in the shank of eyebars, or through the gross section of pin plates with:		
	• Rolled or smoothly ground surfaces	A	23, 24
	• Flame-cut edges	B	23, 24

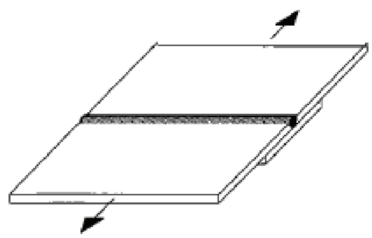
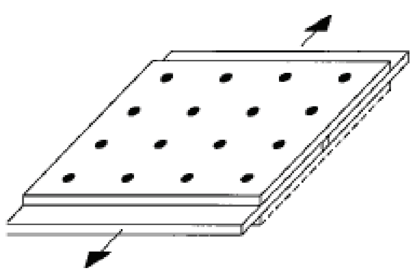
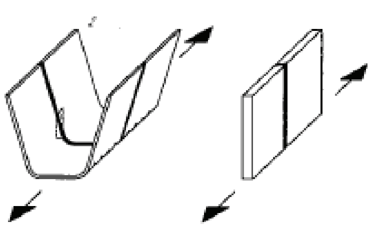
ILLUSTRATIVE EXAMPLE	DETAIL	DESCRIPTION OF CONDITION	DETAIL CATEGORY
	Transverse or longitudinal deck plate splice or rib splice Single-groove butt weld	(1) Ceramic backing bar. Weld ground flush parallel to stress.	B
		(2) Ceramic backing bar	C
		(3) Permanent backing bar. Backing bar fillet welds shall be continuous if outside of groove or may be intermittent if inside of groove.	D
	Bolted deck plate or rib splice	(4) In unsymmetrical splices, effects of eccentricity shall be considered in calculating stress.	B
	Deck plate or rib splice Double-groove welds	(5) Plates of similar cross-section with welds ground flush. Weld run-off tabs shall be used and subsequently removed, plate edges to be ground flush in direction of stress.	B
		(6) The height of weld convexity shall not exceed 20% or weld width. Run-off tabs as for (5).	C

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Table 6.6.1.2.3-1 - Fatigue Categories of Bridge Details, continued

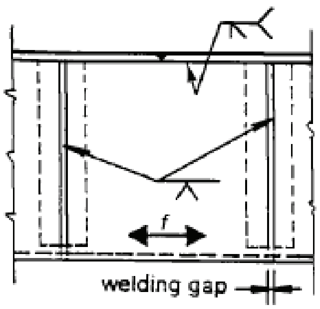
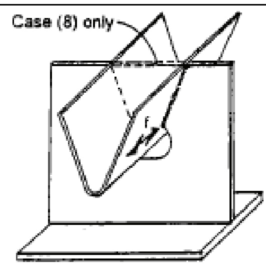
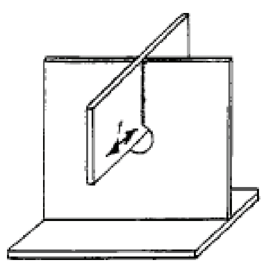
ILLUSTRATIVE EXAMPLE	DETAIL	DESCRIPTION OF CONDITION	DETAIL CATEGORY
	Welded rib "window" field splice Single groove butt weld	(7) Permanent backing bar for rib splice Welding gap> rib wall thickness *f* = axial stress range in bottom of rib	D
 	Rib wall at rib/floorbeam intersection Fillet welds between rib and floorbeam web	(8) Closed rib with internal diaphragm inside the rib or open rib *f* = axial stress range in rib wall at the lower end of rib/floorbeam weld (9) Closed rib, no internal diaphragm inside of rib $f = f_1 + f_2$ f_1 = axial stress range in rib wall f_2 = local bending stress range in rib wall due to out-of-plane bending caused by rib-floorbeam interaction, obtained from a rational analysis	C C

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Table 6.6.1.2.3-1 - Fatigue Categories of Bridge Details, continued

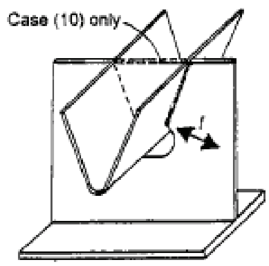
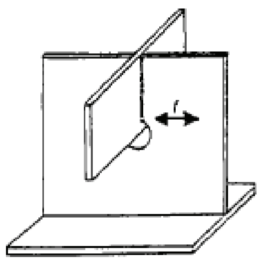
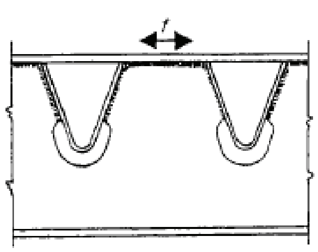
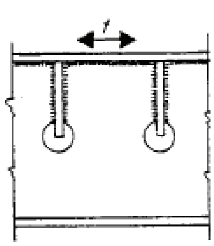
ILLUSTRATIVE EXAMPLE	DETAIL	DESCRIPTION OF CONDITION	DETAIL CATEGORY
 	Floorbeam web at rib/floorbeam intersection Fillet welds between rib and floorbeam web and between rib and internal diaphragm	(10) With internal diaphragm inside of closed rib, or open rib f = axial stress range component in floorbeam web perpendicular to weld = $f_1 + f_2$ f_1 = axial stress range in web f_2 = bending stress range in web due to out-of-plane bending caused by rib rotation at support Both stresses f_1 and f_2 to be obtained from a rational analysis	Lesser of C or Eq.6.6.1.2.5-3 C for combination fillet-groove welds
		(11) No internal diaphragm inside of closed rib $f = f_1 + f_2$ f = interaction stress range between the "tooth" of the floorbeam web and the rib wall obtained from a rational analysis f_2 as defined for (10)	C
 	Deck plate at the connection to floorbeam web	(12) f = axial stress range in the deck plate at the deck/floorbeam weld	E

Figure 8.1.41 *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, with 2005 Interim Revisions, Table 6.6.1.2.3-1 - Fatigue Categories of Bridge Details, continued

8.1.6 Fracture Critical Bridge Types

The following is a list of steel bridge superstructures which are susceptible to fatigue cracking and possible failure:

- Suspended spans with two girders (see Topic 8.3)
- Bar-chain suspension bridge with two eyebars per panel (see Topic 8.7)
- Welded tied arches with box shaped tie girder (see Topic 8.8)
- Simple span truss with two eyebars or single member between panel points (see Topic 8.6)
- Simple span single welded box girders with details such as termination of longitudinal stiffeners or gusset plate (see Topic 8.5)
- Simple span two-girder bridges with welded partial length cover plates on the bottom flange (see Topic 8.3)
- Continuous span two-girder system with cantilever and suspension link arrangement and welded partial length cover plates (see Topic 8.3)
- Simple span two-girder system with lateral bracing connected to horizontal gusset plates which are attached to webs (see Topic 8.3)
- Single welded I-girder or box girder pier cap with bridge girders and stringers attached by welding (see Topic 10.2)

Fatigue cracks can develop in steel bridges as a result of repeated loading. Generally, the stress range, number of cycles and type of detail are the most important factors that influence fatigue cracking. Recognizing and understanding the behavior of connections and details is crucial if the inspector is to properly inspect FCMs. Connections and details are often the locations of highest stress concentrations.

8.1.7 Fracture Criticality

Cracks and fractures have occurred in a large number of steel bridges. A report, *Manual for Inspecting Bridges for Fatigue Damage Conditions*, was prepared in 1990 under the support of the Pennsylvania Department of Transportation and the Federal Highway Administration to aid in the inspection of bridges. It summarizes the basic information on fatigue strength of bridge details and contains examples and illustrations of fatigue damage in welded, bolted, and riveted structures. A number of case histories are contained in *Fatigue and Fracture in Steel Bridges - Case Studies*, by John W. Fisher. *Fatigue Cracking of Steel Bridge Structures*, published by the Federal Highway Administration (FHWA) in March 1990, also contains valuable case studies of actual bridges. These three publications are listed in the Bibliography.

There are many factors which influence the fracture criticality of a bridge with FCMs, including:

- The degree of redundancy
- The live load member stress
- The propensity of the material to crack or fracture

- The condition of specific FCMs
- The existence of fatigue prone design details
- The previous number and size of loads
- The predicted number and size of loads

Details and Defects

Low fatigue strength details, such as cover-plated beams and welded web and flange gusset plates, should be carefully inspected on bridges that have experienced large numbers of stress cycles (see Figure 8.1.42).

Figure 8.1.42 Gusset Plate

Initial Defects

Initial defects, in many cases, are cracks resulting from poor quality welds between attachments and base metal (see Figure 8.1.43). Many of these cracks occurred because the groove-welded element was considered a "secondary" attachment with no established weld quality criteria (e.g., splices in longitudinal web stiffeners). Intersecting welds can provide a path for the crack to travel from secondary members to main members (see Figure 8.1.44).

Figure 8.1.43 Poor Quality Welds Inside Cross Girder

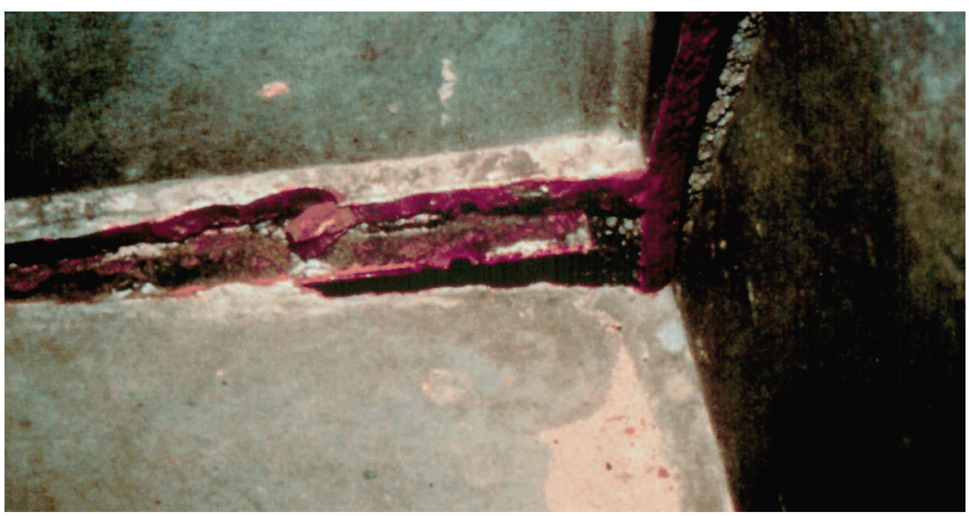


Figure 8.1.44 Intersecting Welds

Another category of fatigue cracks is related to connection geometry, such as at the end connections of stringers and floorbeams.

8.1.8 Inspection Procedures and Locations

While FCMs should be given special attention, care should be taken to assure that the remainder of the bridge or member is not ignored and that it is also inspected thoroughly. Bridge plans and shop drawings for bridges designed after about 1980 should have FCMs clearly identified. If FCMs are not clearly identified, the bridge inspector should use the guidelines previously described in this Topic along with the aid of a structural engineer.

According to the National Bridge Inspection Standards, a fracture critical member inspection is defined as a hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation. FCM inspections are to be performed at intervals not to exceed twenty-four months. Certain FCMs may require a frequency of less than twenty-four months. Criteria must be established to determine the level and frequency of these inspections based on such factors as age, traffic characteristics, and known deficiencies.

Procedures

Visual

The inspection of steel bridge members for defects is primarily a visual activity. Most defects in steel bridges are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is required. More exact visual observations can also be employed by cleaning suspect areas, removing paint when necessary, and using a magnifying unit.

Physical

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size and location of the suspected defect. Care should be taken in cleaning when the suspected defect is a crack. When cleaning steel surfaces, any type of cleaning process that would tend to close discontinuities, such as blasting, should be avoided. The use of degreasing spray before and after removal of the paint may help in revealing the defect.

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Once the presence of a fatigue crack has been verified, the inspector should examine all other similar locations and details.

Advanced Inspection Techniques

Several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Magnetic particle
- Eddy current
- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers
- Corrosion sensors
- Smart paint 1
- Smart paint 2
- Computer tomography
- Robotic inspection
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

Other methods, described in Topic 13.3.3, include:

- Brinell hardness test
- Charpy impact test
- Chemical analysis
- Tensile strength

Inspection of Details

Fracture critical member inspection should focus on those details that are most susceptible to fatigue problems.

Of all the details, those of AASHTO fatigue Categories E and E' are the most susceptible to fatigue crack growth. If Category E or E' details are found on a FCM, they should be carefully inspected during each inspection and their presence clearly documented in the inspection report.

Recordkeeping and Documentation

The consequences of deficiencies on bridges with FCMs can be very serious. The ability to verify a defect at the bridge site or to correctly evaluate it in the office will depend on the proper recording and documenting of field conditions. Since many defects become obvious only as time passes, complete, clear, and concise recordkeeping provides a valuable reference for comparison in future bridge inspections.

When a deficiency is encountered in a FCM, all relevant information should be recorded carefully and thoroughly, including:

- Method of inspection: visual, physical or advanced (see Topics 2.3 and 13.3)
- The date the deficiency was detected, confirmed, and re-examined
- The type of deficiency, such as cracks, notches, nicks, or gouges, defects in welds, excessive corrosion, or apparent distortion, mislocation, or misalignment of the member
- The general location of the deficiency, such as "at Panel Point L5 of the downstream truss" or "at the lower end of connection plate of Floorbeam No. 4 to the north girder of the eastbound bridge"
- Detailed sketches of the location, shape, and size of the deficiency; extra care should be given to determine the location of the ends of cracks
- The dimensions and details of the member containing the deficiency
- Any noticeable conditions at cracks when vehicles traverse the bridge, such as opening and closing of the crack or visible distortion of the local area
- Any changes in shape or condition of adjacent elements or members
- The presence of corrosion or the accumulation of dirt and debris at the general location of the deficiency
- Weather conditions when the deficiency was discovered or inspected

Label the member using paint or other permanent markings: mark the ends of the crack, the date, compare to any previous markings, be sensitive to aesthetics at prominent areas (see Figure 8.1.51). Photograph and sketch the member and the defect.

Refer to Topics 4.3 and 4.4 for general record keeping, documentation and inspection report writing.

Recommendations

When deficiencies are encountered in FCMs, the repair of the condition generally demands a high priority. The defects listed for repair should be listed in order of priority. For example, a crack in a flange is more significant than surface corrosion of the web. There are two general classifications for repairs of FCMs:

Urgent repairs - repairs that are required immediately in order to maintain the life of the structure or to keep the bridge open; these repairs are for bridge-threatening defects

Programmed repairs – may be worked into the normal maintenance schedule; these repairs are for non-threatening deficiencies and activities such as cleaning and painting of structural steel

Locations

Welded Details

Welded details tend to be less forgiving of small weld discontinuities than riveted details because welds are more sensitive to repeated stresses. Once cracking starts to develop, it can destroy the member base metal as a result of the continuous path provided by the welded connections. Cracking has developed more frequently in welded bridges because of flaws that escape detection, the use of details less fatigue resistant than assumed in the design, and secondary and displacement induced stresses.

Fatigue cracks can also develop at welded details in the compression regions of steel bridge members. However, when a crack propagates out of the weld tensile-residual-stress zone and into the adjacent compression region, the crack growth usually stops. Because of this, the inspection of welded details in regions of nominal compressive stress is of lower priority than in tension regions.

Effective and intense inspection for fatigue cracks in welded bridges should be performed at several locations.

For main members, inspect the following locations:

- Ends of welded cover plates
- Groove welds in flange plates
- Butt welds in longitudinal stiffeners
- Web plates with cutouts and filler welds
- Intersecting groove welds
- Welded repairs and reinforcement
- Back-up bar splices
- Stress Risers

For connections and attachments, inspect the following locations:

- Cut short flanges
- Coped beam ends
- Blocked flange plates
- Welded rigid connections of cross girders at bents
- Welded flange attachments
- Intersecting welds at gusset plates and diaphragms

Riveted and Bolted Details

Fatigue cracking can occur at riveted or bolted details, particularly as the result of secondary or displacement-induced stresses, or from assembly tack welds which have not been removed. Framing connections intended to provide only simple support, but by their function tend to resist bending moment, are suspect locations. Fortunately, crack growth is often inhibited by multiple element members, i.e., internal redundancy.

Most riveted bridges were constructed prior to the 1960's when bolted connections became common. Because of their age and the number of stress cycles already experienced, the close inspection of riveted members and connections in bridges with high truck traffic volume is necessary. In general, the locations where fatigue cracks develop in riveted bridges are similar to those in welded bridges:

- ➢ Cracking and prying of rivets/bolts at end connections
- ➢ End connection angle failure (see Figure 8.1.45)
- ➢ Girder webs at floorbeam connections
- ➢ Floorbeam connections to girders
- ➢ Diaphragm connections to girders
- ➢ Cantilever bracket connections to girders
- ➢ Truss hangers
- ➢ Eyebars
- ➢ Tack welds

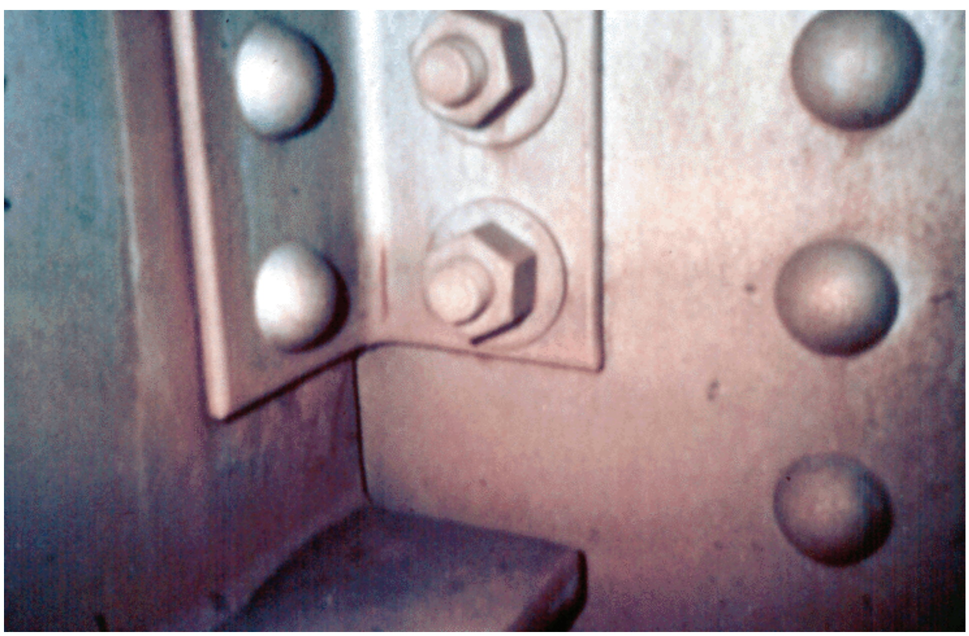


Figure 8.1.45 Cracked Stringer Connection Angle

Copes

Coping or cutting away of the flange and portion of the web, may be necessary to connect stringers, floorbeams, diaphragms and the main girders. Copes are often flame cut, resulting in residual tensile stresses along the cut edges, approaching the yield point (see Figure 8.1.46).

