**PDHonline Course C483 (3 PDH)**

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# FHWA Bridge Inspector's Manual
# Sections 8.3-5 - Steel Superstructures

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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

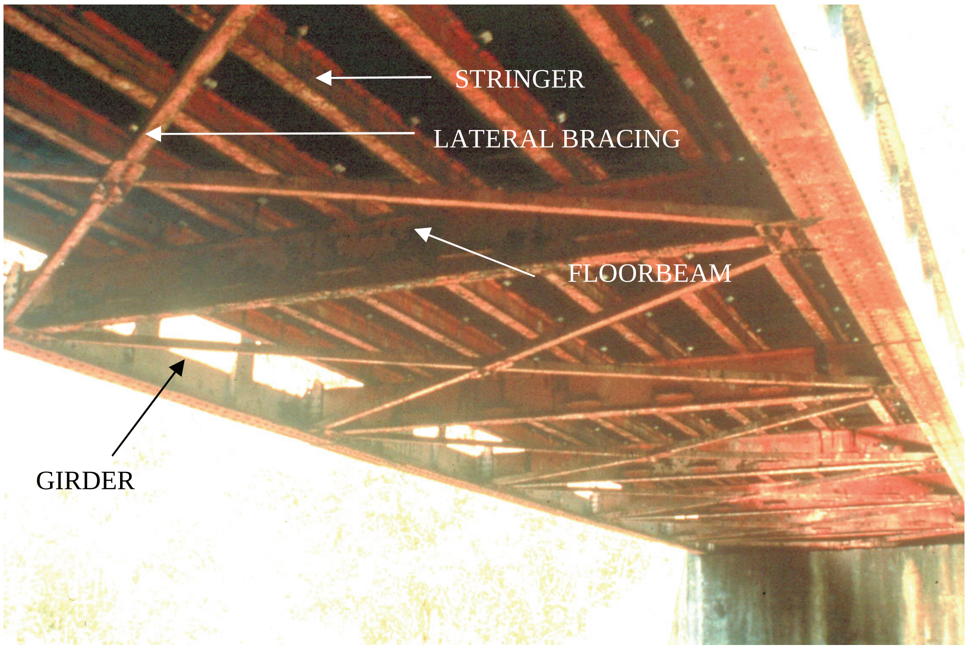


**Figure 8.3.9** Underside View of Through Girder Bridge with Lateral Bracing

**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.

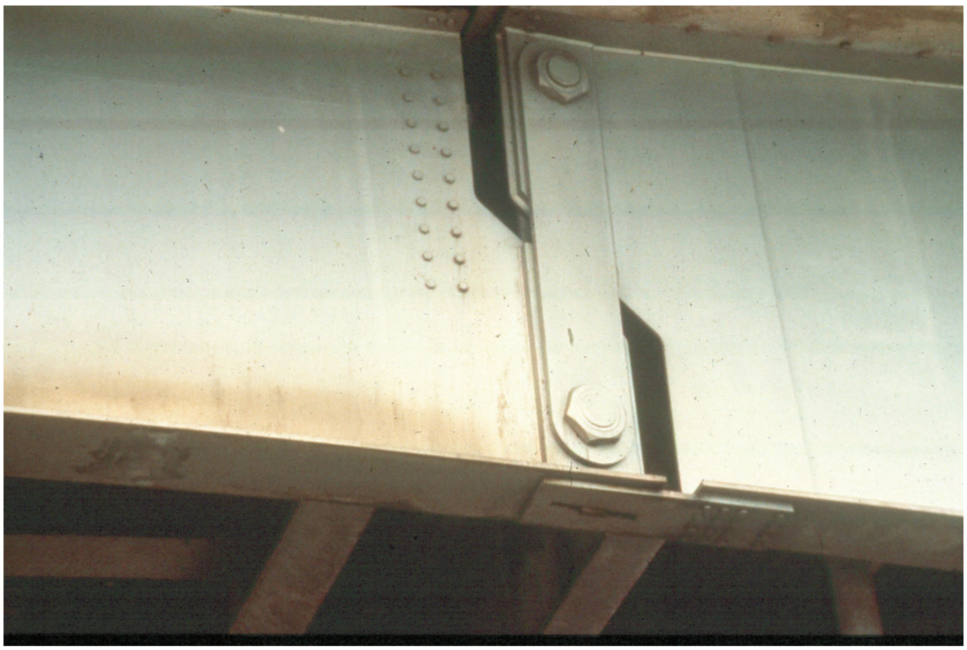

**Figure 8.3.10** Two Girder Bridge with Pin and Hanger Connection

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

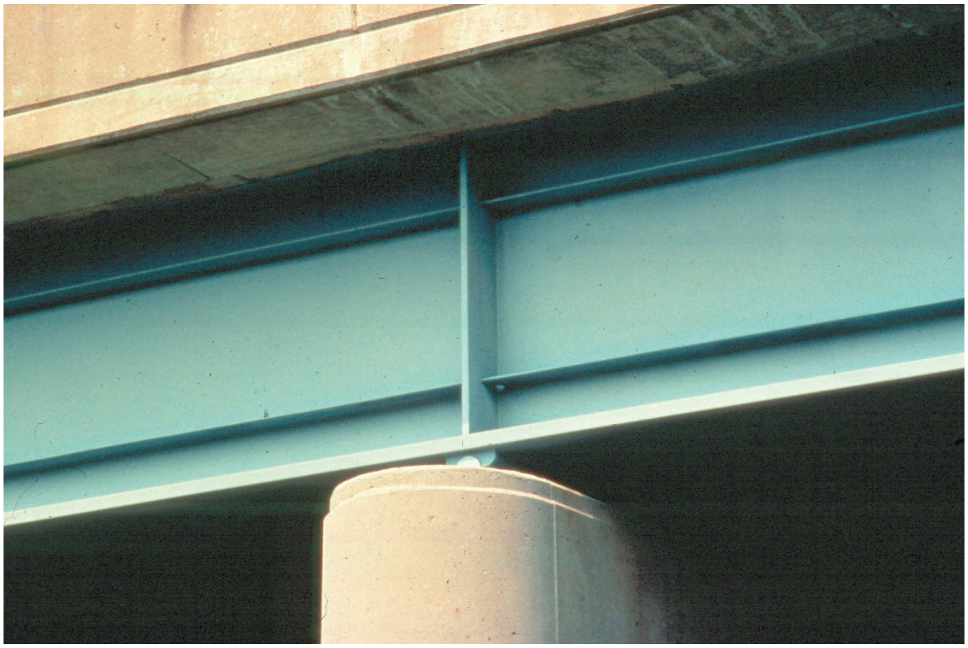