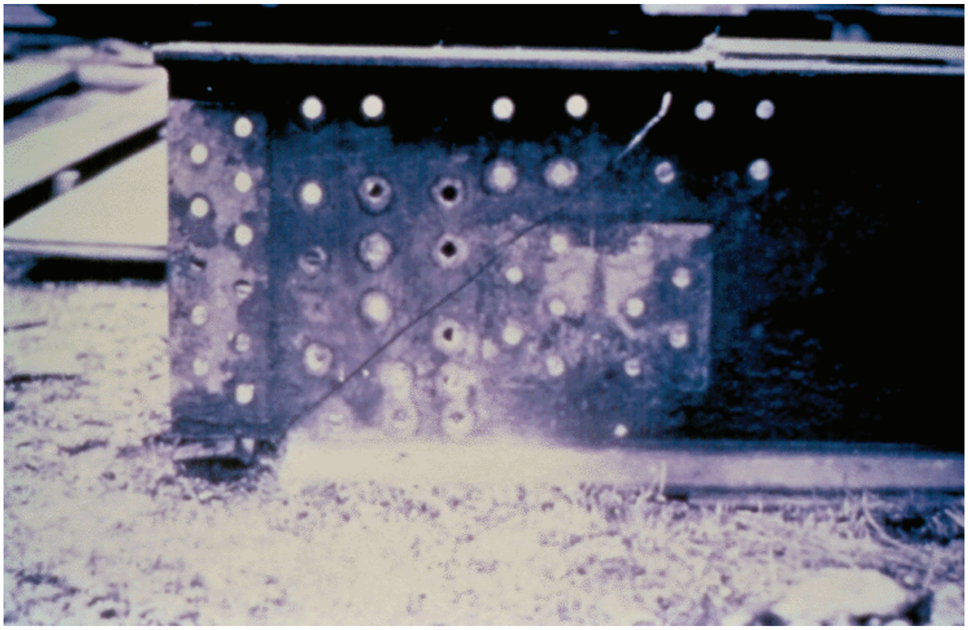


Figure 8.1.46 Fracture of a Coped Member

Flange Terminations

It is also common to terminate the flange before the end connection to facilitate fabrication (see Figure 8.1.47). When one or both flanges are removed, as in a blocked flange cut, the web plate has a lower cross section as compared to the entire member. This can increase the stress in the web plate where the flange is terminated by 200 to 300 percent.

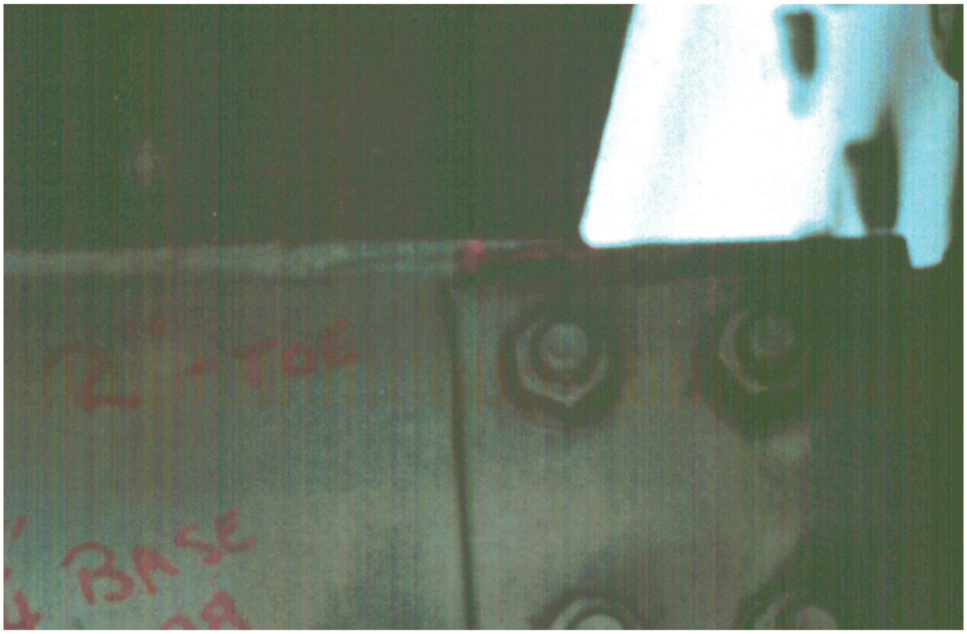


Figure 8.1.47 Flange Termination

End Restraints

A related type of cracking develops in the end connection angles. End rotation can deform the connection angle out-of-plane. This results in cracking of the angle often at the fillet or the bolt/rivet line (see Figure 8.1.48 and 8.1.49). In some cases, the rivet or bolt heads will crack off if the angle is relatively thick.

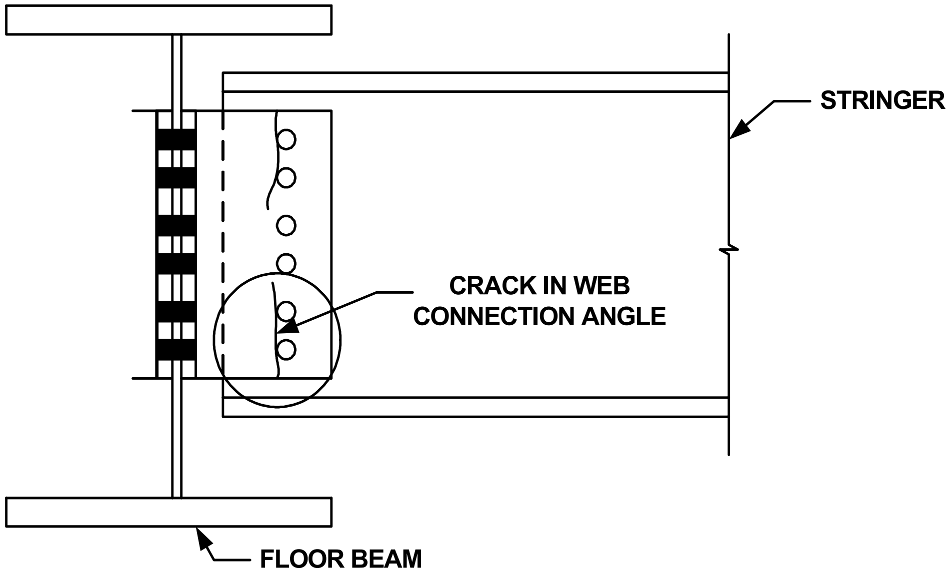


Figure 8.1.48 Cracked Stringer to Floorbeam Connection Angle

Figure 8.1.49 Cracked Stringer to Floorbeam Connection Angle

Out-of-Plane Distortion

Deflection of floorbeams or diaphragms can cause out-of-plane distortion in the girder webs. Cracks caused by out-of-plane distortion are not covered in AASHTO Fatigue Categories A - E'. Out-of-plane distortion occurs across a small web gap between the flanges and end of vertical connection plates (see Figure 8.1.50). When cracks form in planes parallel to the stresses between the flanges and end of vertical connection plates they are not typically detrimental to the performance of the structure.

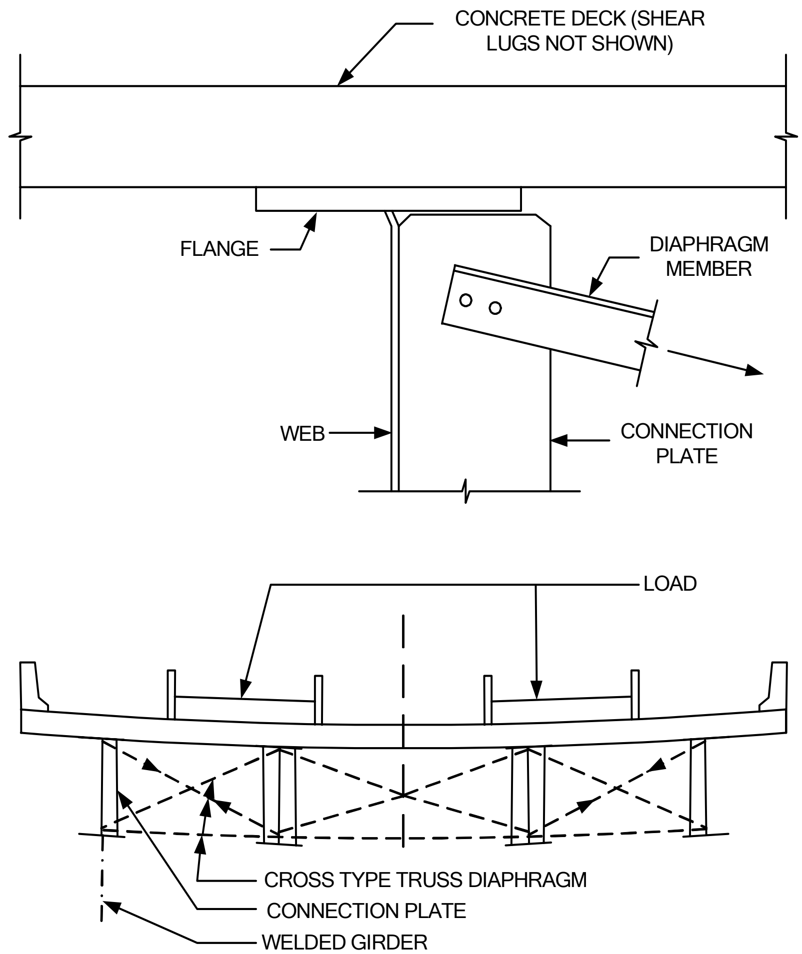


Figure 8.1.50 Out-Of-Plane Distortion in Web Gap at End of Transverse Connection Plate

Two very common instances are in the web gap at unconnected cross-frame connection plates, that is, connection plates which are not attached to the beam or girder flanges, and at similar web gaps at floorbeam connections to main girders.

The problem of distortion-induced fatigue cracking has developed in many types of bridges, including:

- Trusses
- Suspension bridges
- Two-girder bridges
- Multi-beam and multi-girder bridges
- Tied arch bridges
- Box girder bridges

When distortion-induced cracking develops in a bridge, there are usually large numbers of cracks that form before corrective action is taken, because the cyclic stresses are often very high. As a result, many cracks form simultaneously in the structural system (see Figure 8.1.51).

Figure 8.1.51 Crack Near Top Flange at a Diaphragm Connection Plate which is on the Opposite Side of the Web

For out-of-plane distortion, inspect the following locations:

- Girder webs at floorbeam and diaphragm connections (see Figure 8.1.52)
- Ends of diaphragm connection plates in girder bridges
- Box girder webs at diaphragms
- Lateral bracing gusset plates on girder webs at floorbeam connections
- Floorbeam and cantilever bracket connections to girders
- Pin connected hanger plates and fixed pin plates

Once an out-of-plane bending crack is identified, it is extremely important that all similar locations on the structure also be carefully inspected to search for similar damage.

Even though distortion induced cracks are usually parallel to the primary stress direction, they can turn perpendicular. Retrofits such as drilled holes are often used when cracks turn perpendicular to the primary stresses.

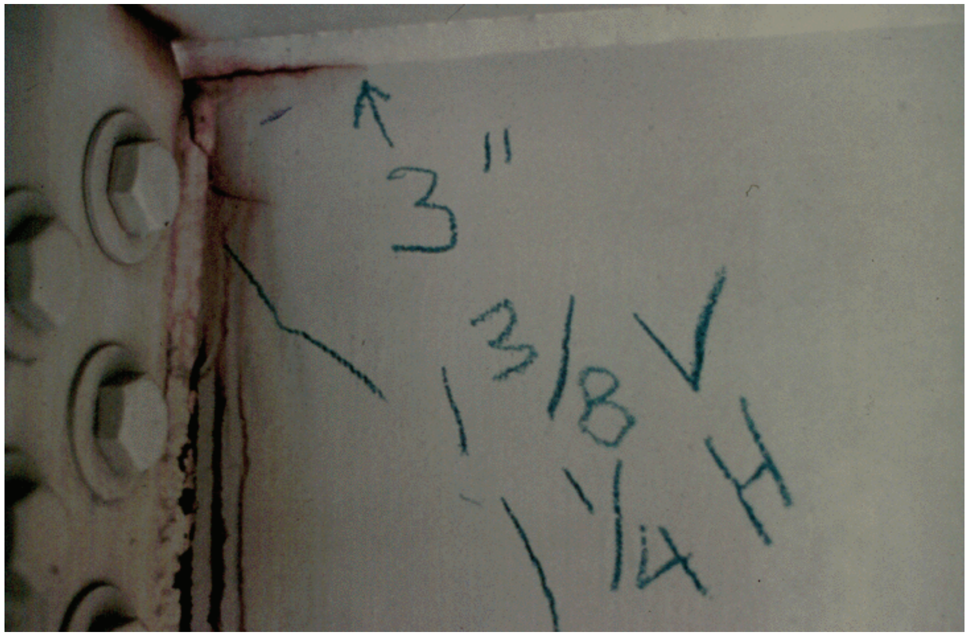


Figure 8.1.52 Cracks at Top of Floorbeam Connection to Girder

Cracks Perpendicular to Primary Stress

Cracks perpendicular to primary stress are very serious because all stresses applied to the member will work towards propagating the crack (see Figure 8.1.53). The inspector should report them immediately so that repairs can be performed.

Cracks Parallel to Primary Stress

Cracks parallel to primary members are less serious than transverse cracks. Cracks parallel to the main direction of stress, do not reduce the capacity load and have less tendency to propagate. These cracks are still important because they can turn perpendicular to the direction of stress at any time (see Figure 8.1.53).

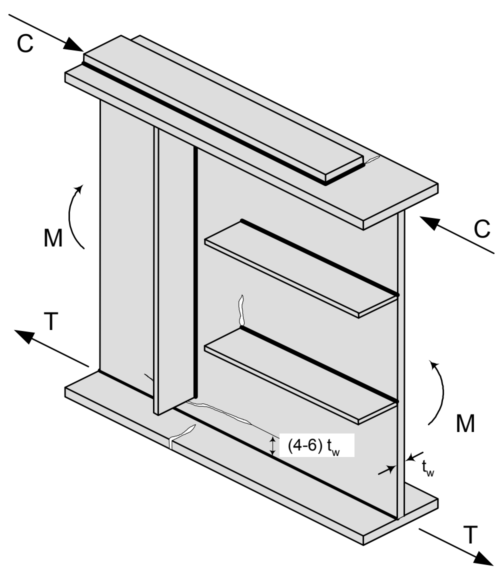


Figure 8.1.53 Cracks Perpendicular or Parallel to Applied Stress

Corrosion

Corrosion is probably the most common form of defect found on steel bridges. More section loss results from corrosion than from any other cause. However, few bridge failures can be attributed solely to corrosion. Shallow surface corrosion is generally not serious but is quite common when the paint system has failed. Measurable section loss is significant as it may reduce the structural capacity of the member.

Nicks or gouges will often need to be evaluated by the bridge engineer responsible for the rating of the structure because they cause stress concentrations and may result in fatigue cracking. If large, they should be evaluated in a manner similar to section loss occurring due to corrosion.

8.1.9 Evaluation

State and federal rating guidelines systems have been developed in order to provide continuity in the inspection of steel superstructures. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component rating method and the AASHTO element level condition state assessment method using the *AASHTO Guide for Commonly Recognized (CoRe) Structural Elements*.

NBI Rating Guidelines and Element Level Condition State Assessment

Refer to Topics 5.3, 8.2 through 8.9 10.1 and 10.2 for specific rating guidelines for the various types of steel bridge components.

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Table of Contents

Section 8
Inspection and Evaluation of Common Steel Superstructures

Topic 8.2 Rolled Steel Multi-beams and Fabricated Steel Multi-girders

8.2.1 Introduction

The two basic steel superstructure types, rolled steel multi-beams and fabricated steel multi-girders, have similar characteristics; however, there are some primary differences that make each superstructure type unique.

One of the simplest differences is the terminology. Although many designers and inspectors use the terms "beam" and "girder" interchangeably, there is a difference. In steel fabrication, the word "beam" refers to rolled shapes, while the word "girder" refers to fabricated members. Girders are fabricated from web and flange plates.

Rolled beams are generally "compact" sections that satisfy ratios for the flange and web thicknesses to prevent buckling. Rolled beams come in a number of different sizes with each size having specific dimensions for the width and thickness for both the flange and web. These dimensions are standard and can be found in a number of publications, such as the *Manual for Steel Construction, Load and Resistance Factor Design* published by the American Institute of Steel Construction, Inc. Also, rolled beams may have bearing stiffeners but no intermediate stiffeners.

Fabricated girders are different from rolled beams in that they are custom made for specific bridge site conditions. The width and thickness of the flanges and webs can be varied to the necessary dimensions to optimize the design. Fabricated girders generally have bearing stiffeners and intermediate stiffeners.

8.2.2 Design Characteristics

Rolled Multi-beam

The steel rolled multi-beam bridge is a configuration of three or more parallel rolled beams with a deck placed on top of the beams. The most common use of this superstructure type is for simple spans, with span lengths from 9 to 15 m (30 to 50 feet) (see Figure 8.2.1). Continuous span designs have also been used, some of which incorporate pin and hanger connections (see Figure 8.2.2). Rolled beams are manufactured in structural rolling mills from one piece of steel (i.e., the flanges and web are manufactured as an integral unit). Rolled beams in the past were generally available no deeper than 915 mm (36 inches) in depth but are now available from some mills as deep as 1200 mm (48 inches).

Figure 8.2.1 Simple Span Rolled Multi-beam Bridge

Figure 8.2.2 Continuous Span Rolled Multi-beam Bridge with Pin & Hanger

In the past, a common method of economically increasing the capacity of a rolled multi-beam bridge was to weld partial length cover plates to the flanges (see Figure 8.2.3). The cover plates increased a beam's bending strength. This practice also creates a fatigue prone detail in the tension flange, which may lead to cracking. The cover plates are attached by riveting or welding. Fatigue cracking occurs in the beam flanges at the ends of partial length cover plates.

Figure 8.2.3 Rolled Multi-beam Bridge with a Cover Plate

Fabricated Multi-girder

The steel fabricated multi-girder bridge is similar to the rolled multi-beam bridge in appearance. However, fabricated girders are larger than those that could be provided by the rolling mills. Older fabricated multi-girders are riveted or bolted built-up members consisting of angles and plates (see Figure 8.2.4). In a riveted or bolted built-up member, the angles are considered part of the flange. Today's fabricated multi-girders are usually welded plate members (see Figure 8.2.5).

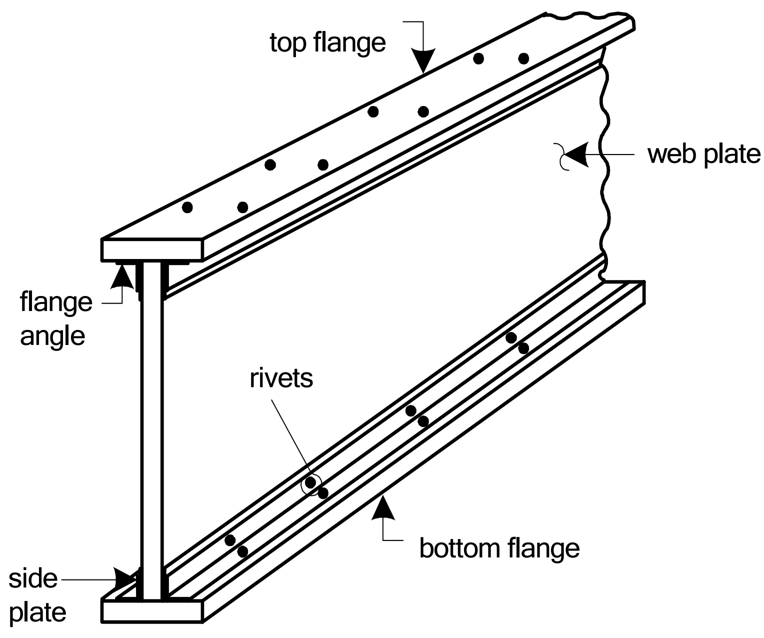

Figure 8.2.4 Built-up Riveted Plate Girder

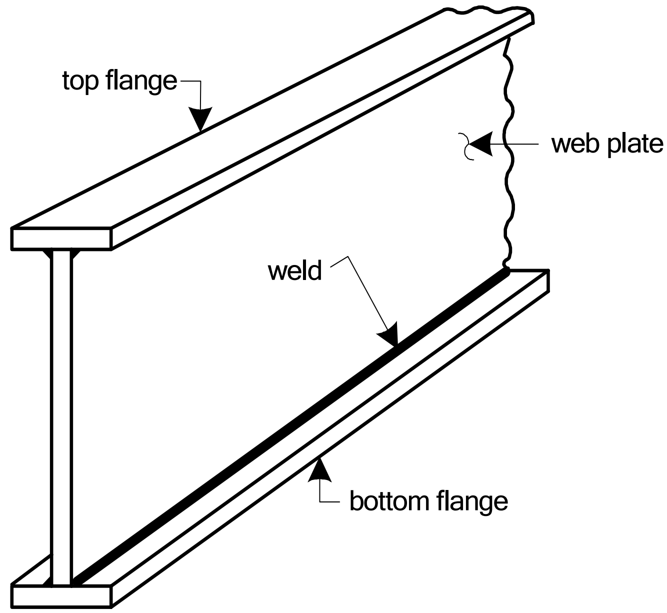

Figure 8.2.5 Welded Plate Girder

This bridge type can be found in single span (see Figure 8.2.6), multiple span, and continuous span designs (see Figure 8.2.7), and it is widely used when curved bridges are required (see Figure 8.2.8). Continuous welded multi-girders have been built for spans of over 152 m (500 feet). Pin and hanger connections are also found in multi-girder construction (see Figure 8.2.9).