**Figure 8.3.14** Longitudinal Stiffener in Tension Zone on a Two Girder Bridge

**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

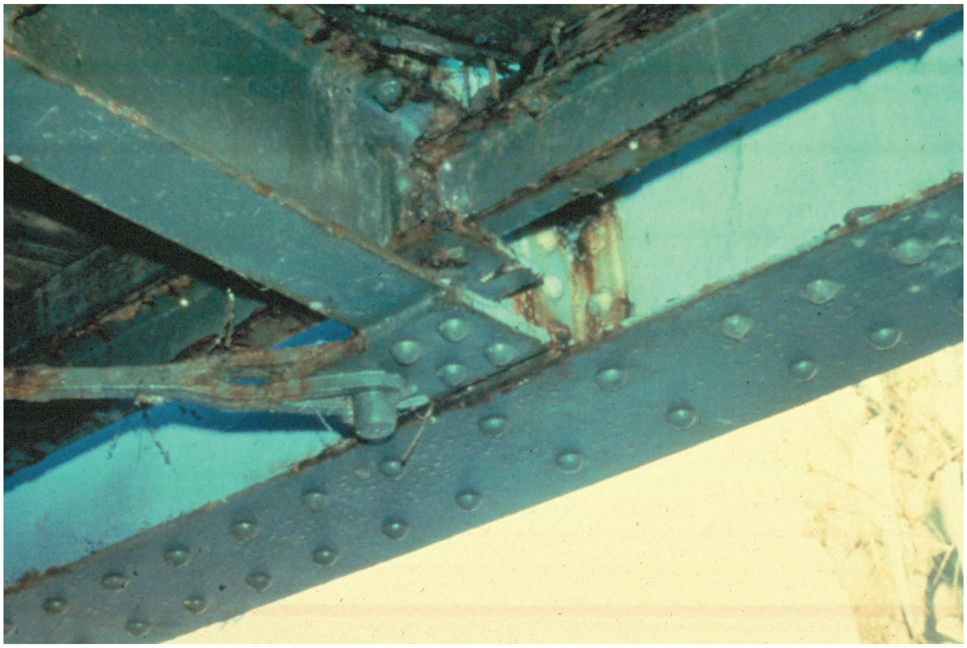

**Figure 8.3.17** Lateral Bracing Connection on a Through Girder Bridge

**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.

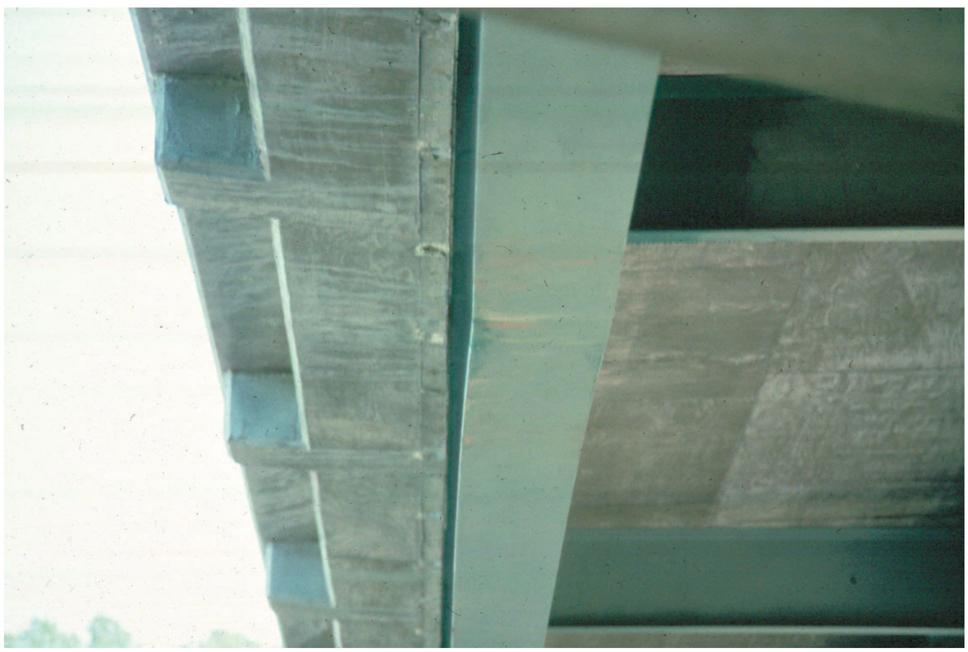

**Figure 8.3.18** Collision Damage to a Two Girder Bridge

**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).

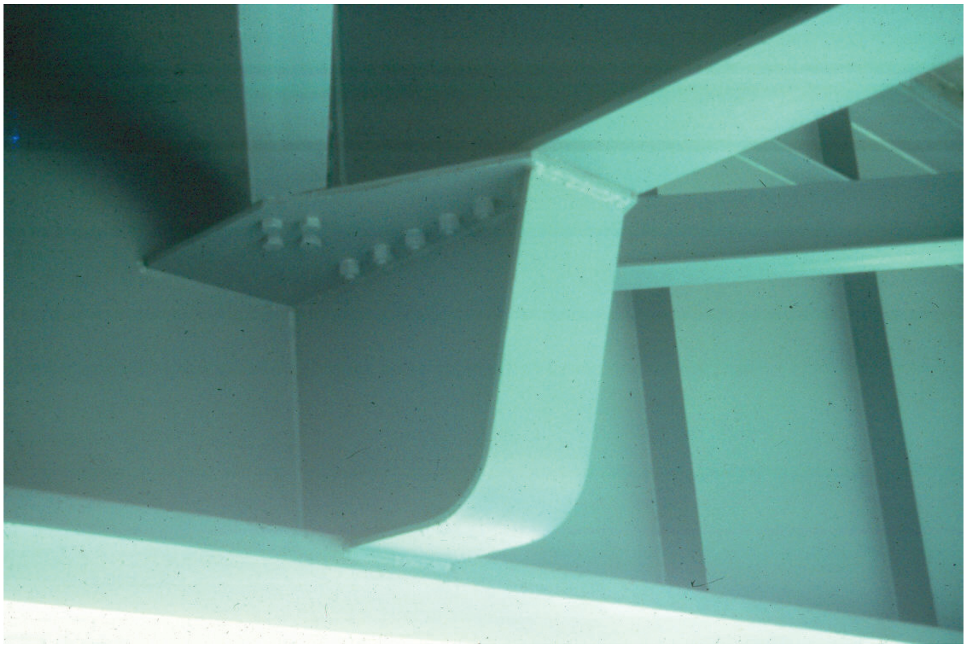

**Figure 8.3.22** Floorbeam to Girder Connection

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.

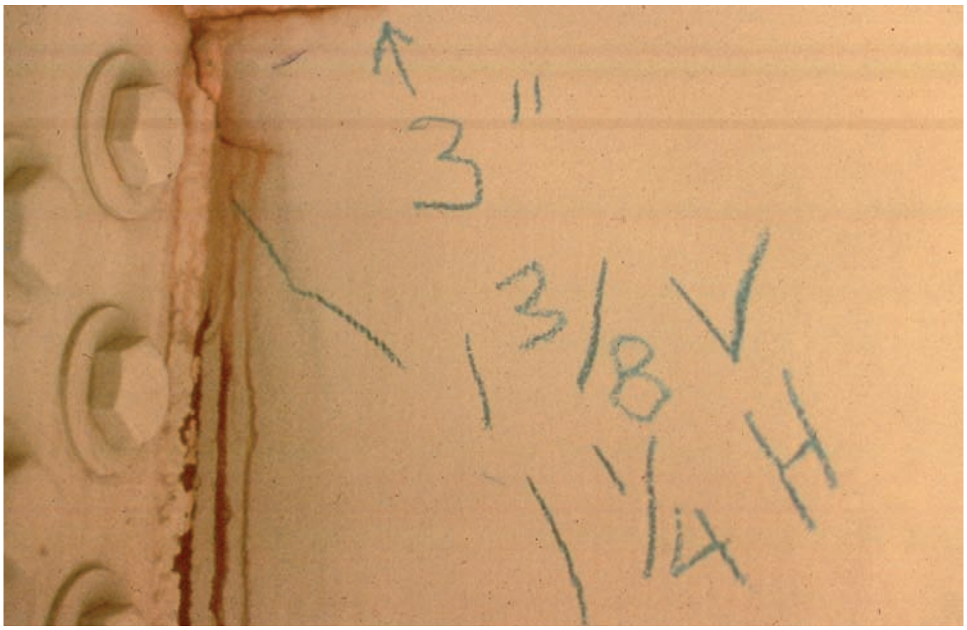


**Figure 8.3.23** Crack Caused by Out-of-plane Distortion

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.

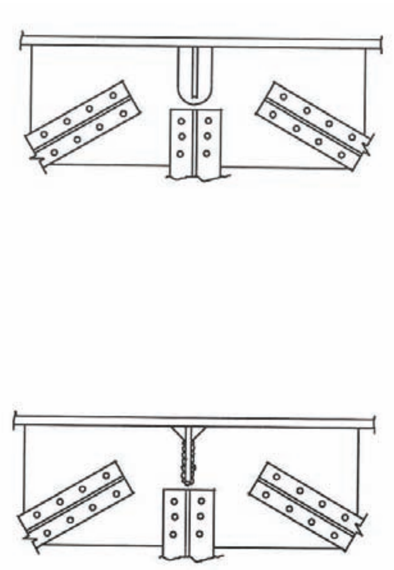

**Figure 8.3.24** Lateral Bracing Gusset at Floorbeam or Diaphragm Connection Plate

**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.

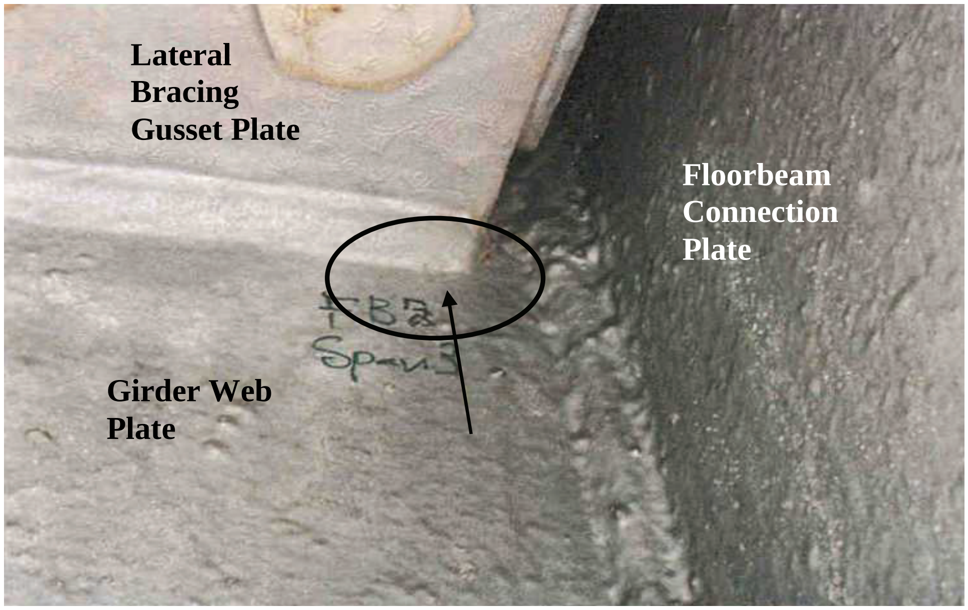


**Figure 8.3.26** Crack Caused by Out-of-plane Distortion

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).

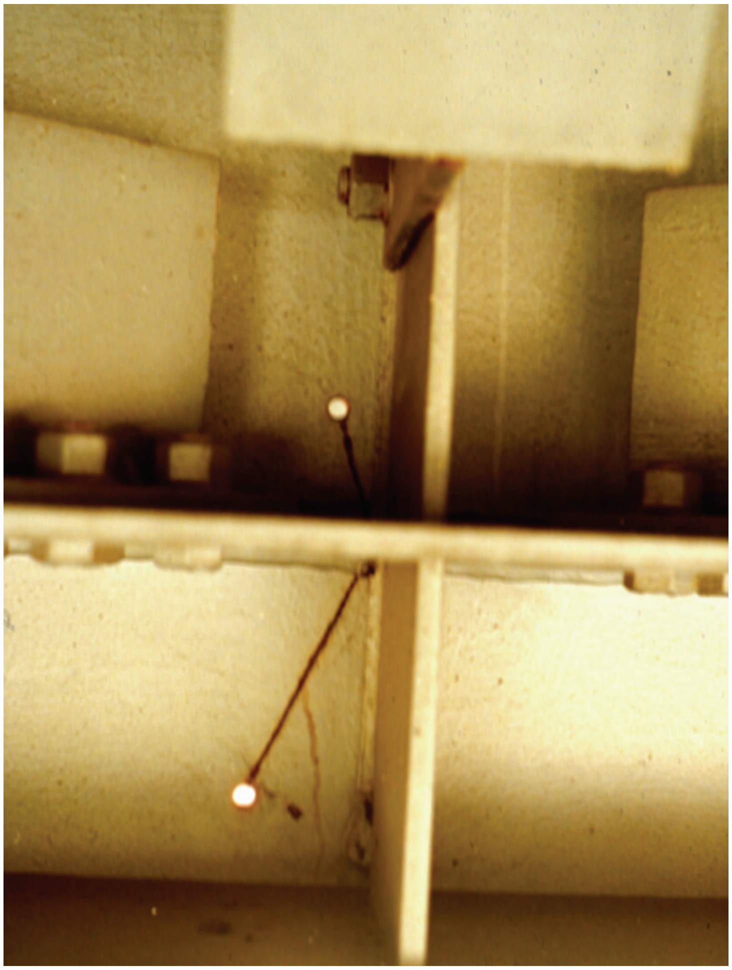

**Figure 8.3.27** Cracking in Girder Web at the Intersection of Horizontal Gusset Plate for Lateral Bracing and Transverse Stiffener

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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# 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.