Figure 8.2.6 Single Span Fabricated Multi-girder Bridge

Figure 8.2.7 Continuous Span Fabricated Multi-girder Bridge

Figure 8.2.8 Curved Fabricated Multi-girder Bridge

Fabricated multi-girder bridges have three or more primary load paths (girders). Two-girder bridge systems are discussed in Topic 8.3.

Figure 8.2.9 Fabricated Multi-girder Bridge with Pin & Hanger Connection

Sometimes, both types of superstructure, rolled steel beams and fabricated steel girders can be used on the same bridge (see Figure 8.2.10). The shorter approach spans are rolled beams while the longer main span utilizes fabricated girders.

Figure 8.2.10 Combination Rolled Beams and Fabricated Girders

Haunched Girder Design

In continuous girder designs, additional girder strength is required in negative moment regions. This is accomplished through a method called haunching. Haunching is the increasing of the web depth for a specified portion of the girder. The regions above intermediate supports (i.e. piers and bents) have negative moments larger than the adjacent positive moments. Typically, the girder depth used at the positive moment regions is not sufficient enough to resist these moments, so the web depth needs to be increased. (See Topic P.2 on Moments and Shear) However, instead of increasing the depth for the full length of the girder, the girder is haunched at the intermediate supports.

Three methods have been used to haunch girders.

To haunch a riveted plate girder, a larger web plate size is used in the region required.

To haunch a rolled beam, the bottom flange is separated from the web and an insert plate of the required depth is welded in place (see Figure 8.2.11).

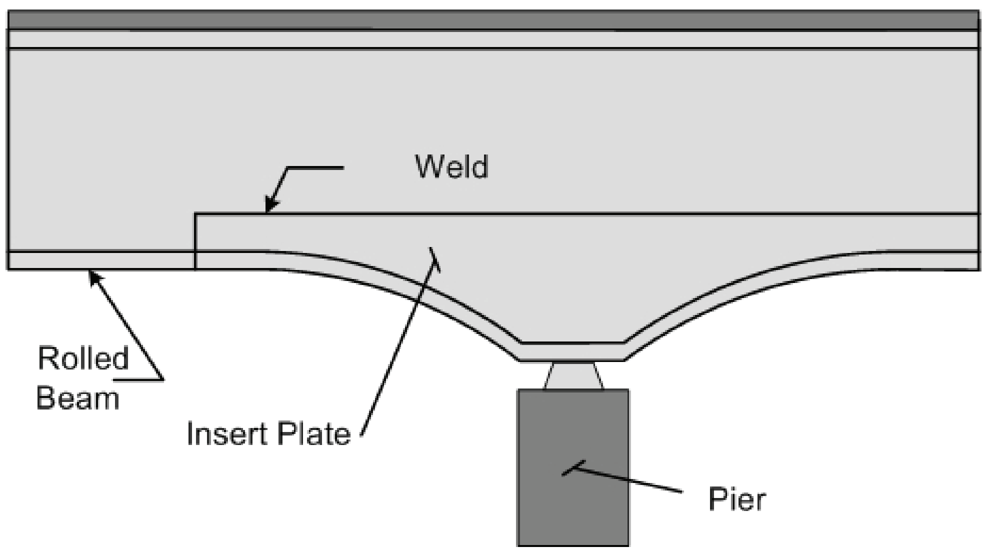

Figure 8.2.11 Web Insert Plate for Multi-beam

A fabricated variable depth girder is the method used today (see Figure 8.2.12). The web plate is simply fabricated to the required depth. The top and bottom flange plates are then welded to the web plate.

Figure 8.2.12 Fabricated Variable Depth Girder Bridge

Function of Stiffeners

As fabricated girders become longer, the depth of the web plate increases, and it becomes susceptible to web buckling (i.e., failure of the web due to compressive or shear stresses). Bridge designers prevent this from occurring by increasing the web thickness or by reinforcing the web with steel stiffener plates. Stiffeners can be either transverse (vertical) or longitudinal (horizontal). They can be placed on one or both sides of the web. The stiffeners limit the unsupported length of the web, which results in increased stability of the girder.

Primary and Secondary Members

The primary members of a rolled multi-beam bridge are the rolled beams, and the secondary members are the diaphragms (see Figure 8.2.13). Intermediate and end diaphragms are provided to stabilize the beams during construction and to help distribute the live load more evenly to the rolled beams. Diaphragms may or may not be present on the multi-beam bridge.

Figure 8.2.13 Rolled Beam (Primary Member) with Diaphragm (Secondary Member)

The primary members of a fabricated multi-girder bridge are the fabricated girders, as well as the diaphragms on a curved bridge. In the case of a curved structure, the diaphragms are designed to withstand the torsional loading attributed to curved structures and therefore, are also considered primary members (see Figure 8.2.14).

On straight multi-girder bridges, diaphragms are considered secondary members. Similar to rolled beam bridges, diaphragms are provided to stabilize the girders during construction and to help distribute secondary live load (see Figure 8.2.15). Diaphragms can be rolled shapes (e.g., I-beams and channels) or they can be cross frames constructed from angles, tee shapes, and plates. They are usually attached to transverse web stiffeners which are normally referred to as connection plates. On older bridges, secondary members also include lateral bracing. Current design specifications discourage the use of lateral bracing. This is due to connections for lateral bracing being fatigue-prone.

Figure 8.2.14 Curved Multi-girder Bridge

Figure 8.2.15 Straight Multi-girder Bridge

Fatigue Prone Details

Some common areas for fatigue prone details are:

- Welded cover plates on the tension flange
- Attachment welds in the tension zone
- Longitudinal and transverse stiffeners (intersections of welds)
- Fatigue cracks can also occur due to web-gap distortion and out-of-plane distortion
- Welded, bolted, or riveted connections

Inspection of these areas will be discussed further detail in Topic 8.2.4.

Fracture Critical Areas

Both rolled multi-beam bridges and steel multi-girder bridges consist of a minimum of three beams or girders and have load path redundancy. Since load path redundancy is achieved, these bridge types do not contain any fracture critical members.

8.2.3 Overview of Common Defects

Common defects that occur on steel multi-beam and fabricated multi-girder bridges are:

- Paint failures
- Corrosion
- Fatigue cracking
- Collision damage
- Overloads
- Heat damage

See Topic 2.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 8.1 for Fatigue and Fracture in Steel Bridges.

8.2.4 Inspection Procedures and Locations

Inspection procedures to determine other causes of steel deterioration are discussed in detail in Topic 2.3.8.

Procedures

Visual

The inspection of steel bridge members for defects is primarily a visual activity.

Most defects in steel bridges are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is typically required. More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size and location of the suspected defect. Care should be taken in cleaning when the suspected defect is a crack. When cleaning steel surfaces, any type of cleaning process that would tend to close discontinuities, such as blasting, should be avoided. The use of degreasing spray before and after removal of the paint may help in revealing the defect.

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The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, the inspector should examine all other similar locations and details.

Advanced Inspection Techniques

Several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Acoustic emissions testing
- Computer tomography
- Corrosion sensors
- Smart paint 1
- Smart paint 2
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 13.3.3, include:

- Brinell hardness test
- Charpy impact test
- Chemical analysis
- Tensile strength test

Locations

Bearing Areas

Examine the web areas over the supports for cracks, section loss or buckling. If bearing stiffeners, jacking stiffeners and diaphragms are present at the supports inspect them for cracks, section loss and buckling also.

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. See Topic 9.1 for a detailed presentation on the inspection of bearings.

Shear Zones

Examine the web areas near the supports for any section loss or buckling (see Figure 8.2.16). Shear stresses are greatest near the supports. Therefore, the condition of the web is more critical near the supports than at mid-span. Also investigate the web for buckling due to overloads. If girders have been haunched by the use of insert plates, check the weld between the web and the insert plate.

Figure 8.2.16 Corroded Shear Zone on a Rolled Multi-beam Bridge

Flexure Zones

The flexure zone of each beam/girder includes the entire length between the supports (see Figures 8.2.17 and 8.2.18). Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, and gouges. Check the flanges in high stress areas for bending or flexure- related damage. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the beam/girder over the intermediate supports have high flexural stresses due to negative moment (see Figures 8.2.19 and 8.2.20). If welded cover plates are present, check carefully at the ends of the cover plates for cracks.

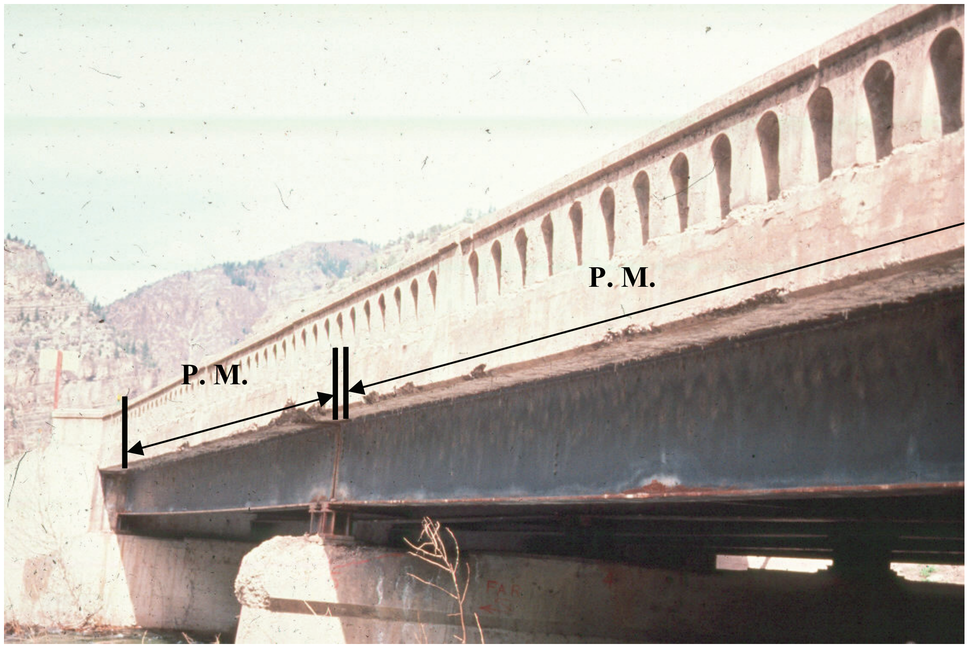

Figure 8.2.17 Flexural Zone on a Simple Rolled Multi-beam Bridge is Entire Length of Beams Between Supports

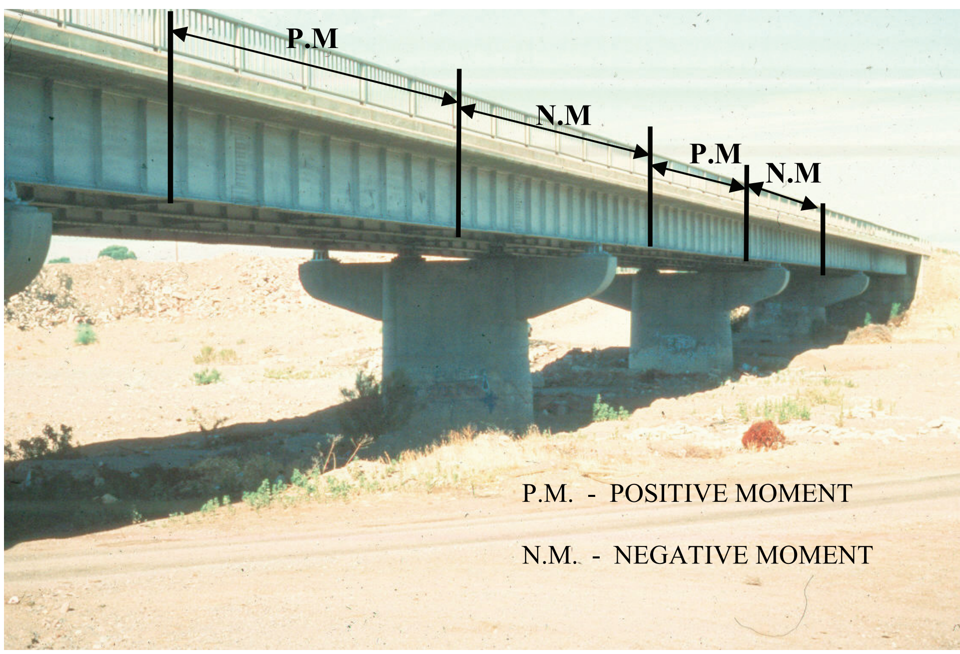

Figure 8.2.18 Flexural Zone on a Fabricated Multi-girder Bridge

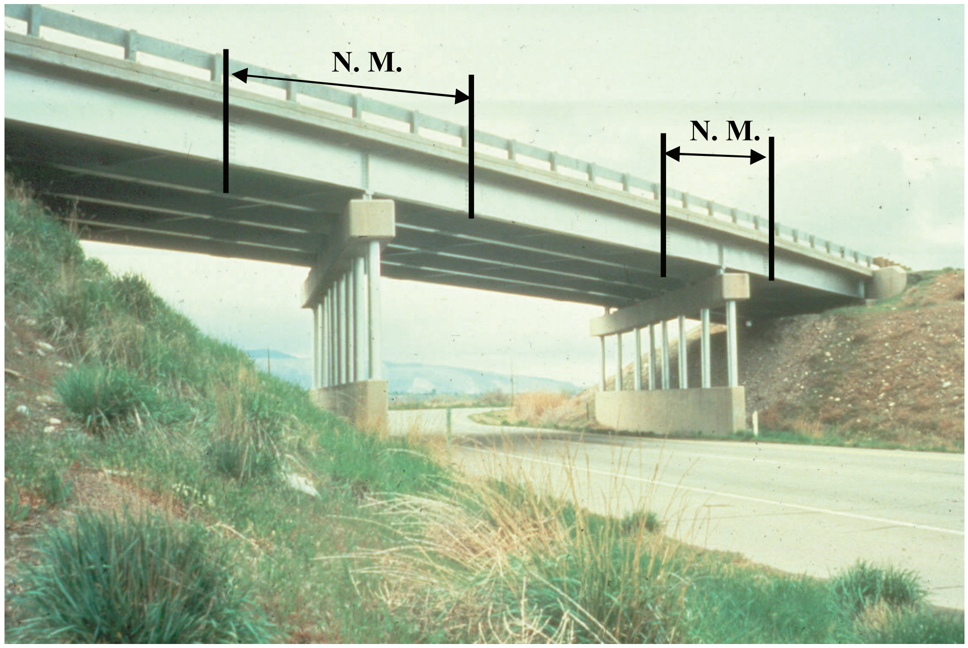

Figure 8.2.19 Negative Moment Region on a Continuous Rolled Multi-beam Bridge

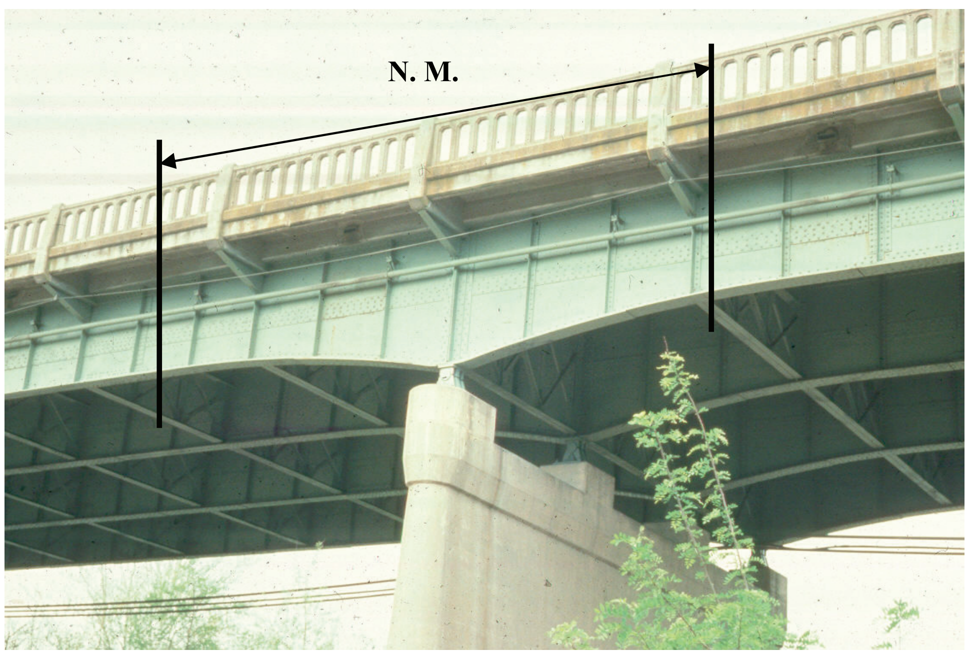

Figure 8.2.20 Negative Moment Region on a Continuous Fabricated Multi-girder Bridge

Secondary Members

Examine the diaphragm and bracing connections for loose fasteners or cracked welds (see Figures 8.2.21 and 8.2.22). This problem is most common on skewed bridges, and it has also been observed on bridges with a high frequency of combination truck loads. Check horizontal connection plates, which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Check for distorted members. Distorted secondary members may be an indication the primary members may be overstressed or the substructure may be experiencing differential settlement.

Figure 8.2.21 End Diaphragm

Figure 8.2.22 Intermediate Diaphragm

Areas That Trap Water and Debris

Check horizontal surfaces that can trap debris and moisture which are susceptible to a high degree of corrosion and deterioration. Areas that trap water and debris can result in active corrosion cells and excessive loss in section. This can result in notches susceptible to fatigue or perforation and loss of section.

On multi-beam and fabricated multi-girder bridges check:

- Along the bottom flanges
- Pockets created by diaphragm connections
- Lateral bracing gusset plates
- Areas exposed to drainage runoff

Areas Exposed to Traffic

Check underneath the bridge for collision damage to the main girders and bracing if the bridge crosses over a highway, railway, or navigable channel. Document any cracks, section loss, or distortion found (see Figures 8.2.23 and 8.2.24).

Figure 8.2.23 Collision Damage on a Rolled Multi-beam Bridge

Figure 8.2.24 Collision Damage on a Fabricated Multi-girder Bridge

Fatigue Prone Details

Dirt and debris traps can result in active corrosion cells when water and salt are present. These corrosion cells can lead to excessive section loss. This corrosion can result in notches that are susceptible to fatigue or perforation.

If the structure has been painted, breaks in the paint accompanied by rust staining indicate the possible existence of a fatigue crack. Investigate the areas surrounding field splice plates on the tension flange (see Figure 8.2.25). The suspected crack area should be cleaned to determine the existence of a crack and its extent. If a crack with rust staining exists in the paint, the fatigue cracks in the steel can already be up to 6 mm (1/4 inch) deep in the beam flange. Check any attachment welds located in the superstructure tension zones, such as traffic safety features, lighting brackets, utility attachments, catwalks and signs (see Figure 8.2.26).