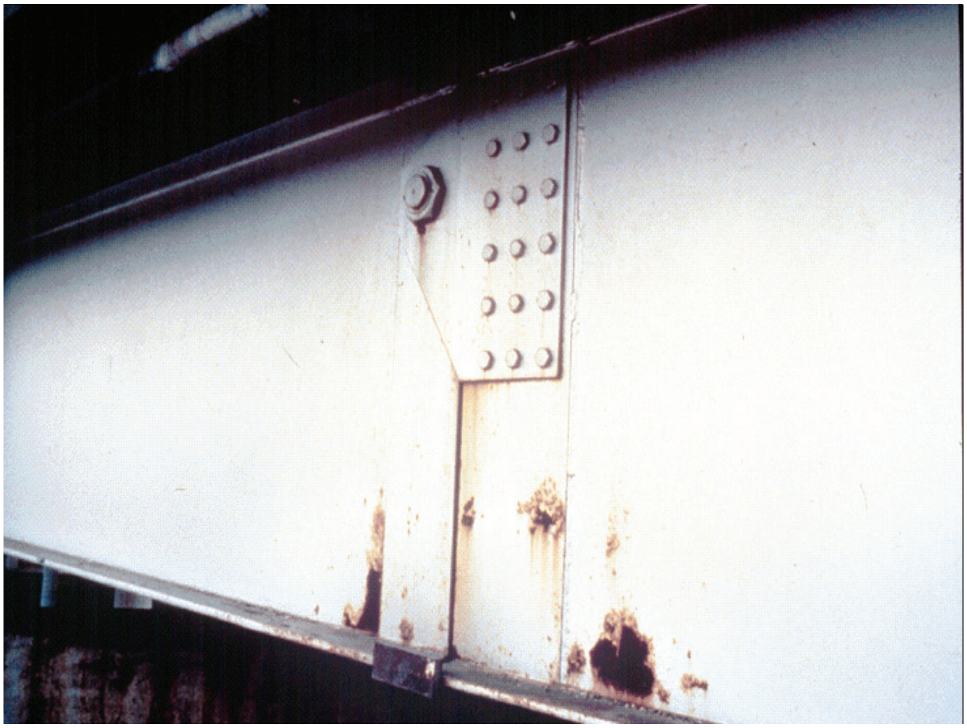

**Figure 8.4.2** Single Pin with Riveted Pin Plate

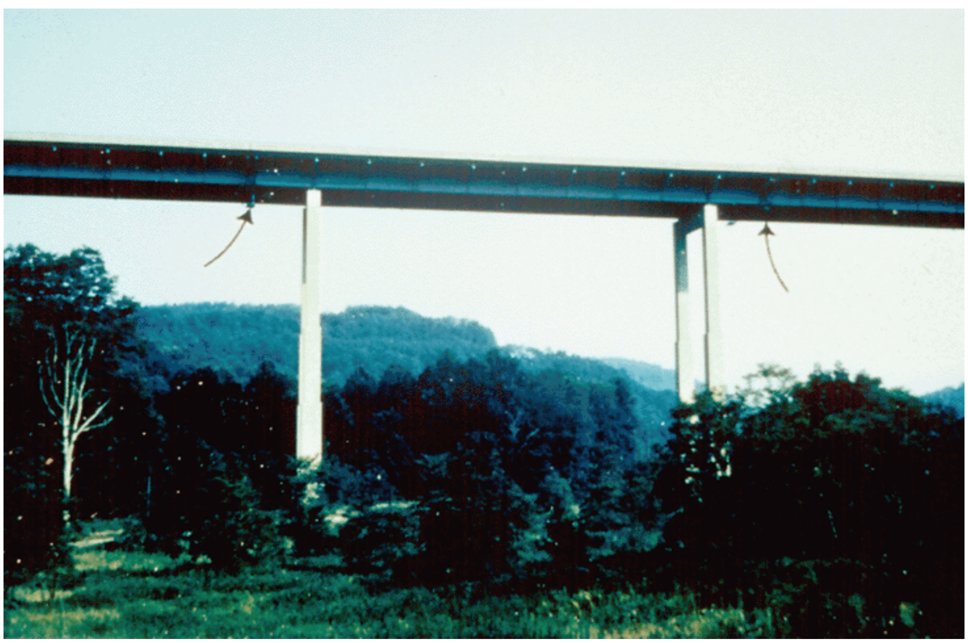

**Figure 8.4.3** Pin and Hanger Assembly Locations Relative to Piers

## 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.

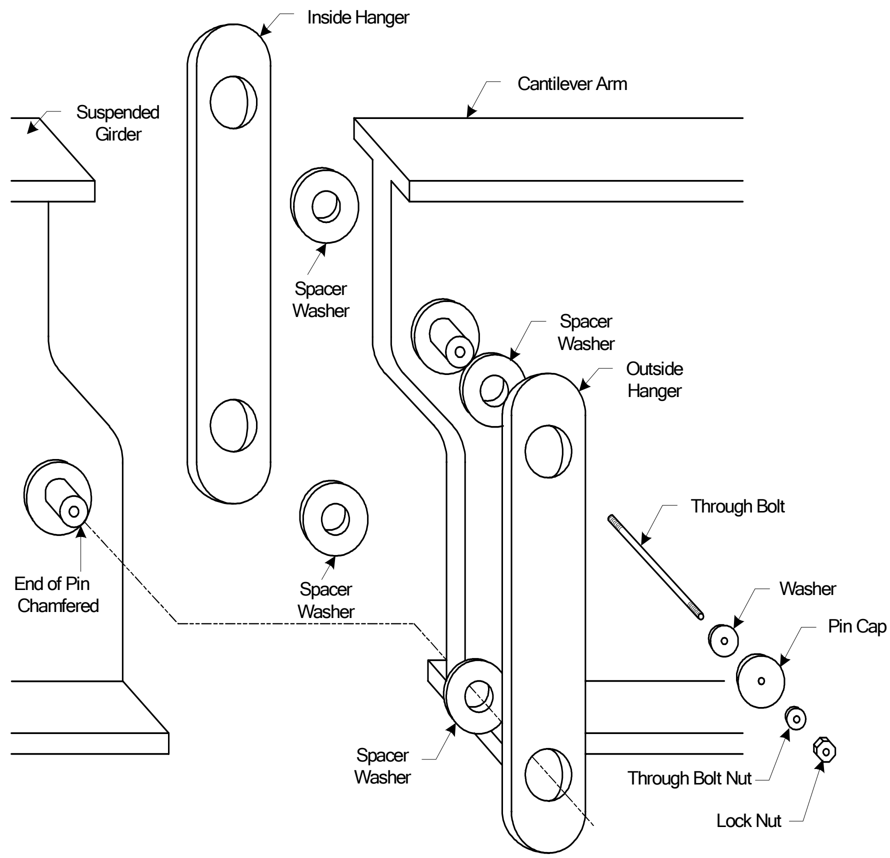


**Figure 8.4.4** Pin and Hanger Assembly

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).