Check web stiffener welds, welded web/flange splices and intersecting welds. Welds are considered to be intersecting if they run through each other, overlap, touch or are within 6 mm (1/4 inch) from each other. Intersecting welds or narrow gaps between perpendicular welds are stress risers and can lead to crack initiation. The restraining effect of the intersecting plate elements cause large residual stresses during the cooling process of fabrication. These residual stresses can lead to cracking and reduced fatigue strength. To avoid intersecting welds, welds should terminate short of the intersection by at least ¼". In most cases, it is desirable to allow the longitudinal weld (parallel with the applied stress) to be continuous. This avoids Category E type detail at the weld termination if it is interrupted. The end termination of a transverse weld does not directly affect its fatigue strength and is classified as Category C' for plates.

If the girder is riveted or bolted, check all rivets and bolts to determine that they are tight and in good condition. Check for cracked or missing bolts, rivets and rivet heads. Also, check the base metal around the bolts and rivets.

Inspect the member for misplaced holes or repaired holes that have been filled with weld material. Check for plug welds which are possible sources of fatigue cracking.

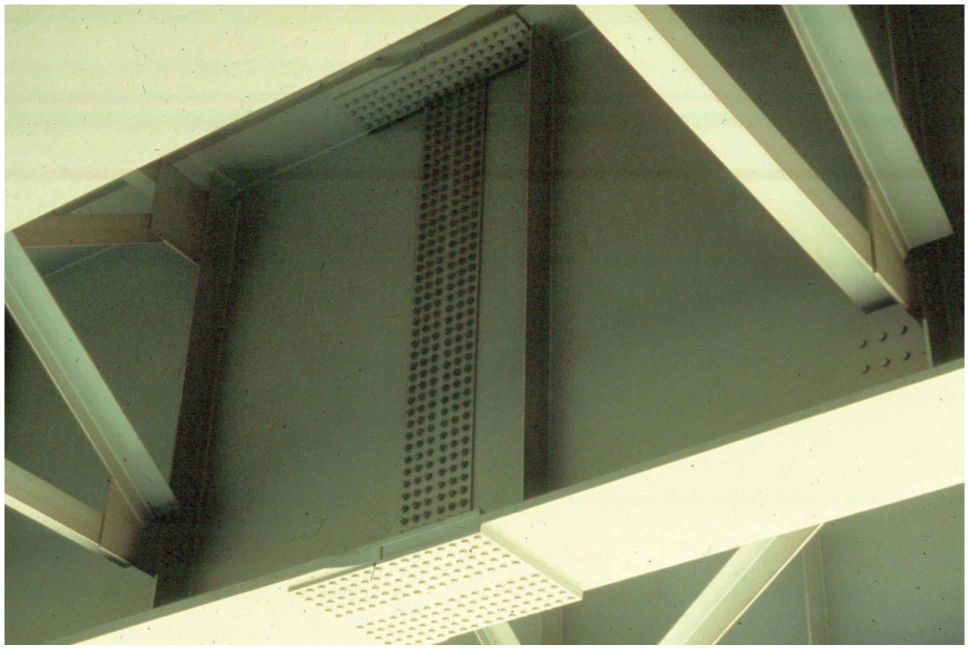

Figure 8.2.25 Field Splice

Figure 8.2.26 Welded Attachment in Tension Zone of a Beam

Fracture Critical Members

Both rolled multi-beam bridges and fabricated multi-girder bridges have load path redundancy, and therefore have no fracture critical members.

Out-of-plane Distortion

Out-of-plane distortion can occur in several areas that can lead to web cracks near the flanges of steel bridges. The following are some common areas for out-of-plane distortion.

Girder Webs at Diaphragm Connections

Diaphragms between multi-girders exert out-of-plane forces to the girder webs through the vertical connection plates. The connection plates are usually sufficient to transmit the forces to the girders. The structural details at the ends of the connection plates sometimes are inadequate to accommodate the deflections and rotations.

Connection plate at top flange - One type of connection detail which has incurred a large number of fatigue cracks is the end of diaphragm connection plates which are not attached to the top tension flange of continuous girder bridges. While the top flange is rigidly embedded in the bridge deck slab, and the connection plate itself is stiff enough to resist rotation and bending from the diaphragm, most of the out-of-plane distortions (perpendicular to the web) concentrate in the local region of the web above the upper end of the connection plate. Fatigue cracks develop in this region as a result of the web plate bending. The cracks are usually horizontal along the web-to-flange weld, and also propagate as an upside down U along the upper ends of the fillet welds of the connection plate. Detection of cracks of such length is not difficult. Knowing that unattached ends of diaphragm connection plates are likely locations of fatigue cracks increases the certainty of early detection of these cracks (see Figures 8.2.27 and 8.2.28).

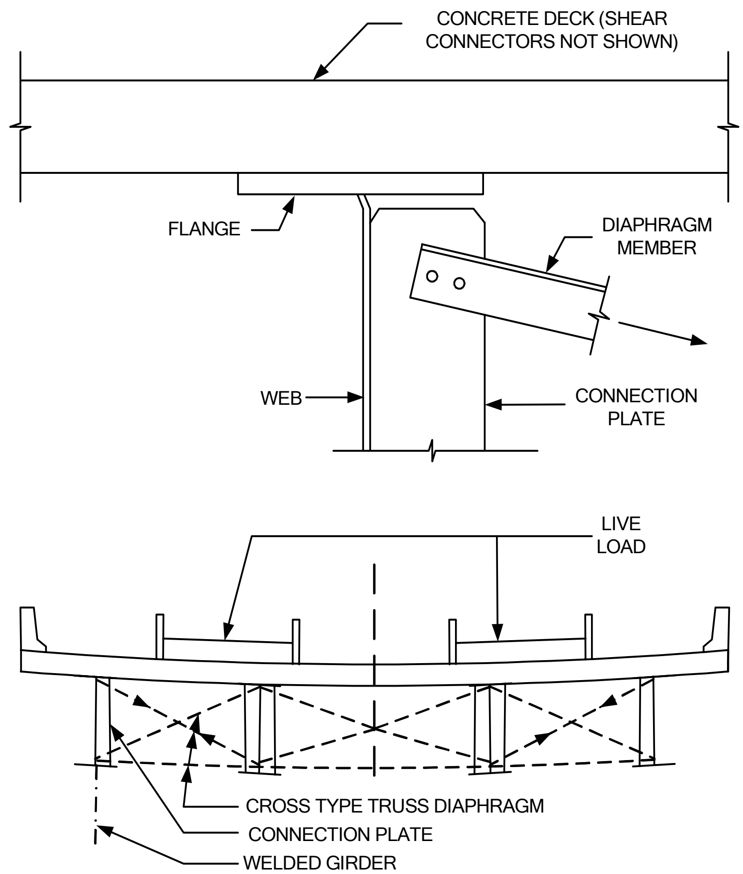

Figure 8.2.27 Out-of-plane Distortion in Web Gap at Diaphragm Connections

Figure 8.2.28 Web Crack due to Out-of-plane Distortion at Top Flange

Connection plate at bottom flange - At the lower end of diaphragm connection plates which are not welded to the tension flange of girders, the condition of local out-of-plane distortion and bending of the web plate usually is less severe. This is because the tension flange is not restrained from lateral movement, which is sufficient to reduce the web plate bending. However, if the bottom flange is restrained from lateral deflection (e.g., at bearings), fatigue cracks will develop along the web to flange weld (see Figure 8.2.29).

Figure 8.2.29 Web Crack due to Out-of-plane Distortion at Bottom Flange

Skewed bridges - Another location where fatigue cracking has developed at the unattached lower end of diaphragm connection plate is in skewed bridges. Most of these diaphragms are perpendicular to the girders and thus are subjected to large differential vertical deflections which in turn cause out-of-plane distortion at the lower end of the diaphragm connection plate. If the girder flange is relatively thick and stiff against lateral displacement, most of the deflection is accommodated by bending of the web plate within the gap between the flange and the end of the connection plate welds. Fatigue cracks may initiate.

The crack starts at the bottom of the vertical plate, grows upward in a U-shape and then propagates horizontally into the web. "Bleeding" of the crack indicates that there is relative movement of the crack surface, and moisture will combine with the oxide to streak down the surface. Severely skewed bridges with relatively heavy flanges should have the lower ends of diaphragm connection plates inspected frequently, if these connection plates are not attached to the bottom flange.

Ends of Diaphragm Connection Plates in Girder Bridges

Current design specifications and standards call for diaphragm connection plates to be positively attached to the girder flanges in order to resist the forces and deflections induced by the diaphragm members. If the attachment or detail condition is not adequate, fatigue cracks can develop at the end connection. One of these conditions is insufficient fillet weld between the end of a connection plate and the girder flange. This weld must be able to endure the lateral forces from the diaphragm components. If the fillet weld cracks, it will eventually sever the diaphragm connection plate from the flange. A horizontal fatigue crack can then develop in the web plate because of the out-of-plane distortion.

Sometimes, the diaphragm components are connected to gusset plates, which are welded to the vertical connection plates. The ends of the groove weld between the gusset plate and the connection plate have an abrupt change in plate geometry with re-entrant corners at the top of the connection plates. Fatigue cracks have developed in this region.

Unless these fatigue cracks are accompanied by movement and by oxide powder, their existence may not be obvious. Careful inspection from both sides of the diaphragm is necessary.

Lateral Gussets on Plate Girder Webs at Connections

Many fatigue cracks resulting from out-of-plane distortion of girder webs have been detected in web plates at the junction of lateral bracing gussets and diaphragm connection plates. The unequal lateral forces from the bracing members introduce lateral deflection and twisting of the junction in the direction perpendicular to the web. If the gusset plate is not attached to the vertical connection plate, the web plate in the small horizontal gap between the gusset plate and the connection plate is subjected to relative out-of-plane distortion and development of fatigue cracking. The vertical deflection of the lateral bracing causes stresses in the lateral bracing gusset plates. The welds connecting the lateral bracing gusset plates to the girder web may experience fatigue cracking.

8.2.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of steel superstructures. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component rating method and the AASHTO element level condition state assessment method.

NBI Rating Guidelines

Using the NBI rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBI Rating Guidelines.

The previous inspection data should be considered along with current inspection findings to determine the correct rating.

Element Level Condition State Assessment

In an element level condition state assessment of a steel girder bridge, the AASHTO CoRe element is:

Element No.	Description
106	Unpainted Steel Girder/beam
107	Painted Steel Girder/beam

The unit quantity for the girder/beam is meters or feet, and the total length must be distributed among the four available condition states for unpainted and five available condition states for painted structures depending on the extent and severity of deterioration. In both cases, Condition state 1 is the best possible rating. See the *AASHTO Guide for Commonly Recognized (CoRe) Structural Elements* for condition state descriptions.

A Smart Flag is used when a specific condition exists, which is not described in the CoRe element condition state. The severity of the damage is captured by coding the appropriate Smart Flag condition state. The Smart Flag quantities are measured as each, with only one each of any given Smart Flag per bridge.

For damage due to fatigue, the “Steel Fatigue” Smart Flag, Element No. 356, can be used and one of the three condition states assigned. For rusting between riveted members, the “Pack Rust” Smart Flag, Element No. 357, can be used and one of the four condition states assigned. For damage due to traffic impact, the “Traffic Impact” Smart Flag, Element No. 362, can be used and one of the three condition states assigned. For girders/beams with section loss due to corrosion, the “Section Loss” Smart Flag, Element No. 363, can be used and one of the four condition states assigned.

Table of Contents

Section 8

Inspection and Evaluation of Common Steel Superstructures

Topic 8.3 Steel Two Girder Systems

8.3.1 Introduction

The steel two girder bridge, like the fabricated multi-girder bridge, can use either riveted or welded construction. The difference is that it has only two girders. Two girder bridges can also have features similar to those of fabricated multi-girder bridges, such as web insert plates, transverse web stiffeners, and longitudinal web stiffeners (see Figure 8.3.1).

However, unlike the fabricated multi-girder bridge, the two girder bridge has a floor system of smaller stringers and floorbeams. The floor system supports the deck while the girders support the floor system.

Two girders can be found in simple span and continuous span configurations. They can also be found on curved bridges, and pin and hanger connections are common details with this bridge type. Two girder bridges are either deck girder or through girder systems.

In a deck girder system, the deck is supported by the floor system and top flanges of the two girders (see Figure 8.3.1). In a through girder system, the deck is supported by the floor system between the two girders (see Figure 8.3.2).

Figure 8.3.1 General View of a Dual Deck Girder Bridge

While few through girders are constructed today, they were commonly used prior to the early 1950's. Since many through girder bridges were constructed in the 1940’s and 1950’s, they are commonly riveted. Their most common use was where vertical under-clearance was a concern, such as over railroads (see Figure 8.3.3).

Figure 8.3.2 Through Girder Bridge

Figure 8.3.3 Through Girder Bridge with Limited Underclearance

A rare type of through girder has three or more girders, with the main girders actually separating the traffic lanes (see Figure 8.3.4). These structures are most likely converted railroad or trolley bridges.

Figure 8.3.4 Through Girder Bridge with Three Girders

8.3.2 Design Characteristics

Floor System Arrangement

Floor systems are similar in deck girder and through girder systems.

The floor system supports the deck. There are two types of floor systems found on two girder bridges:

- Girder-floorbeam system
- Girder-floorbeam-stringer system

The girder-floorbeam (GF) system consists of floorbeams connected to the main girders. The floorbeams are considerably smaller than the girders and are perpendicular to traffic. The deck is supported by the floorbeams, which in turn transmit the loads to the main girders. The floorbeams can be either rolled beams, fabricated girders, or fabricated cross frames (see Figure 8.3.5).

Figure 8.3.5 Two Girder Bridge with Girder-Floorbeam System

The girder-floorbeam-stringer (GFS) system consists of floorbeams connected to the main girders, and longitudinal stringers, parallel to the main girders, connected to the floorbeams (see Figure 8.3.6). The stringers may either connect to the web of the floorbeams or be stacked on top of the floorbeams, in which case they may be continuous stringers. Stringers are usually rolled beams and are considerably smaller than the floorbeams. It is also possible to find floorbeams that are stacked on top of the main girders, and the floorbeams may extend or overhang from the girders (see Figure 8.3.7).

Figure 8.3.6 Two Girder Bridge with Girder-Floorbeam-Stringer System

Figure 8.3.7 Two Girder Bridge with GFS System with Stacked Floorbeam and Stringers

Primary and Secondary Members

The primary members of a two girder bridge are the girders, floorbeams, and stringers, if present. The secondary members are diaphragms and the lateral bracing members, if present. These secondary members usually consist of angles or tee shapes placed diagonally in horizontal planes between the two main girders. The lateral bracing is generally in the plane of the bottom flange. Lateral bracing serves to minimize any differential longitudinal movement between the two girders (see Figure 8.3.8). Not all two girder bridges will have a lateral bracing system. Diaphragms, if present, are usually placed between stringers.

Figure 8.3.8 Underside View of Deck Girder Bridge with Lateral Bracing System

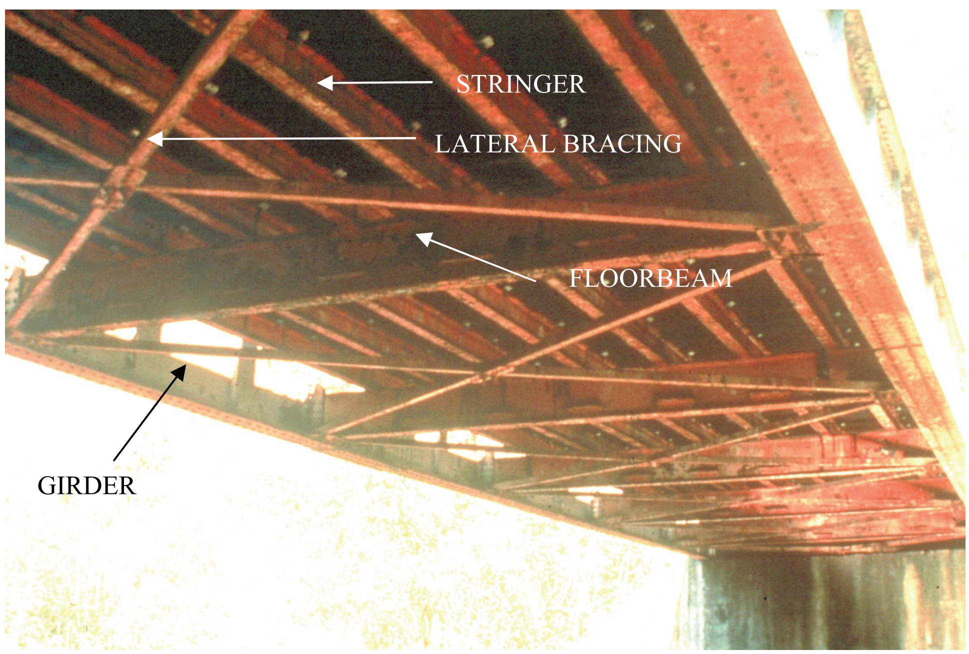

Figure 8.3.9 Underside View of Through Girder Bridge with Lateral Bracing

Fatigue Prone Details and Failure

Some common areas for fatigue prone details are:

- Fabrication welds
- Pin and hanger connections (if present)
- Welded cover plates
- Web stiffener welds

- Welded flange splices
- Intersecting welds
- Attachment welds located in the tension zone
- Web gaps
- Mechanical splices

Inspection of these areas is discussed further in Topic 8.3.4.

Fracture Critical Areas

Girders

Two girder bridges (deck girder and through girder) do not have load path redundancy. Both systems are therefore classified as fracture critical bridge types. The main girders are fracture critical members.

Pin and hanger assemblies in two girder bridges are fracture critical members (see Figure 8.3.10). Failure of one pin or one hanger will cause collapse of the suspended span since there is no alternate load path (e.g., Mianus River Bridge). Pins are considered "frozen" when corrosion restricts rotation. The pins and hangers experience additional bearing, torsion, bending and shear stresses when the pin and hanger assembly is frozen. This is a critical situation when it occurs on a (load path) nonredundant two girder bridge.

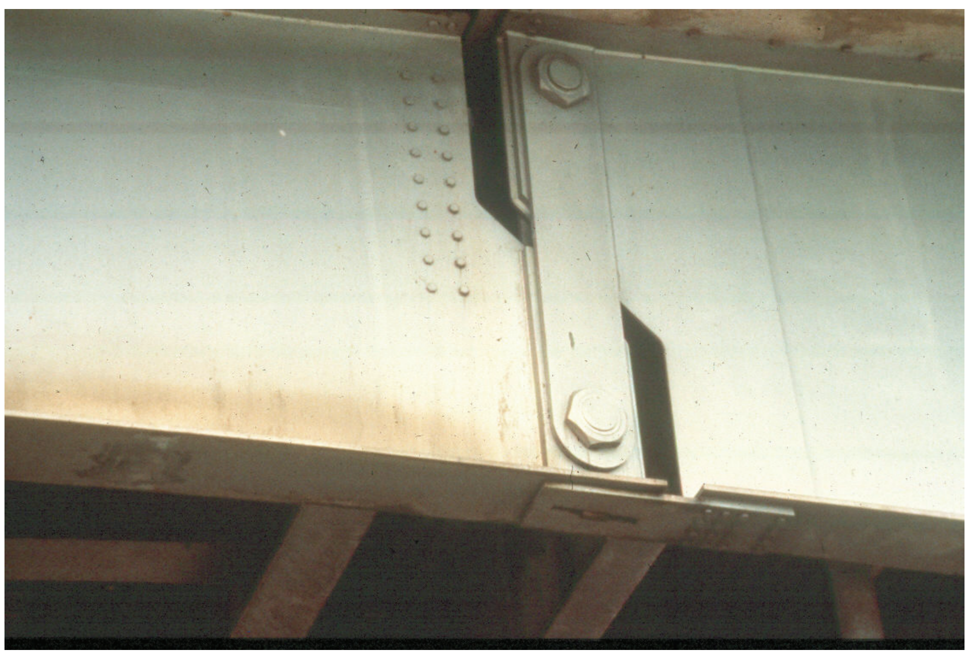

Figure 8.3.10 Two Girder Bridge with Pin and Hanger Connection

In the interest of conservatism, AASHTO chooses to neglect structural and internal redundancy and classify all two girder bridges as (load path) nonredundant.

Floorbeams

A floorbeam may be fracture critical if it satisfies one or more of the following conditions:

- Flexible or hinged connection to support at the girder/floorbeam connection
- Floorbeam spacing greater than 4 m (14'-0")
- No stringers supporting the deck
- Stringers are configured as simple beams

Several states consider floorbeams with spacing greater than 4 m (14'-0") to be fracture critical. A three dimensional finite element structural analysis may be performed to determine the exact consequences to the bridge if a floorbeam or floorbeam connection fails.

8.3.3 Overview of Common Defects

Common defects that occur on steel two girder and steel through girder bridges include:

- Paint failures
- Corrosion
- Fatigue cracking
- Collision damage
- Overloads
- Heat damage

See Topic 2.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 8.1 for Fatigue and Fracture in Steel Bridges.

8.3.4 Inspection Procedures and Locations

Inspection procedures to determine other causes of steel deterioration are discussed in detail in Topic 2.3.8.

Procedures

Visual

The inspection of steel bridge members for defects is primarily a visual activity.

Most defects in steel bridges are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is typically required. More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size of the suspected defect. Care should be taken in cleaning when the suspected defect is a crack. When cleaning steel surfaces, any type of cleaning process that would tend to close discontinuities, such as blasting, should be avoided. The use of degreasing spray before and after removal of the paint may help in revealing the defect.

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, the inspector should examine all other similar locations and details.

Advanced Inspection Techniques

Several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Acoustic emissions testing
- Computer tomography
- Corrosion sensors
- Smart paint 1
- Smart paint 2
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 13.3.3, include:

- Brinell hardness test
- Charpy impact test
- Chemical analysis
- Tensile strength test

Locations

Bearing Areas

Examine the web areas over the supports for cracks, section loss and buckling. If bearing stiffeners, jacking stiffeners and diaphragms are present at the supports inspect them for cracks, section loss and buckling also.

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. See Topic 9.1 for a detailed presentation on the inspection of bearings

Shear Zones

Examine the web areas of the girders, floorbeams, and stringers near their supports for section loss or buckling (see Figures 8.3.11 and 8.3.12). This is a critical area, especially if the web is coped or the flange is blocked.

Figure 8.3.11 Shear Zone on a Deck Girder Bridge

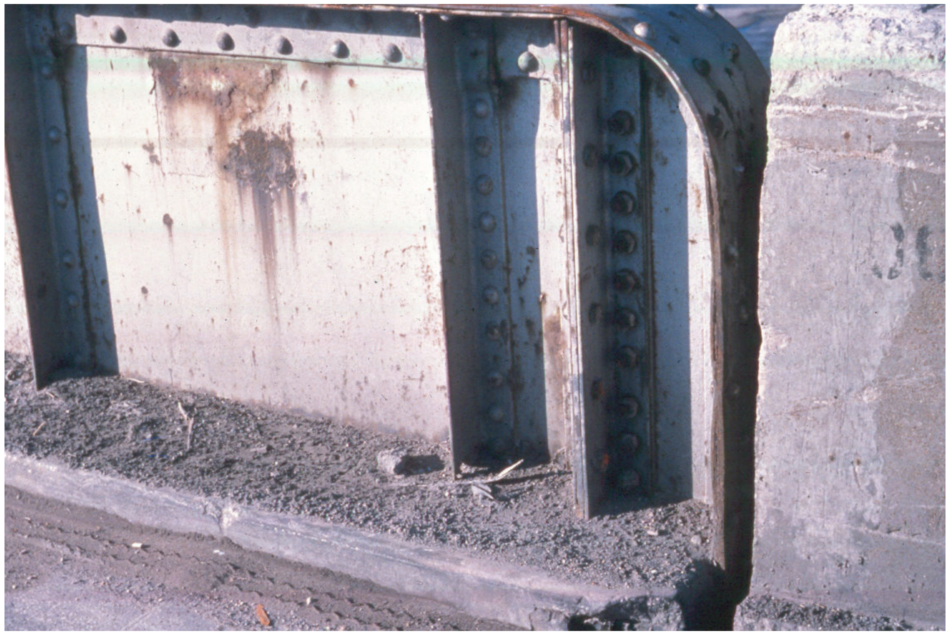

Figure 8.3.12 Web Area Near Support on a Through Girder Bridge

Flexure Zones

The flexure zone of each girder includes the entire length between the supports (see Figures 8.3.13 and 8.3.15). Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, and gouges. Check the flanges in high stress areas for bending or flexure- related damage. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the beams over the intermediate supports have high flexural stresses due to negative moment. Check flange splice welds and longitudinal stiffener splice welds in tension areas (see Figure 8.3.14).

Figure 8.3.13 Flexural Zone on a Two Girder Bridge

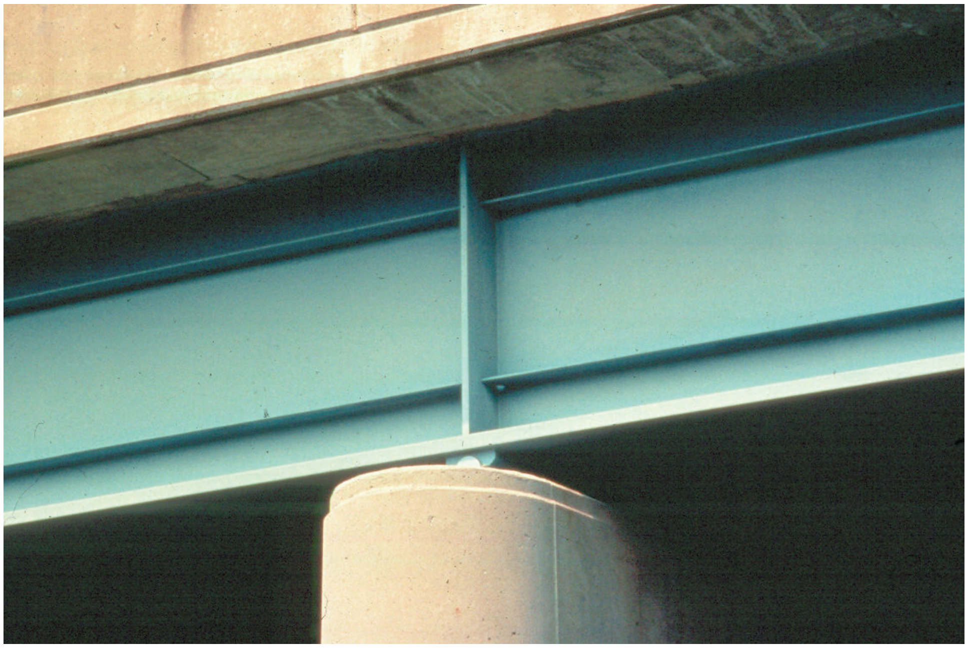

Figure 8.3.14 Longitudinal Stiffener in Tension Zone on a Two Girder Bridge

Figure 8.3.15 Flexural Zone on a Through Girder Bridge

Secondary Members

Investigate the diaphragms, if present, and the connection areas of the lateral bracing for cracked welds, fatigue cracks, and loose fasteners. Inspect the bracing members for any distortion or corrosion (see Figures 8.3.16 and 8.3.17). Distorted or cracked secondary members may be an indication the primary members may be overstressed or the substructure may be experiencing differential settlement.

Figure 8.3.16 Lateral Bracing Connection on a Deck Girder Bridge

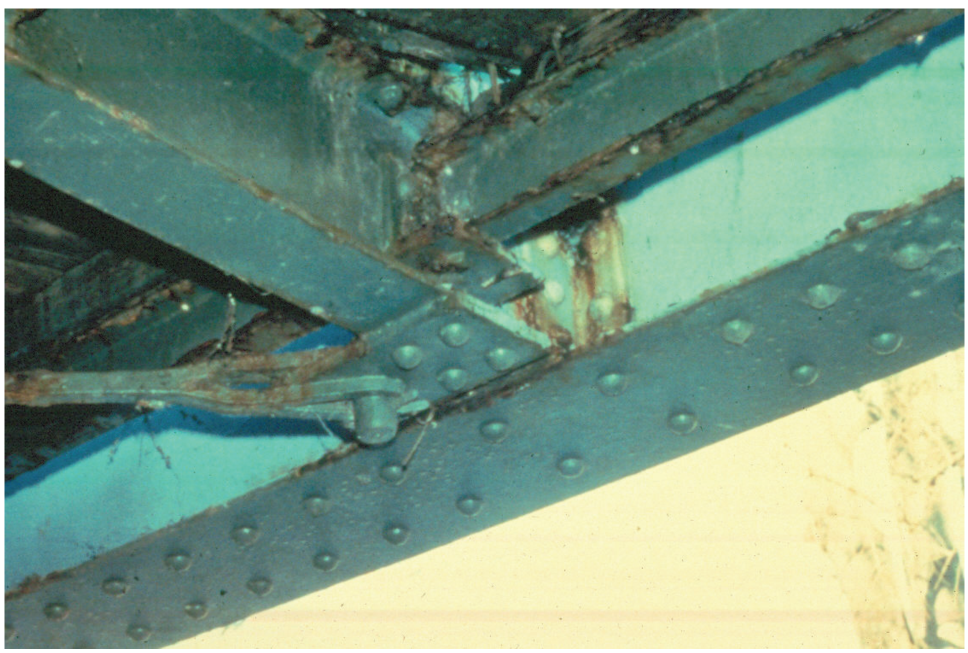

Figure 8.3.17 Lateral Bracing Connection on a Through Girder Bridge

Areas That Trap Water and Debris

Check horizontal surfaces that can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Areas that trap water and debris can result in active corrosion cells and excessive loss in section. This can result in notches susceptible to fatigue or perforation and loss of section.

On two girder bridges check:

- Along the bottom flanges of the girders
- Pockets created by girder-floorbeams and floorbeam-stringer connections
- Lateral bracing gusset plates
- Areas exposed to drainage runoff
- Along the girder webs at the curb line (through girder system)

Areas Exposed to Traffic

Check underneath the bridge for collision damage to the main girders and bracing if the bridge crosses over a highway, railway, or navigable channel. Document any cracks, section loss, or distortion found (see Figures 8.3.18 and 8.3.19). On a through girder bridge, investigate the main girders along the curb lines and at the ends for collision damage.

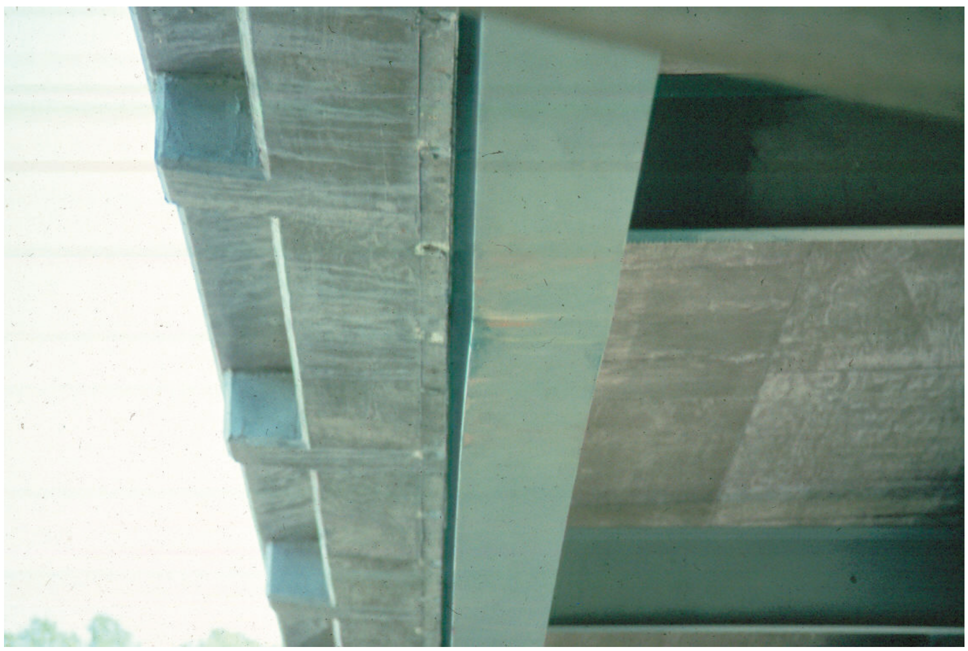

Figure 8.3.18 Collision Damage to a Two Girder Bridge

Figure 8.3.19 Collision Damage to a Through Girder Bridge

Fatigue Prone Details

Dirt and debris traps can result in active corrosion cells when water and salt are present. These corrosion cells can lead to excessive section loss. This corrosion can result in notches that are susceptible to fatigue or perforation.

Check web stiffener welds, welded web/flange splices, and intersecting welds (see Figures 8.3.20 and 8.3.21). Also inspect any attachment welds located in the tension zone of the girder and floorbeam bracket tie plate (see Figure 8.3.22), especially unplanned miscellaneous attachment welds, such as utility brackets.

If the structure has been painted, breaks in the paint accompanied by rust staining indicate the possible existence of a fatigue crack. Investigate the areas surrounding field splice cover plates on the tension flange. The suspected crack area should be cleaned to determine the existence of a crack and its extent. If a crack with rust staining exists in the paint, the fatigue cracks in the steel can already be up to 6 mm (1/4 inch) deep in the beam flange. Check any attachment welds located in the superstructure tension zones, such as traffic safety features, lighting brackets, utility attachments, catwalks and signs. Welds are considered to be intersecting if they are within 6 mm (1/4 inch) from each other (see Figure 8.3.21).

Figure 8.3.20 Web Stiffeners and Welded Flange Splice

Figure 8.3.21 Intersecting Welds

Check for fatigue cracks due to web-gap distortion. This is the major source of cracking in steel bridges.

If the girder or floorbeam is riveted or bolted, check all rivets and bolts to determine that they are tight and in good condition. Check for cracked or missing bolts, rivets and rivet heads. Check the base metal around the bolts and rivets for any signs of cracking.

Inspect the member for misplaced holes or repaired holes that have been filled with weld material. Check for plug welds which are possible sources of fatigue cracking.

Fracture Critical Members

Since two girder bridges have no load path redundancy and are fracture critical, it is important to inspect the main girders thoroughly. Floorbeams may also be fracture critical if they meet the requirements specified in Topic 8.3.2. Any defects such as cracks, section loss and out-of plane distortions should be measured and documented. All previous reports should be reviewed before performing the inspection to note any areas of particular concern. All reported deficiencies should be checked to ensure no further development has occurred.

Out-of-plane Distortion

Out-of-plane distortion can occur in several areas that can lead to web cracks near the flanges of steel bridges. The following are some common areas for out-of-plane distortion.

Girder Webs at Floorbeam Connections

Floorbeams between bridge girders exert out-of-plane forces to the girder webs through the vertical connection plates. The connection plates are usually sufficient to transmit the forces but the structural details at the ends of the connection plates sometimes are inadequate to accommodate the deflections and rotations.

Sometimes, floorbeam support brackets are welded to the tension flange of the girder (see Figure 8.3.22).

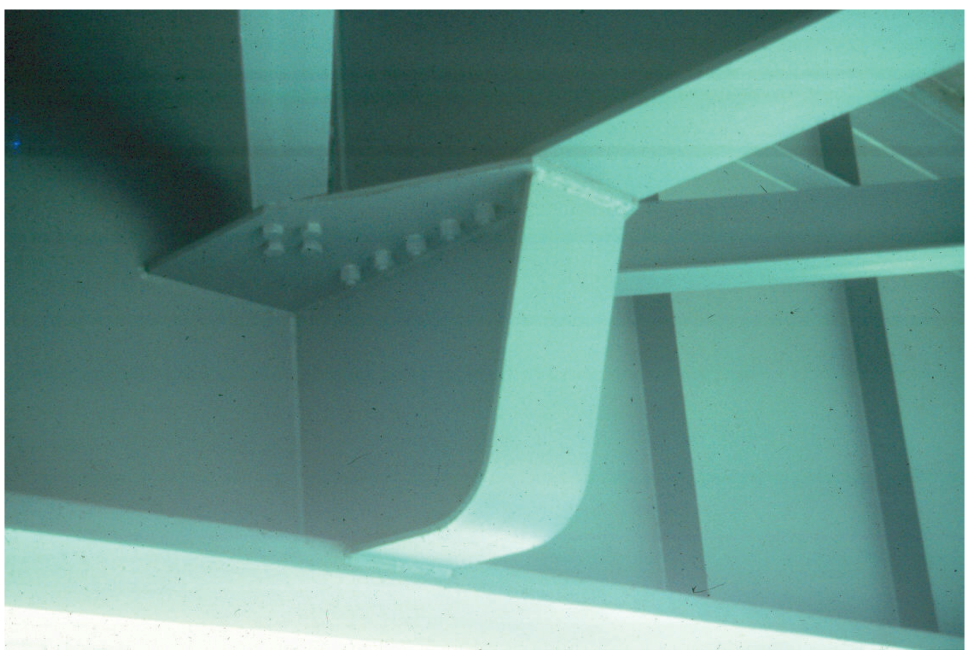

Figure 8.3.22 Floorbeam to Girder Connection

One type of connection detail that has incurred a large number of fatigue cracks is the end of floorbeam connection plates that are not attached to the top tension flange of continuous girder bridges. While the top flange is rigidly embedded in the bridge deck slab, and the connection plate itself is stiff enough to resist rotation and bending from the floorbeam, most of the out-of-plane distortions (perpendicular to the web) concentrate in the local region of the web above the upper end of the connection plate. Fatigue cracks develop in the region as a result of the web plate bending. The cracks are usually horizontal along the web-to-flange welds, and also propagate as an upside-down U along the upper ends of the fillet welds of the connection plate (see Figure 8.3.23). Movement at or near such small cracks often generates oxide powder that combines with moisture to cause apparent bleeding.

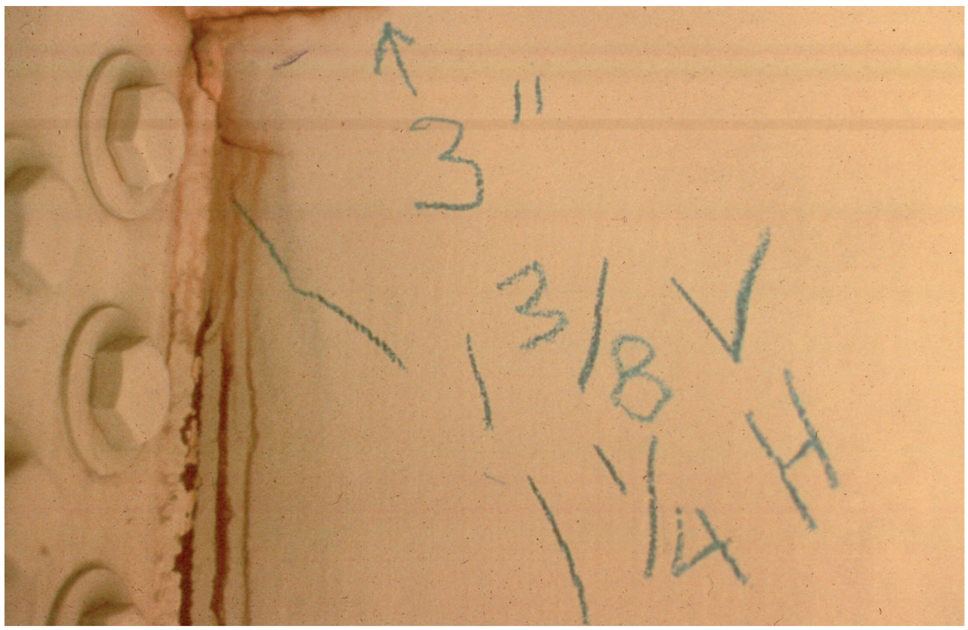

Figure 8.3.23 Crack Caused by Out-of-plane Distortion

Detection of cracks of fairly significant length is not difficult. Knowing that unattached ends of floor beam connection plates are likely locations of fatigue cracks increases the certainty of early detection of these cracks.