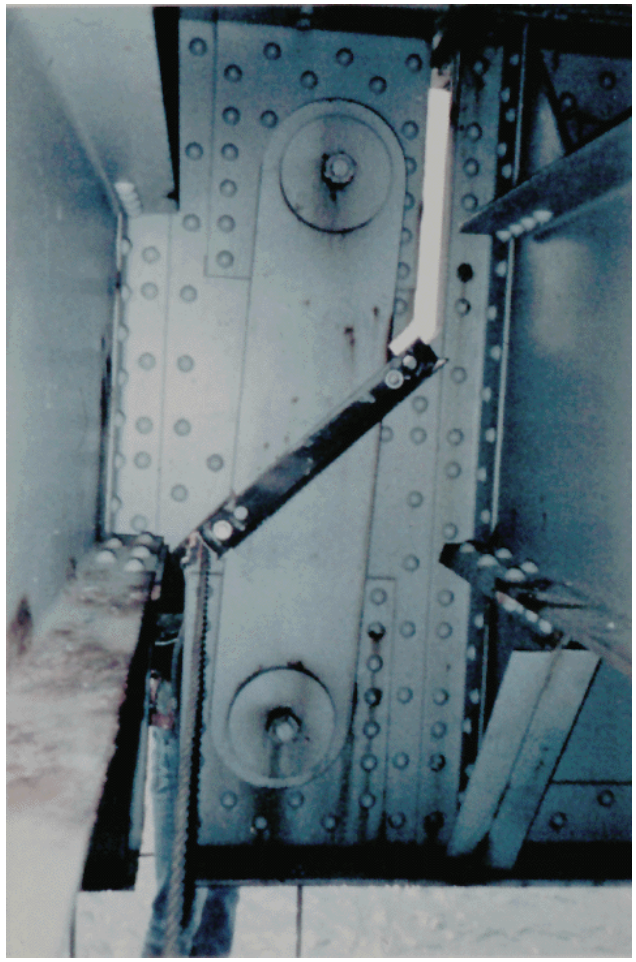

**Figure 8.4.5** Pin Cap with Through Bolt

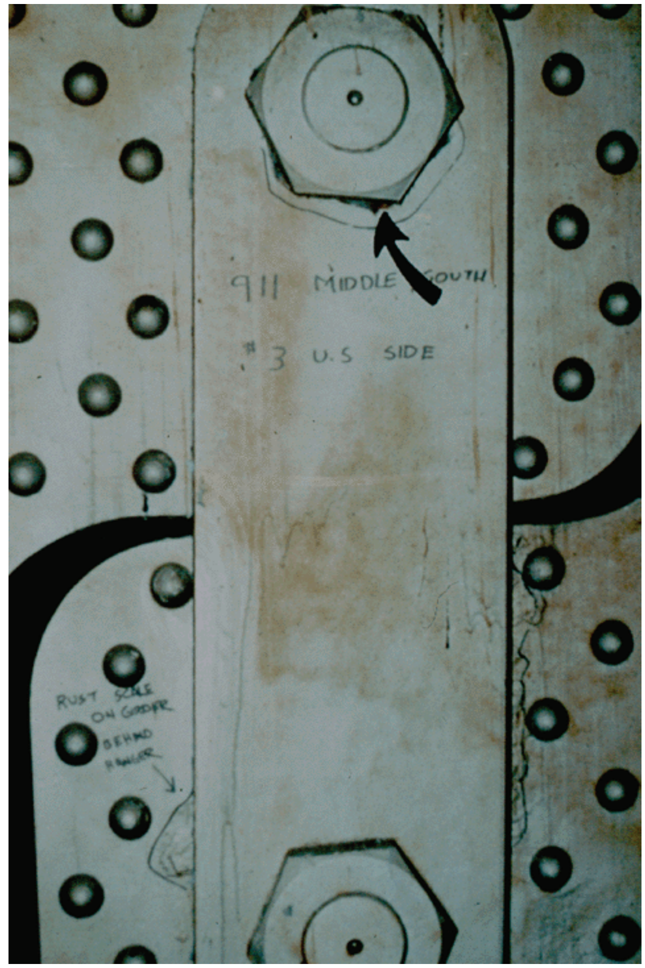


**Figure 8.4.6** Threaded Pin with Retaining Nut

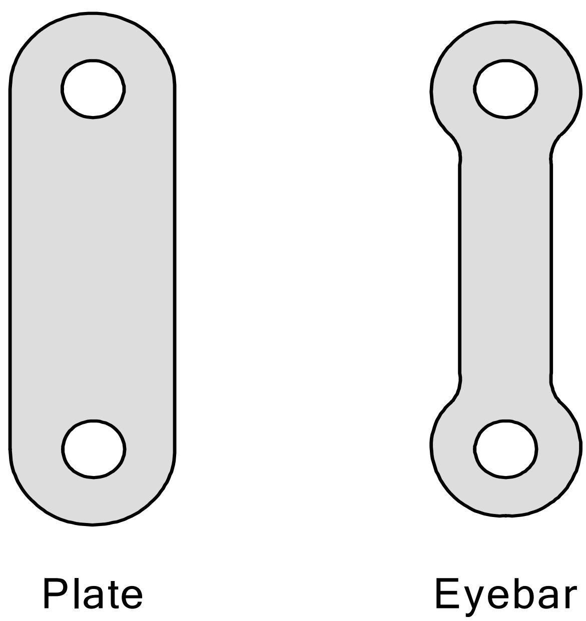


**Figure 8.4.7** Plate Hanger and Eyebar Shape Hanger Link

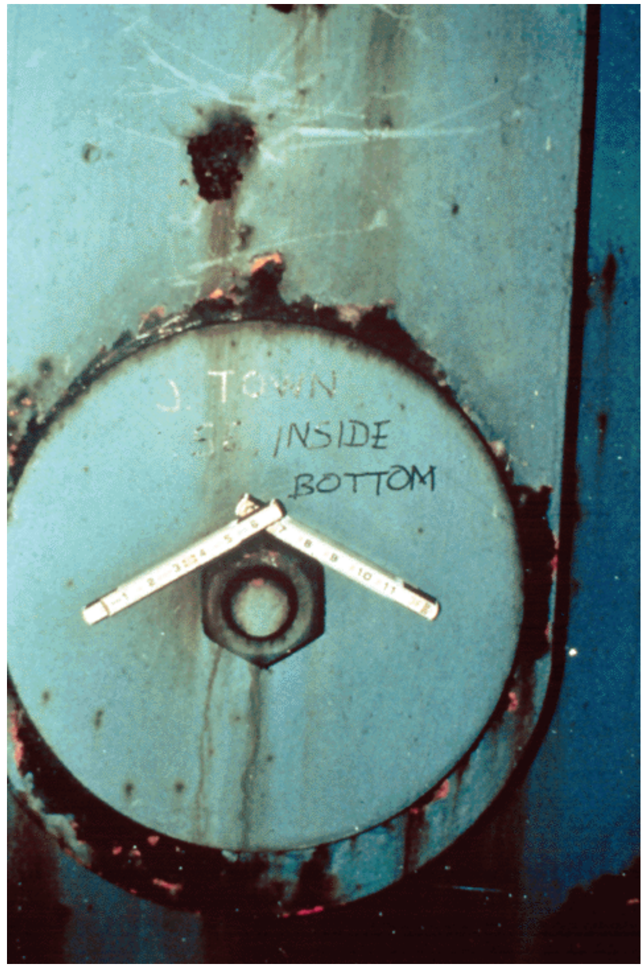


**Figure 8.4.8** Pin Cap, Through Bolt and Nut

**Figure 8.4.9** Retaining Nut

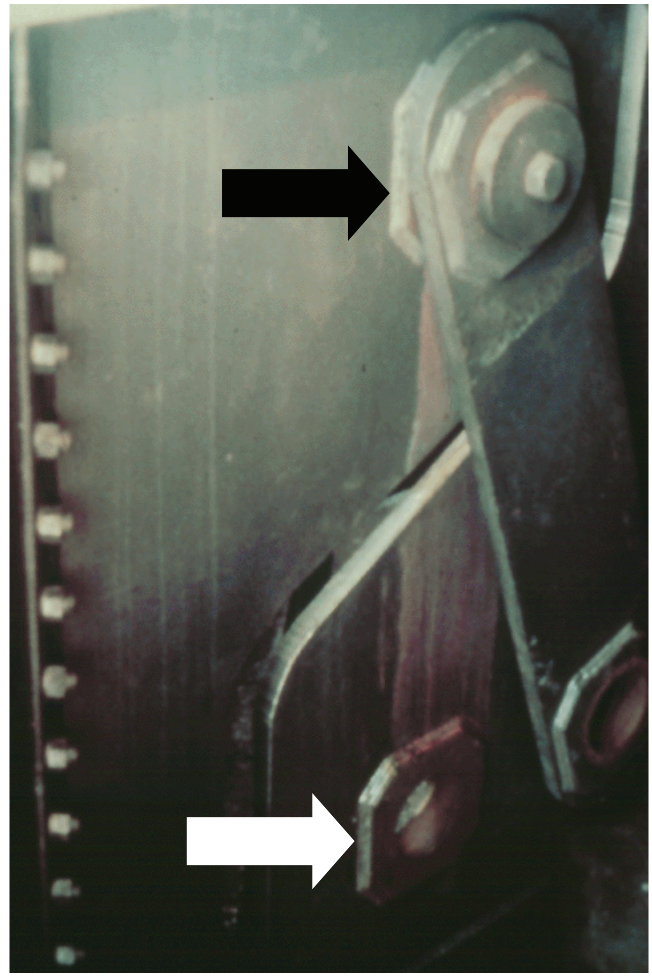