At the lower end of floorbeam connection plates that are not welded to the tension flange of girders, the condition of local out-of-plane distortion and bending of the web plate usually is less severe. This is because the tension flange is not restrained from lateral movement, which is sufficient to reduce the web plate bending. However, if the bottom flange is restrained from lateral deflection, fatigue cracks will develop along the web to flange weld.

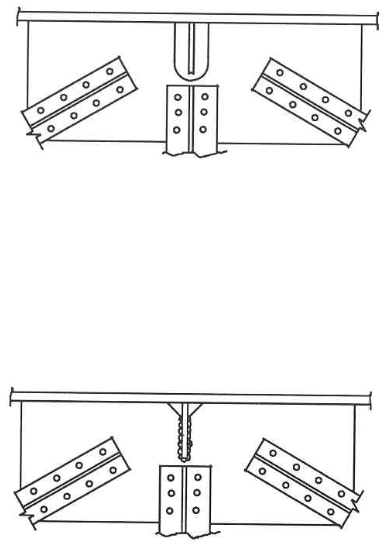

Figure 8.3.24 Lateral Bracing Gusset at Floorbeam or Diaphragm Connection Plate

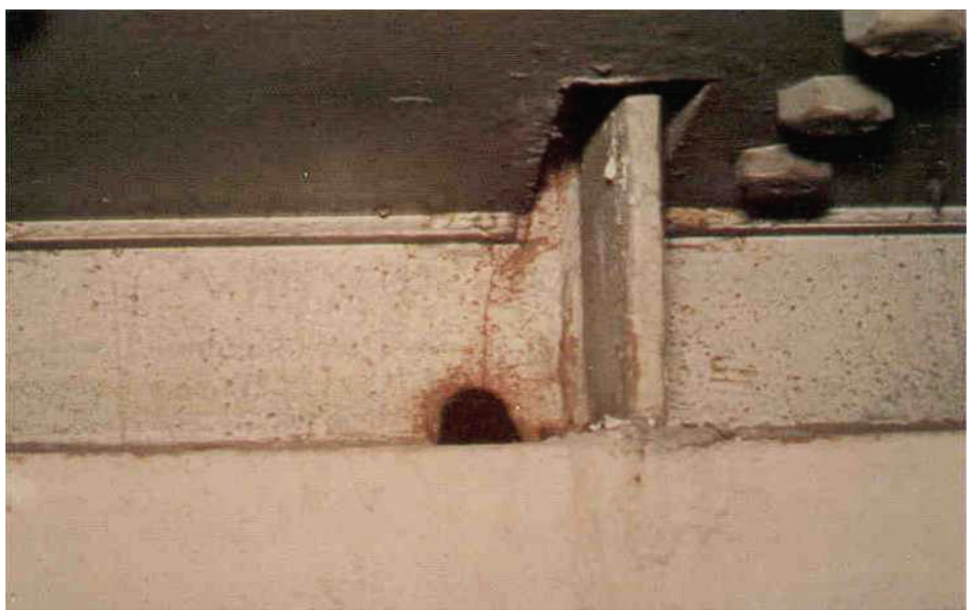

Figure 8.3.25 Crack Caused by Out-of-plane Distortion

Lateral Gussets on Plate Girder Webs at Floorbeam Connection

The above figures show examples of potential for lateral gusset plate out-of-plane distortion problems. Vertical deflection of the lateral bracing causes stresses in the lateral bracing gusset plates. The welds connecting the lateral bracing gusset plate to the girder web may experience fatigue cracking. In addition to possible cracks at the internal gap, the ends of the gusset fillet weld should be equally suspect.

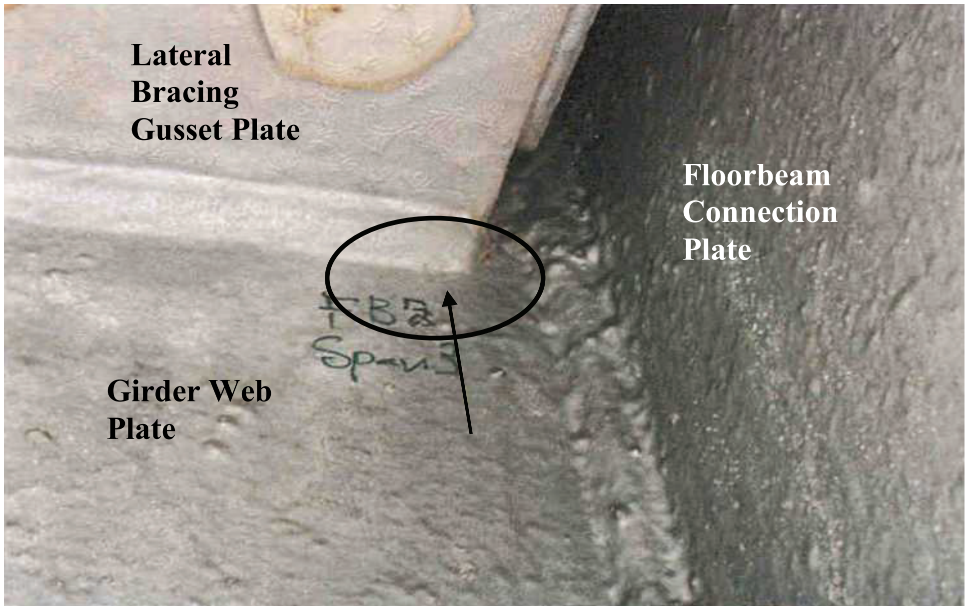

Figure 8.3.26 Crack Caused by Out-of-plane Distortion

Figure 8.3.26 shows another example of a crack in the gap between the lateral bracing gusset and the floorbeam connection plate. The crack was very small when detected and the photograph was taken. However, on the opposite side of the web plate, at the elevation of the gusset, a crack more than an inch long was detected along the weld toe of the vertical fillet weld that joins the web and the fascia transverse stiffener in alignment with the floorbeam. This situation of staggered cracks on opposite surfaces of a web plate in a small gap is typical of out-of-plane distortion induced cracks at lateral gusset to floorbeam connection details.

Fatigue cracks may also develop at the weld toe on the web surface at the far ends of a horizontal gusset attached to the web for lateral bracing members (the ends away from the floorbeam connection plate). With the out-of-plane distortion and twisting of the junction, the web is subjected to plate bending stresses that add to the primary stresses in the girder web (see Figure 8.3.27).

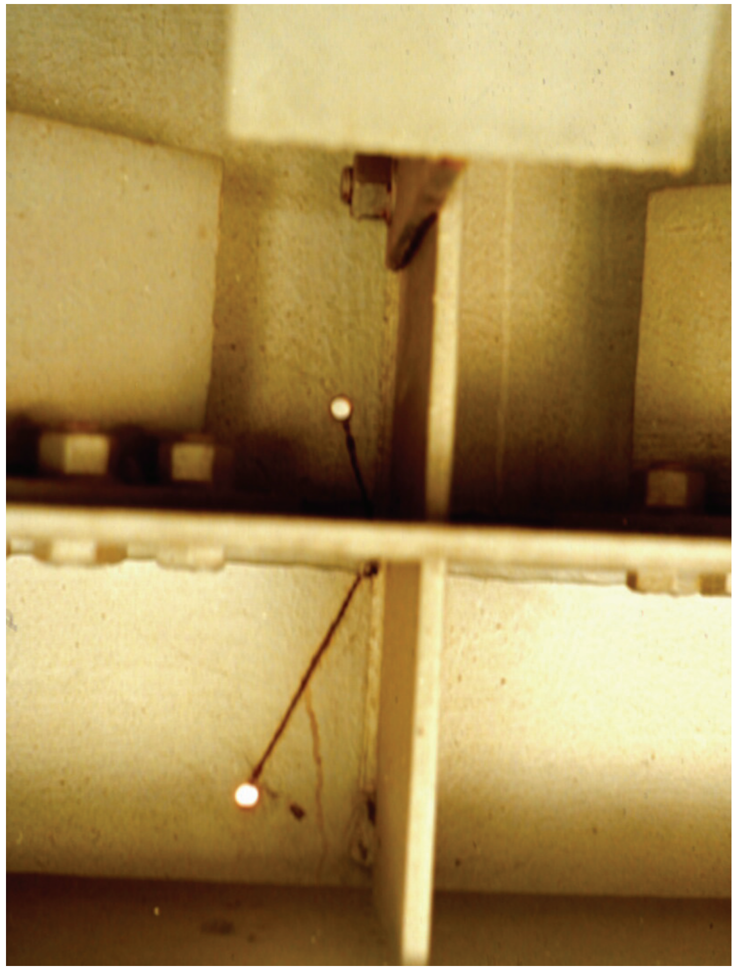

Figure 8.3.27 Cracking in Girder Web at the Intersection of Horizontal Gusset Plate for Lateral Bracing and Transverse Stiffener

Floorbeam and Cantilever Bracket Connection to Girders

In order to increase deck width, floorbeams are often cantilevered past the main longitudinal girders. The floorbeams may be stacked on top of the girders or framed into the girders.

The floorbeam may be connected to the girder web (see Figure 8.3.28). Inspect for cracks in the floorbeam and girder. A tie plate may be utilized to reduce the fatigue stresses in the floorbeam/girder connection (see Figure 8.3.29). Carefully inspect the tie plate for fatigue cracking.

Figure 8.3.28 Cracked Cantilever Floorbeam

In bridges with deep girders and floorbeams, such cracks have also been detected in small gaps at boundaries of floorbeam access holes at catwalks and at ends of stiffeners on web plate which stiffen the web plate and concentrate the out-of-plane distortion in the small gaps.

Figure 8.3.29 Tie Plate for Cantilever Floorbeam

8.3.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of steel superstructures. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component rating method and the AASHTO element level condition state assessment method.

NBI Rating Guidelines

Using the NBI rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBI Rating Guidelines.

The previous inspection data should be used along with current inspection findings to determine the correct rating.

Element Level Condition State Assessment

In an element level condition state assessment of a steel two girder system, the AASHTO CoRe element is:

Element No.	Description
106	Unpainted Steel Girder/beam
107	Painted Steel Girder/beam
112	Unpainted Steel Stringer
113	Painted Steel Stringer
151	Unpainted Steel Floorbeam
152	Painted Steel Floorbeam
160	Unpainted Steel Pin and/or Pin & Hanger Assembly
161	Painted Steel Pin and/or Pin & Hanger Assembly

The unit quantity for the girder is meters or feet, and the total length must be distributed among the four available condition states for unpainted and five available condition states for painted structures depending on the extent and severity of deterioration. In both cases, Condition state 1 is the best possible rating. See the *AASHTO Guide for Commonly Recognized (CoRe) Structural Elements* for condition state descriptions. For pin and hanger assemblies, see Topic 8.4.

A Smart Flag is used when a specific condition exists, which is not described in the CoRe element condition state. The severity of the damage is captured by coding the appropriate Smart Flag condition state. The Smart Flag quantities are measured as each, with only one each of any given Smart Flag per bridge.

For damage to fatigue, the "Steel Fatigue" Smart Flag, Element No. 356, can be used and one of the three condition states assigned. For rust, the "Pack Rust" Smart Flag, Element No. 357, can be used and one of the four condition states assigned. For damage due to traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned. For girders with section loss, the "Section Loss" Smart Flag, Element No. 363, can be used and one of the four condition states assigned.

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Section 8
Inspection and Evaluation of Common Steel Superstructures

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Topic 8.4 Pin and Hanger Assemblies

8.4.1 Introduction

Pin and hanger assemblies are devices put in bridges to permit expansion movement and rotation (see Figure 8.4.1). If only rotation of the joint is desired, one pin is used (see Figure 8.4.2). When expansion (longitudinal) movement is also required, a system consisting of two pins with hanger links between them is used.

Figure 8.4.1 Typical Pin and Hanger Assembly

Pin and hanger joints are usually found in multi-span bridges designed prior to 1970. Incorporating a hinge in a structure simplifies analysis. It also moves expansion joints (and drainage related damage) away from the abutments and piers (see Figure 8.4.3).

Modern design techniques and computer programs enable the engineer to design multi-span bridges without hinges. The problems associated with pin and hanger details far outweigh any advantages of placing expansion joints away from substructure units.

Although pin and hanger designs are no longer used, many bridges with these assemblies are still in service and will remain for the foreseeable future. Therefore, it is very important to pay special attention to these details during inspection.

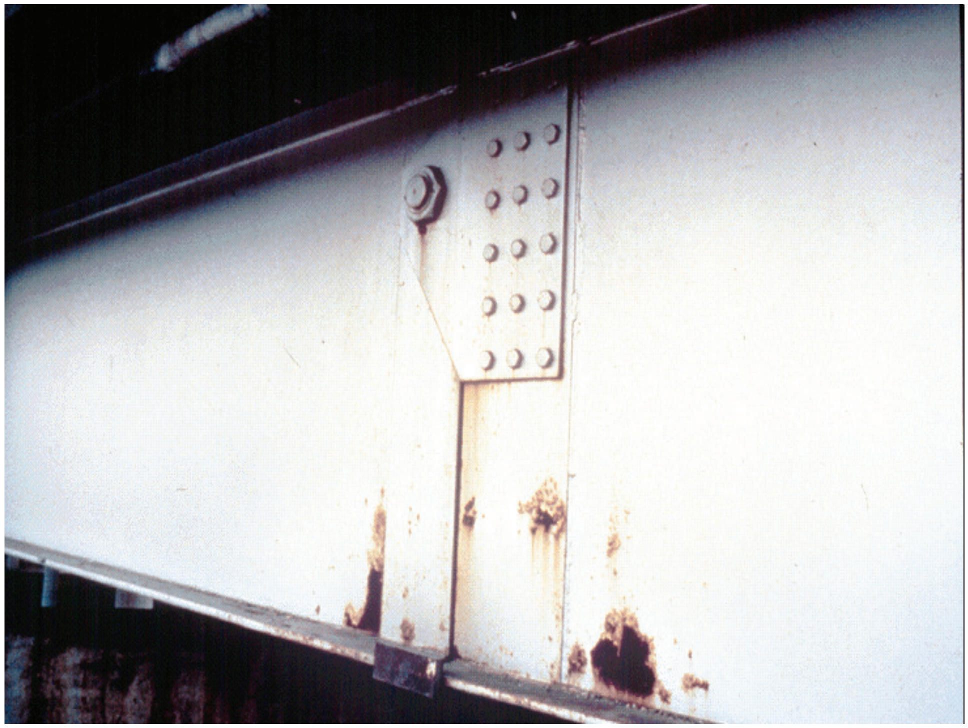

Figure 8.4.2 Single Pin with Riveted Pin Plate

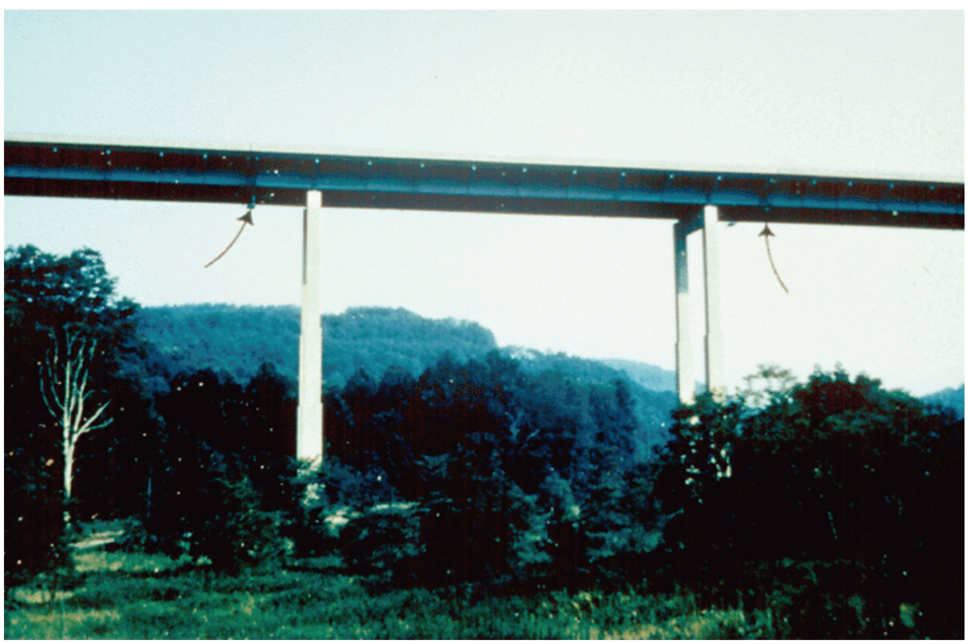

Figure 8.4.3 Pin and Hanger Assembly Locations Relative to Piers

8.4.2 Design Characteristics

Primary and Secondary Members

There are many different components to a pin and hanger assembly as Figure 8.4.4 demonstrates.

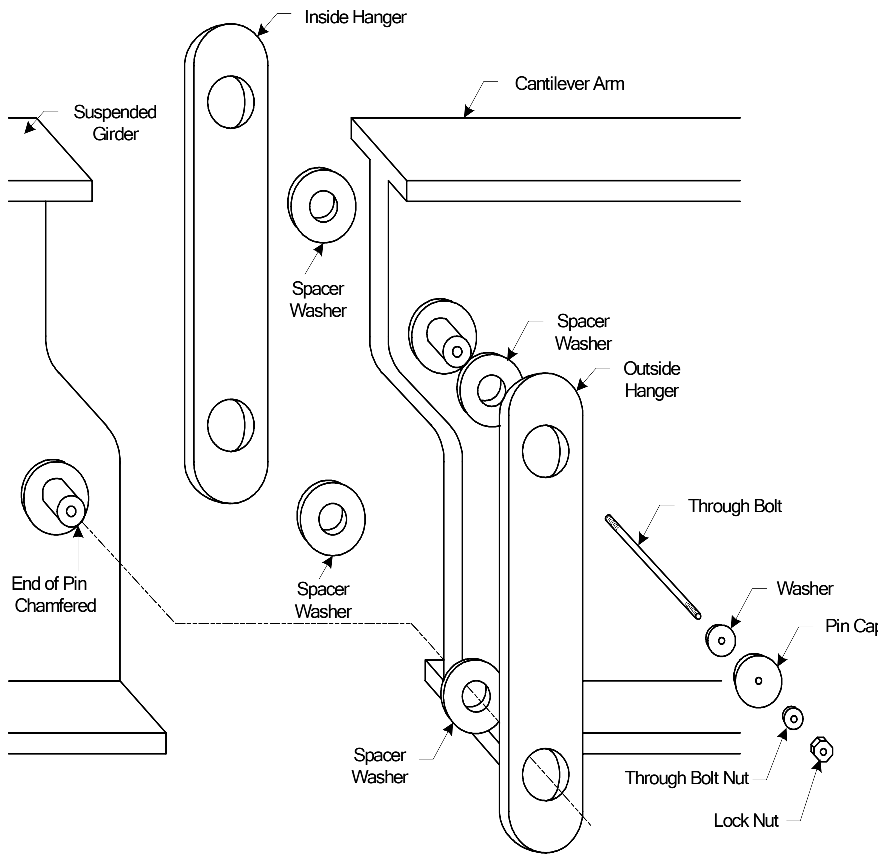

Figure 8.4.4 Pin and Hanger Assembly

The primary members of a pin and hanger assembly are the pin and the hanger link. The pin may be drilled to accept a through-bolt (see Figure 8.4.5) or threaded to accept a large nut (see Figure 8.4.6). Threaded pins are often stepped (or shouldered) to accept a small diameter nut. The hanger link may be a plain flat plate with two holes or an eyebar shaped plate (see Figure 8.4.7).

The secondary members of a pin and hanger assembly include through-bolts and the pin cap (see Figure 8.4.8), nuts (see Figure 8.4.9), cotter pins on small assemblies with pins less than 100 mm (4 inches) in diameter, spacer washers and doubler plates which reinforce the beam web around the pin hole (see Figure 8.4.10).