**Figure 8.4.10** Web Doubler Plates

**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.

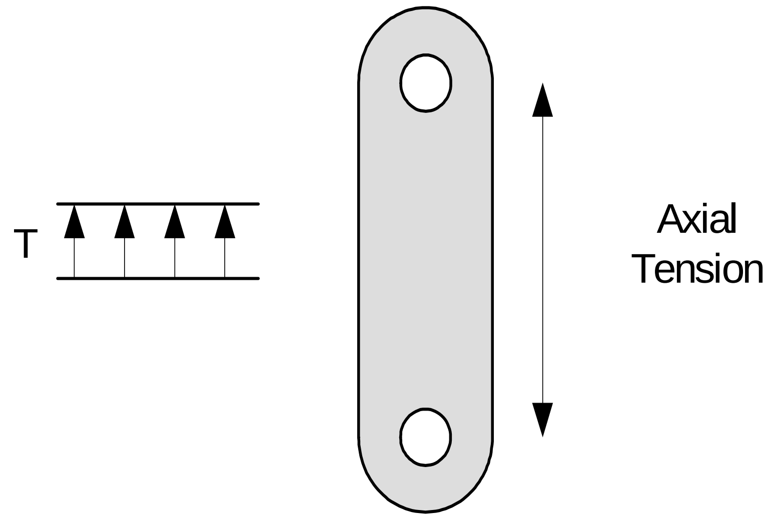


**Figure 8.4.11** Design Stress in a Hanger Link(Tension Only)

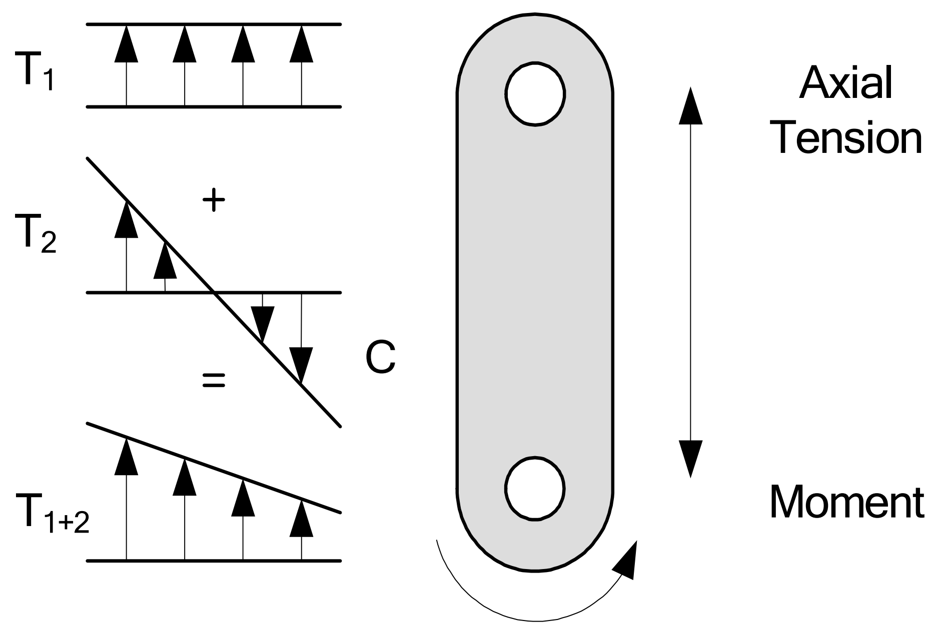


**Figure 8.4.12** Actual Stress in a Hanger Link (Tension and Bending)

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).