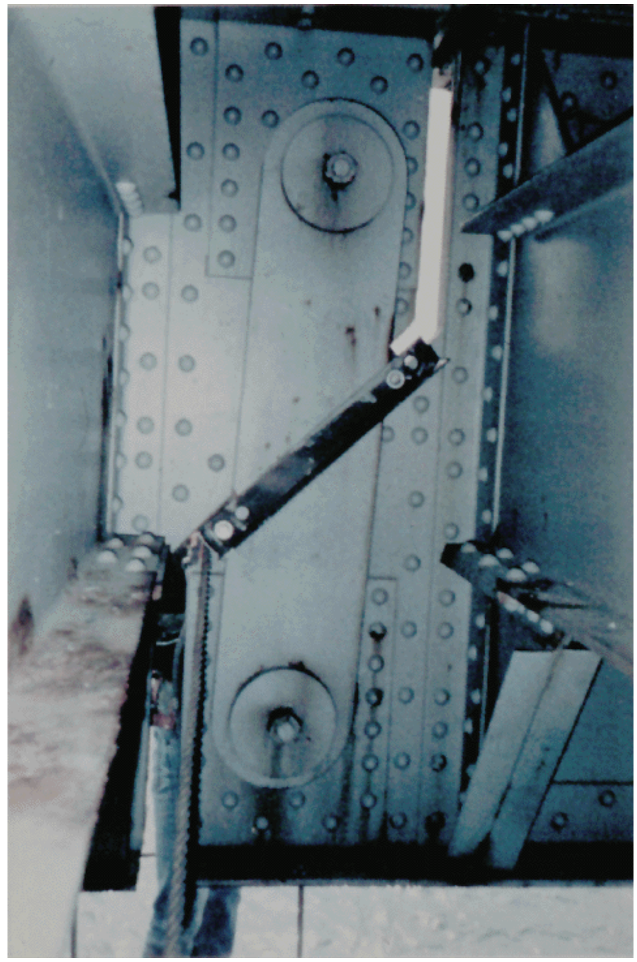

Figure 8.4.5 Pin Cap with Through Bolt

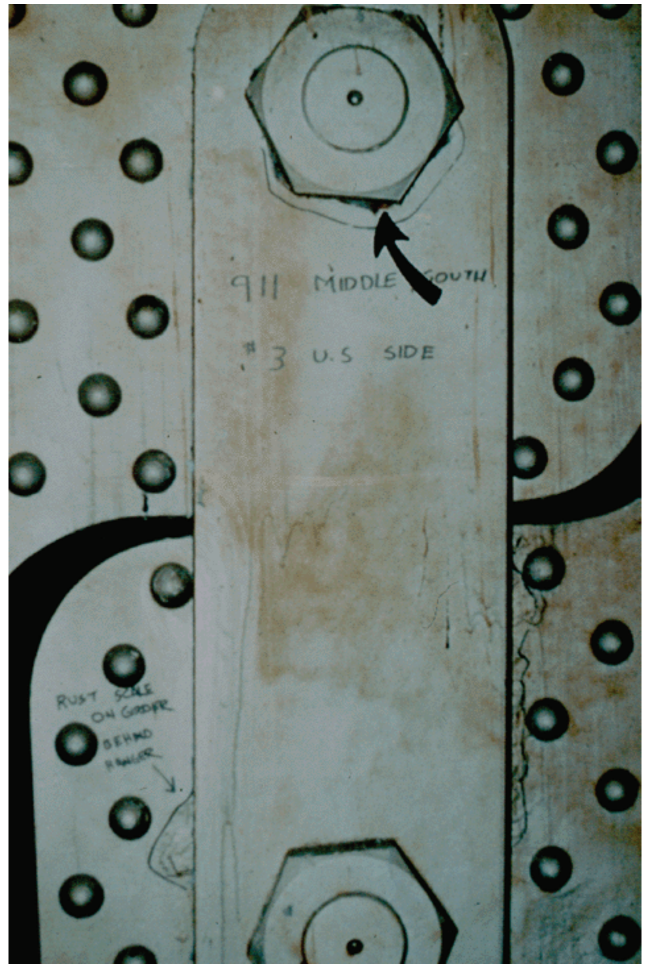

Figure 8.4.6 Threaded Pin with Retaining Nut

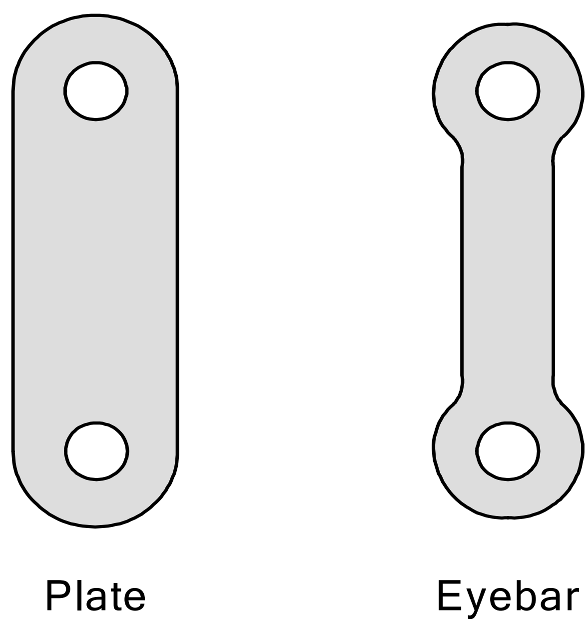

Figure 8.4.7 Plate Hanger and Eyebar Shape Hanger Link

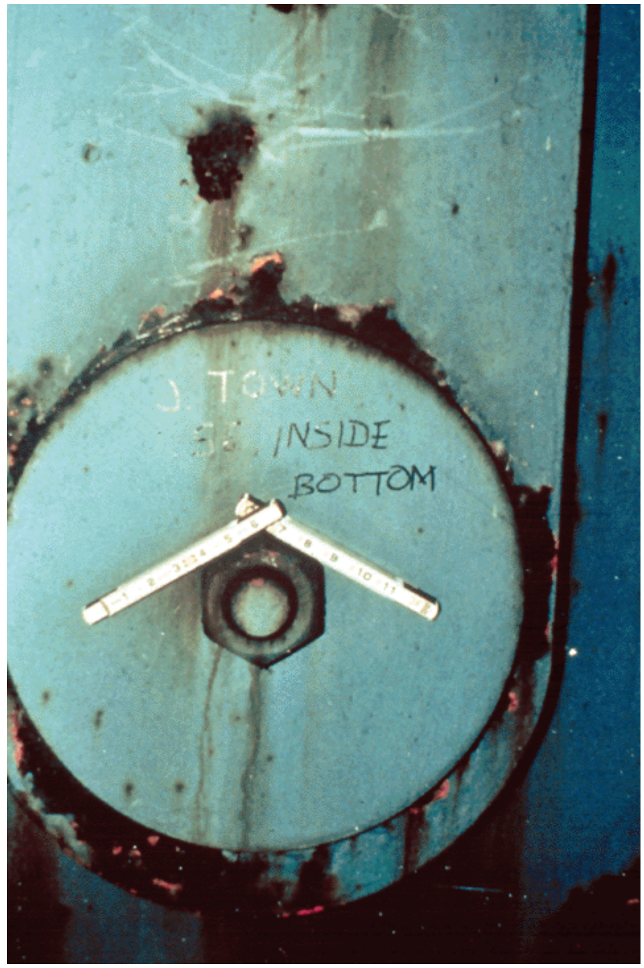

Figure 8.4.8 Pin Cap, Through Bolt and Nut

Figure 8.4.9 Retaining Nut

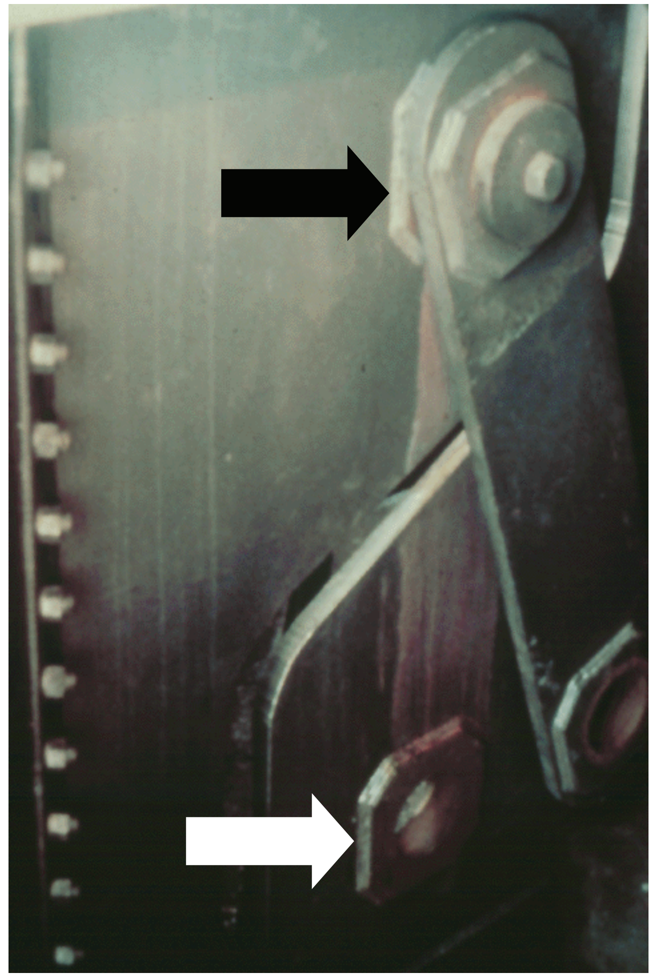

Figure 8.4.10 Web Doubler Plates

Forces in a Pin – Design vs. Actual

Some of the problems with the pin and hanger assembly can be attributed to deficiencies that cause forces that were not accounted for in the design. The hanger or links are designed for pure tension forces only (see Figure 8.4.11). However, in actuality, hangers see both pure tension and bending. In-plane bending results from binding on the pins due to corrosion between the pin and the hanger (see Figure 8.4.12). Out-of-plane bending (perpendicular to the wide face) results from misalignment, pack rust or skewed geometry.

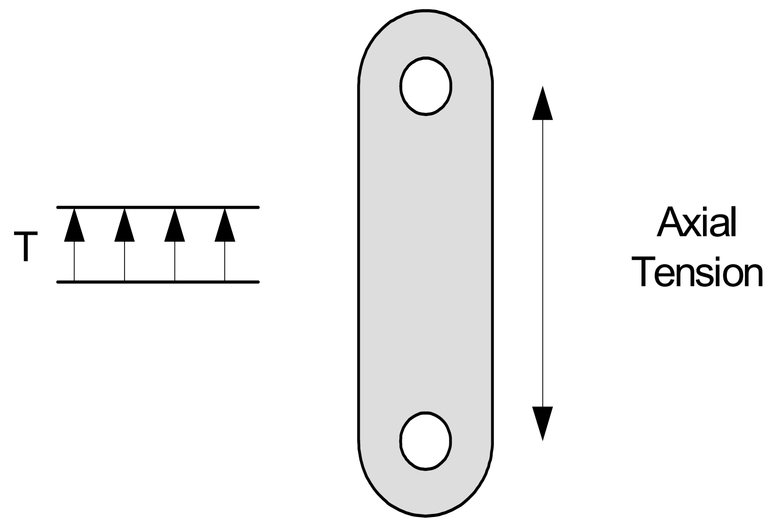

Figure 8.4.11 Design Stress in a Hanger Link(Tension Only)

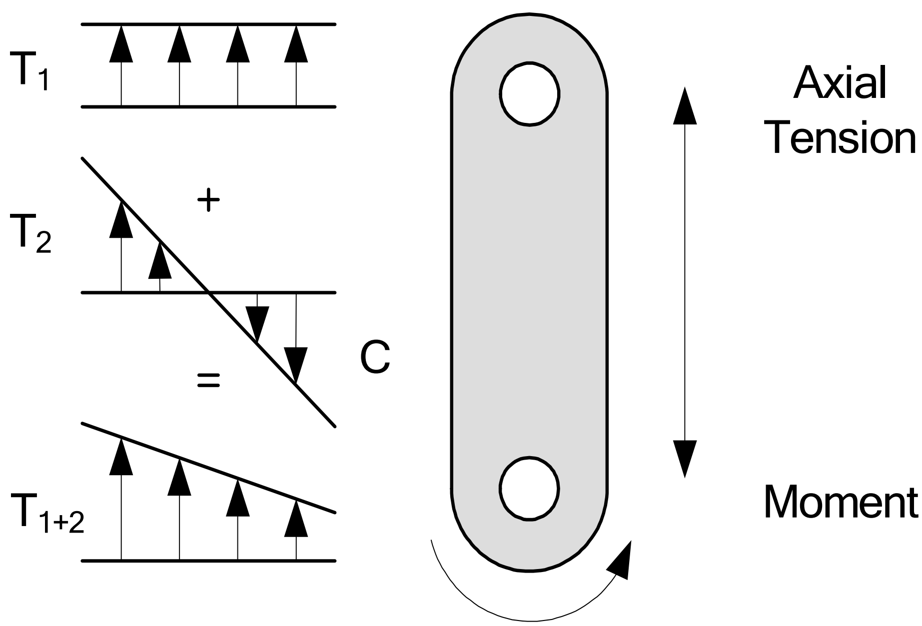

Figure 8.4.12 Actual Stress in a Hanger Link (Tension and Bending)

Pins are designed to resist shear and bearing on the full thickness of the hanger (see Figure 8.4.13). However, in addition to the designed forces, pins can see very high torsion (twisting) forces if they lose their ability to turn freely (see Figure 8.4.14). Corrosion and rust packing can inhibit or prevent the pins from turning properly. Pins can also be subjected to excessive bearing stress if the hanger shifts over the pin shoulder (see Figure 8.4.14).

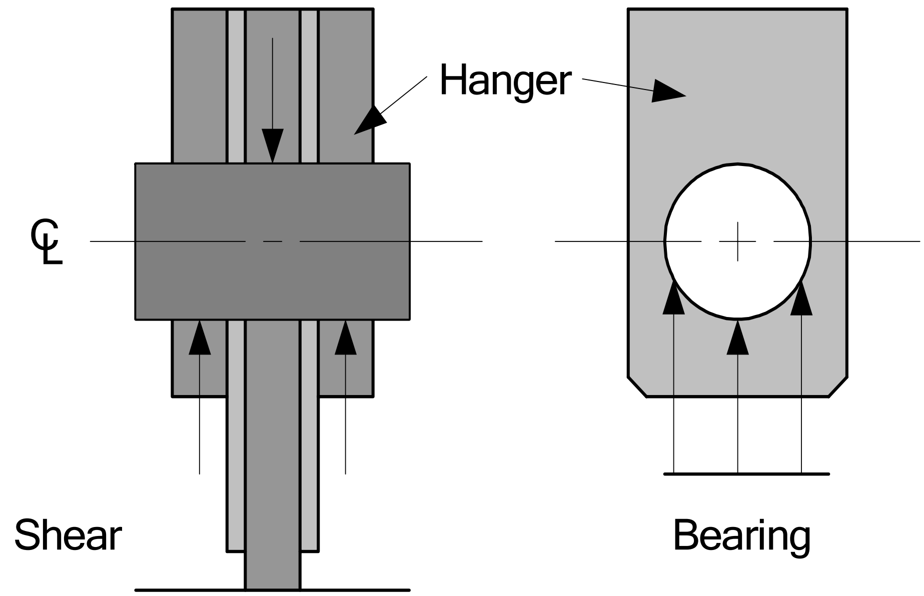

Figure 8.4.13 Design Stress in a Pin (Shear and Bearing)

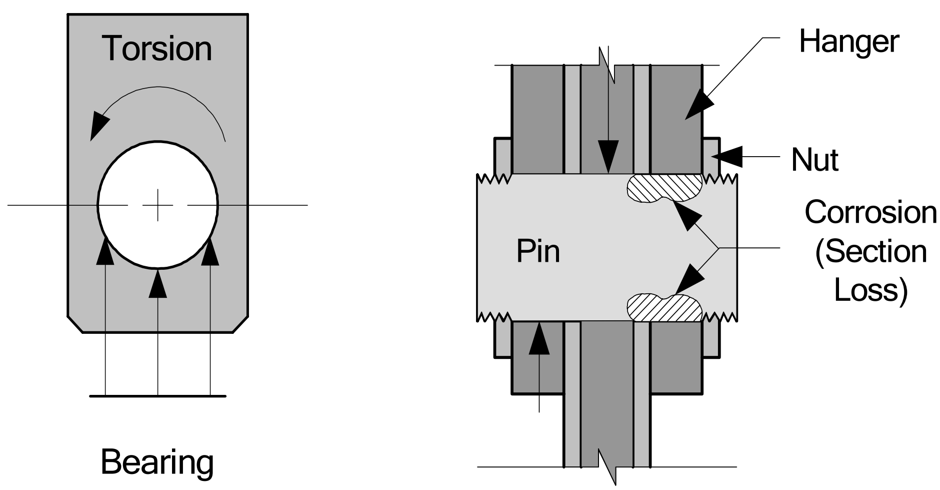

Figure 8.4.14 Actual Stress in a Pin (Shear, Bearing and Torsion)

Fracture Critical Pin and Hanger Assemblies

AASHTO "Manual for Condition Evaluation of Bridges", Section 3.12 calls for special attention during the inspection of pin and hanger connections on two or three girder systems. Failure of one pin or one hanger will cause collapse of the suspended span since there is no alternate load path. The collapse can be catastrophic as demonstrated by the Mianus River Bridge failure shown in Figure 8.4.15. The Mianus River Bridge failed due to the formation of rust between the hangers and the girder webs. As steel rusts, the rust can occupy up to 10 times the original steel volume causing unwanted expansion forces when in a confined space. When rust creates this type of expansion force, it is called "rust packing". In the case of the Mianus River Bridge, the rust packing pushed the hangers to the ends of the deteriorated pins and the pins eventually failed in bearing.

Figure 8.4.15 Mianus River Bridge Failure

Pin and hanger assemblies in multi-girder structures are not technically fracture critical, since multiple load paths are available. However, they do have the potential for progressive collapse. If all the pin and hanger assemblies at a joint location are frozen and consequently overstressed, the failure of one could cause an adjacent assembly to fail and so on (see Figure 8.4.16).

Figure 8.4.16 Multi-girder Bridge with Pin and Hanger Assemblies

8.4.3 Overview of Common Defects

Common defects that occur on steel pin and hanger bridge assemblies include:

- Paint failures
- Corrosion
- Fatigue cracking
- Collision damage
- Overloads
- Heat damage

See Topic 2.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 8.1 for Fatigue and Fracture in Steel Bridges

8.4.4 Inspection Procedures and Locations

Inspection procedures to determine other causes of steel deterioration are discussed in detail in Topic 2.3.8.

Procedures

Visual

The inspection of steel bridge members for defects is primarily a visual activity.

Most defects in steel bridges are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is required. More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size and location of the suspected defect. The use of degreasing spray before and after removal of the paint may help in revealing the defect.

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, the inspector should examine all other similar locations and details.

Advanced Inspection Techniques

Several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Acoustic emissions testing
- Computer programs
- Computer tomography
- Corrosion sensors

- Smart paint 1
- Smart paint 2
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 13.3.3, include:

- Brinell hardness test
- Charpy impact test
- Chemical analysis
- Tensile strength test

Visual inspection of the pin may not be very effective. The majority of the pin is concealed inside the assembly and at best only the surface is available for inspection. Many internal flaws and defects can go undetected if an advanced inspection technique such as ultrasonic testing is not used.

Ultrasonic testing is currently the most common means available of checking pins in place (see Figure 8.4.17). For the results to be valid, careful planning and testing by trained individuals is required. For a more detailed look at ultrasonic testing refer to Topic 13.3.