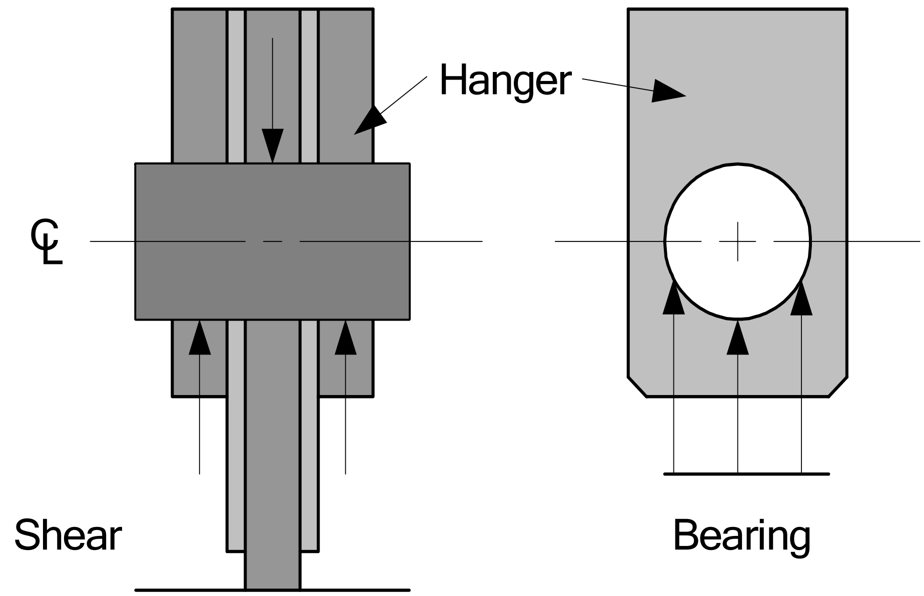


**Figure 8.4.13** Design Stress in a Pin (Shear and Bearing)

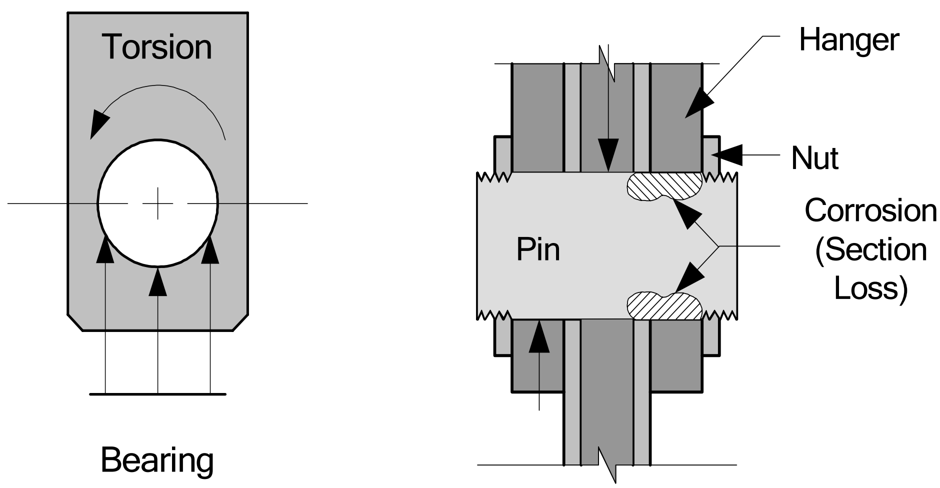


**Figure 8.4.14** Actual Stress in a Pin (Shear, Bearing and Torsion)

**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).

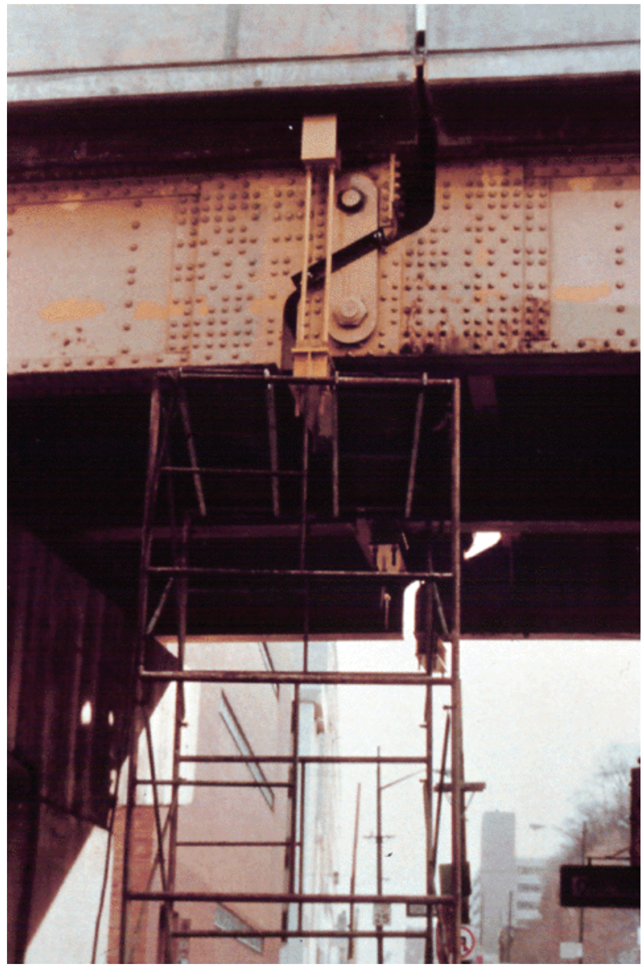

**Figure 8.4.18** Alternate Hanger Link Retaining System

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).