Figure 8.4.17 Ultrasonic Testing of a Pin

Another method for inspecting the pin is to disassemble the pin and hanger unit. Disassembly of a pin and hanger joint should be undertaken only after proper engineering design is performed and auxiliary support supplied. It is not a routine bridge inspection procedure (see Figure 8.4.18).

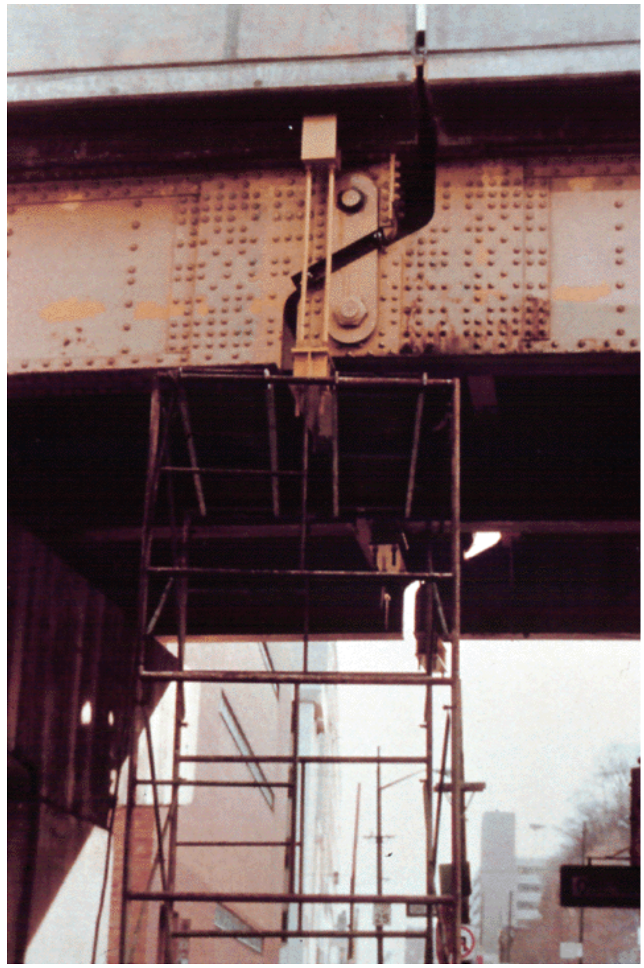

Figure 8.4.18 Alternate Hanger Link Retaining System

Hanger links and pins are often difficult to remove even after the retaining assemblies are taken off. This is not always true, however, and a pin on the verge of failure due to rust pack could fail suddenly when the nut is loosened.

Locations

General

Observe and record the general condition of the pin and hanger assembly. Check for alignment of the adjacent beam webs and flanges with a straight edge. If present, inspect the wind lock for signs of excessive transverse movement. A wind lock consists of steel or neoprene members attached to both the suspended and cantilever bottom flanges. Note if deck drainage is entering the assembly.

Measure the actual dimensions between the pins and also the distance from each pin to the end of the hanger assembly and compare these values to the as-built dimensions (see Figure 8.4.19).

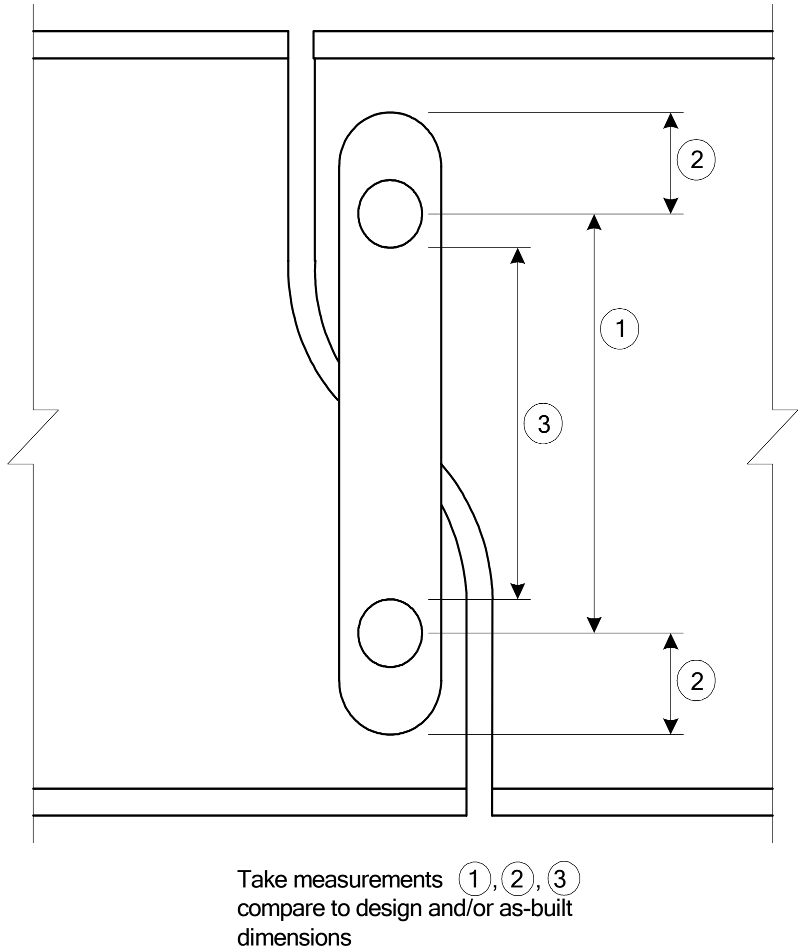

Figure 8.4.19 Pin Measurement Locations

Try to determine if movement is taking place. Corrosion can cause fixity at pin and hanger connections. This changes the structural behavior of the connection and is a source of cracking. Powdery red or black rust where surfaces rub indicates movement (see Figure 8.4.20). It may or may not indicate appreciable section loss. An unbroken paint film across a surface where relative movement should be taking place indicates the pin is frozen.

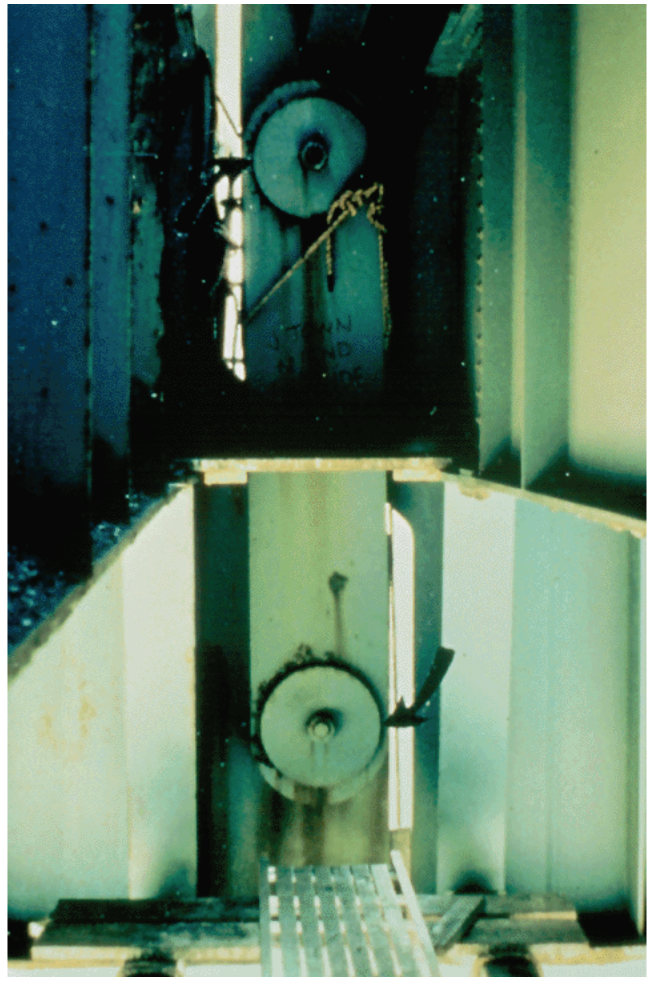

Figure 8.4.20 Rust Stains from Pin Corrosion

Some movement due to traffic vibration may be observable. If this movement is excessive, or if there is significant vertical movement with live load passage, the pins or pin holes may be excessively worn.

The expansion dam, beam ends, and any other structural components in the hinge area should be studied to see if any unusual displacements have taken place.

Hangers

Due to the rotation of the pins and hangers under live load and thermal expansion, they tend to incur wear over a period of time. Since portions of the assembly are inaccessible, they are not normally painted by maintenance crews and will, with time, begin corroding. This type of connection may be exposed to the elements and the spray of passing traffic. It may also be directly underneath an expansion dam where water and brine solutions may collect. This moist, corrosion-causing solution will slowly dry out, only to be reactivated during the next wet cycle.

Hangers are easier to inspect than pins since they are exposed and readily accessible. Try to determine whether the hanger-pin connection is frozen, as this can induce large moments in the hanger plates.

Examine accessible surface and edges closely for cracks (see Figure 8.4.21). The most critical areas are the ends beyond the pin centerlines and the juncture between the heads and shanks of eyebars. Note surface condition and section loss.

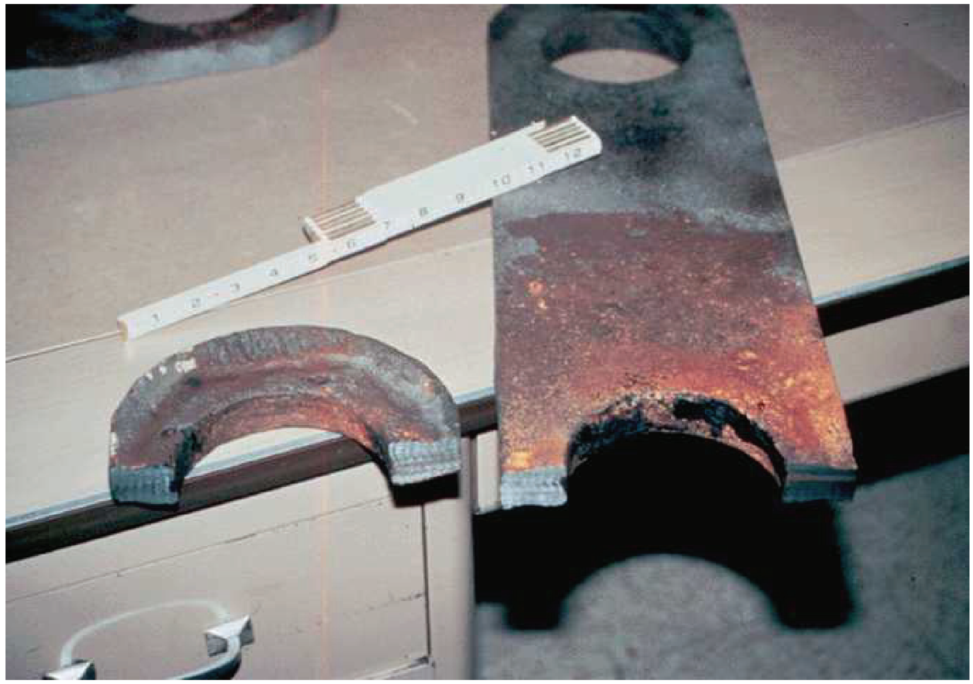

Figure 8.4.21 Corroded Hanger Plate

Assess the condition of the back side of the link by use of light and inspection mirror, if possible. Note the presence of corrosion. It may be helpful to probe with a wire or slender steel ruler.

Examine both sides of the plate for cracks due to bending of the plate from a frozen pin connection. Observe the amount of corrosion buildup between the webs of the girders and the back faces of the plates. Inspect the hanger plate for bowing or out-of-plane distortion from the webs of the girders (see Figure 8.4.22). Any welds should be investigated for cracks. If the plate is bowed, check carefully at the point of maximum bow for cracks that might be indicated by a broken paint film and corrosion.

Measure the distance between the back of the hanger and the face of the web at several locations. Compare these measurements from location to location and hanger to hanger. Variations greater than 3 mm (1/8 inch) could indicate twisting of the hanger bars or lateral movement due to rust packing. These measurements should be carefully described and recorded in permanent notes for comparison with as-built drawings and/or measurements taken at the next inspection.

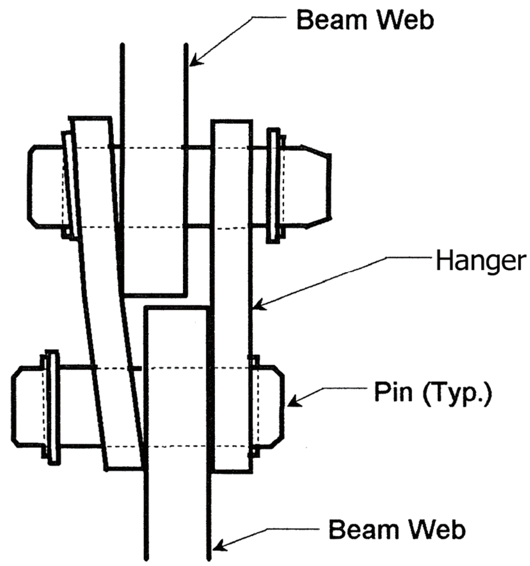

Figure 8.4.22 Bowing Due to Out of Plane Distortion of Hanger

Pins

Rarely is the pin directly exposed in a pin and hanger assembly. As a result, its inspection is difficult but not impossible. By carefully taking certain measurements, the apparent wear can be determined. If more than 3 mm (1/8 inch) net section loss of the diameter has occurred, it should be brought to the attention of the bridge engineer at once (see Figure 8.4.23). Wear to the pins and hangers will generally occur in two locations: at the top of the pin and top of the hanger on the cantilevered span and at the bottom of the pin and the bottom of the hanger on the suspended span. Sometimes wear, loss of section, or lateral slippage may be indicated by misalignment of the deck expansion joints or surface over the hanger connection. When inspecting a pin and hanger assembly, locate the center of the pin, measure the distance between the center of the pin and the end of the hanger, and compare to the plan dimensions, if available. Remember to allow for any tolerances since the pin was not machined to fit the hole exactly. Generally, this tolerance will be 1 mm (1/32 inch). If plans are not available, compare to previous measurements. The reduction in this length will be the apparent wear on the pin.

In a fixed pin and girder, wear will generally be on the top surface of the pin due to rotation from live load deflection and attractive forces. Locate the center of the pin, and measure the distance between the center of the pin and some convenient fixed point, usually the bottom of the top flange. Compare this distance to the plan dimensions to determine the decrease in the pin diameter.

The pin cap, if part of the assembly, should be checked with a straight edge for flatness.

Figure 8.4.23 Corroded Pin and Hanger Assembly

Retrofits

Since there are many problems associated with pin and hanger assemblies, several retrofit schemes have been devised to repair and/or provide redundancy in pin and hanger assemblies:

- Rod and saddle
- Underslung catcher
- Seated beam connection
- Continuity (field splice)
- Stainless steel replacements
- Non-metallic inserts and washers

The first two (rod & saddle and underslung catcher), are added to the structure and only carry load if the pin or hanger in a joint fails (see Figure 8.4.24). The gap between the "catcher" and the girder must be kept as small as possible to limit impact loading. If it is too tight, however, joint movement may be restrained. A neoprene bearing may be included in the assembly to lessen impact. The inspector should find out what the relative positions of the components should be by design and measure the critical points in the field for comparison.

The seated beam connection completely replace the pin and hanger assemblies. Vacant pin holes may be left under some schemes. Inspection of these details should be the same as inspection at field splices and bearings.

Sometimes a pin and hanger assembly is retrofitted by using a bolted field splice. This is done only after a structural engineer analyzes the bridge to determine if the members can support continuous spans instead of cantilevered spans. The inspector must remember to inspect both the positive and negative moment regions of the superstructure. Additional deflections may be introduced into piers and more movements may take place at expansion bearings when continuity is introduced. The areas should therefore receive extra attention.

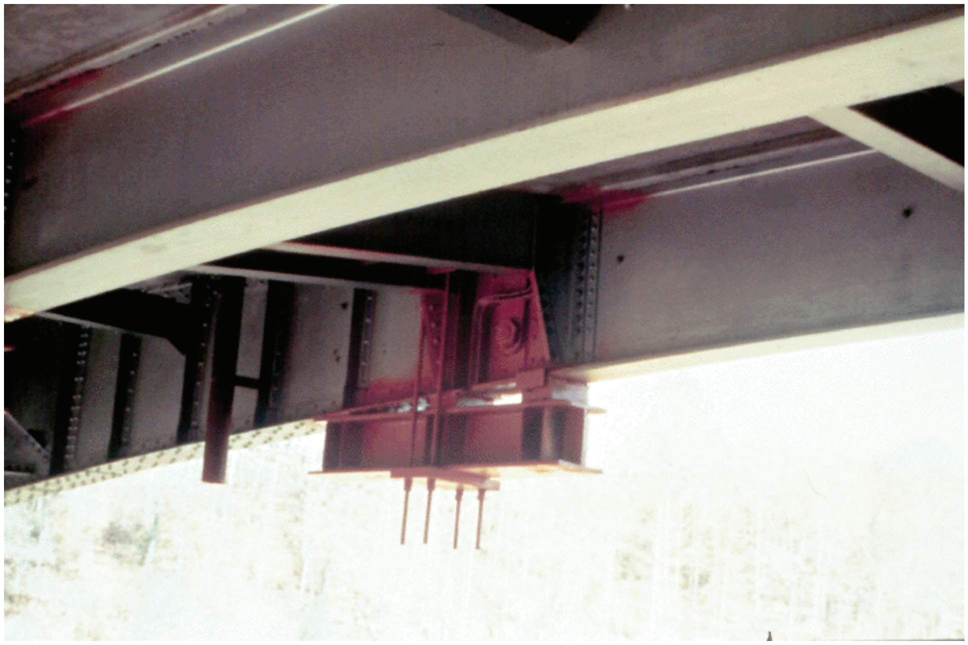

Figure 8.4.24   Underslung Catcher Retrofit

Replacing the pin and hanger assembly in kind with a structural grade of stainless steel eliminates potential failures due to corrosion related problems. Placing a non-metallic insert and washer prevents corrosion between the pin and hanger and allows for normal rotation.

Figure 8.4.25   Stainless Steel Pin and Hanger Assembly

8.4.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of pin and hanger assemblies. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component rating method and the AASHTO element level condition state assessment method.

NBI Rating Guidelines

Under the NBI rating guidelines, the pin and hanger assembly is considered part of the superstructure and does not have an individual rating. The rating for the superstructure should take into account the condition of the pin and hanger assembly and may be lowered due to a deficiency in the pin and hanger. The superstructure is still rated as a whole unit but the pin and hanger may be the determining factor in the given rating.

Using the NBI rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBI Rating Guidelines.

The previous inspection data should be considered along with current inspection findings to determine the correct rating.

Element Level Condition State Assessment

In an element level condition state assessment of a steel girder bridge with a pin and hanger assembly, the AASHTO CoRe element is:

Element No.	**Description**
160	Unpainted Pin & Hanger Assembly
161	Painted Pin & Hanger Assembly

The unit quantity for the pin and hanger assembly is each, and must be placed in one of the four available condition states for unpainted and five available condition states for painted assemblies depending on the extent and severity of deterioration. Condition State 1 is the best possible rating. See the *AASHTO Guide for Commonly Recognized (CoRe) Structural Elements* for condition state descriptions.

A Smart Flag is used when a specific condition exists, which is not described in the CoRe element condition state. The severity of the damage is captured by coding the appropriate Smart Flag condition state. The Smart Flag quantities are measured as each, with only one each of any given Smart Flag per bridge.

For damage due to fatigue, the “Steel Fatigue” Smart Flag, Element No. 356, can be used and one of the three condition states assigned. For rusting between members, the “Pack Rust” Smart Flag, Element No. 357, can be used and one of the four condition states assigned. For damage due to traffic impact, the “Traffic Impact” Smart Flag, Element No. 362, can be used and one of the three condition states assigned. For pin and hanger assemblies with section loss due to corrosion, the “Section Loss” Smart Flag, Element No. 363, can be used and one of the four condition states assigned.

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