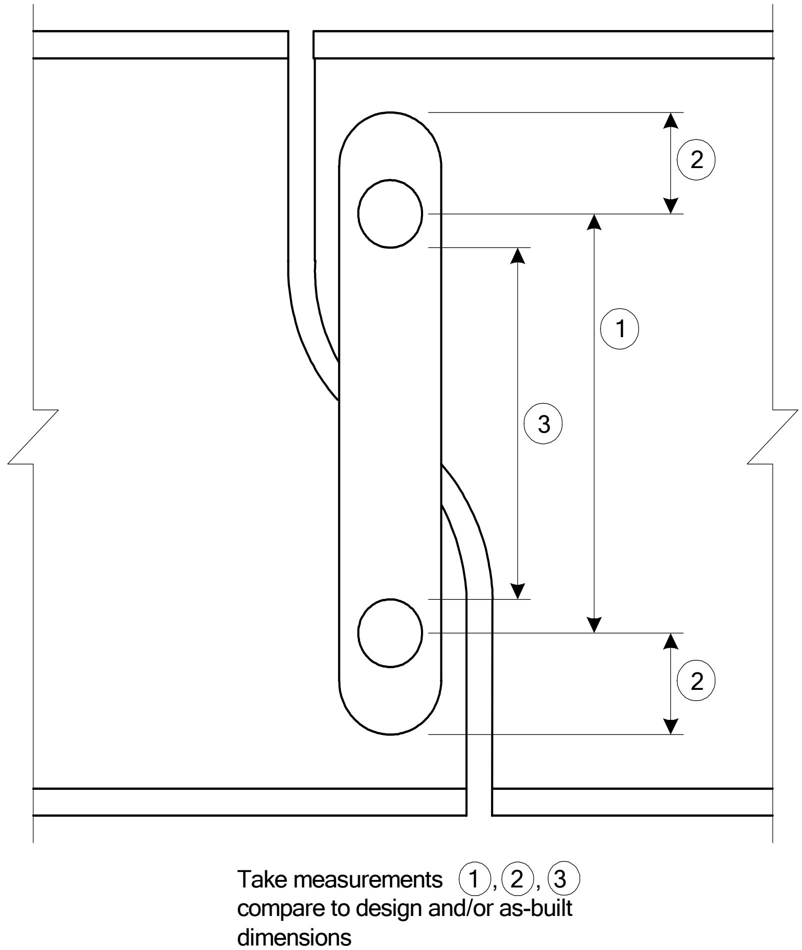


**Figure 8.4.19** Pin Measurement Locations

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.

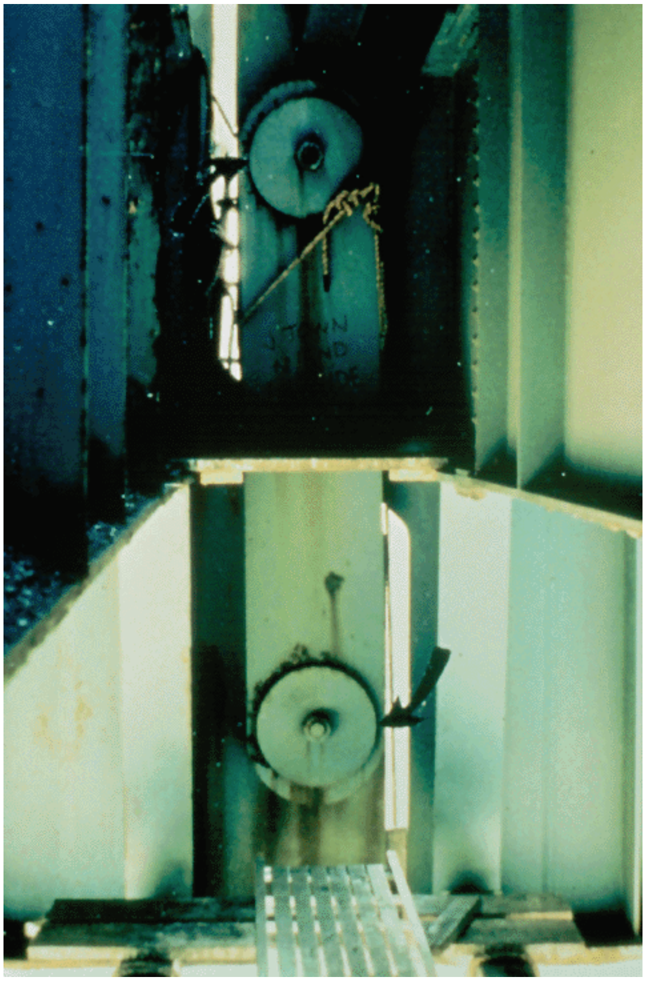

**Figure 8.4.20** Rust Stains from Pin Corrosion

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.

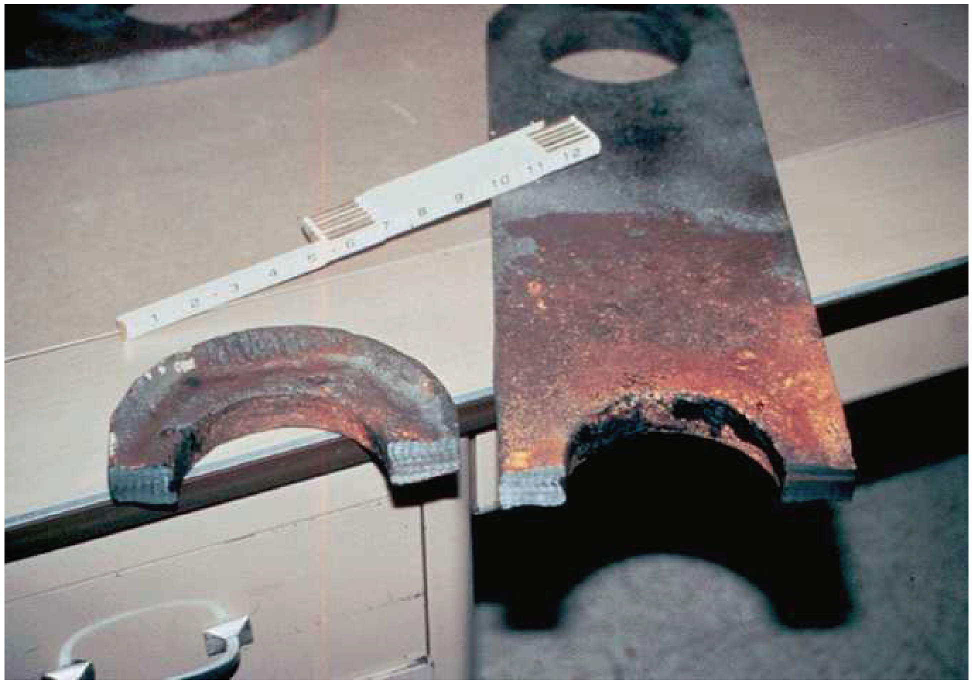

**Figure 8.4.21** Corroded Hanger Plate

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.

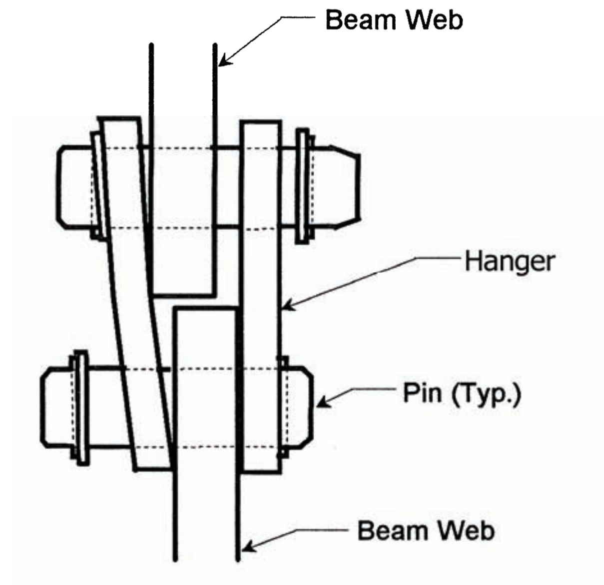


**Figure 8.4.22** Bowing Due to Out of Plane Distortion of Hanger

**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.

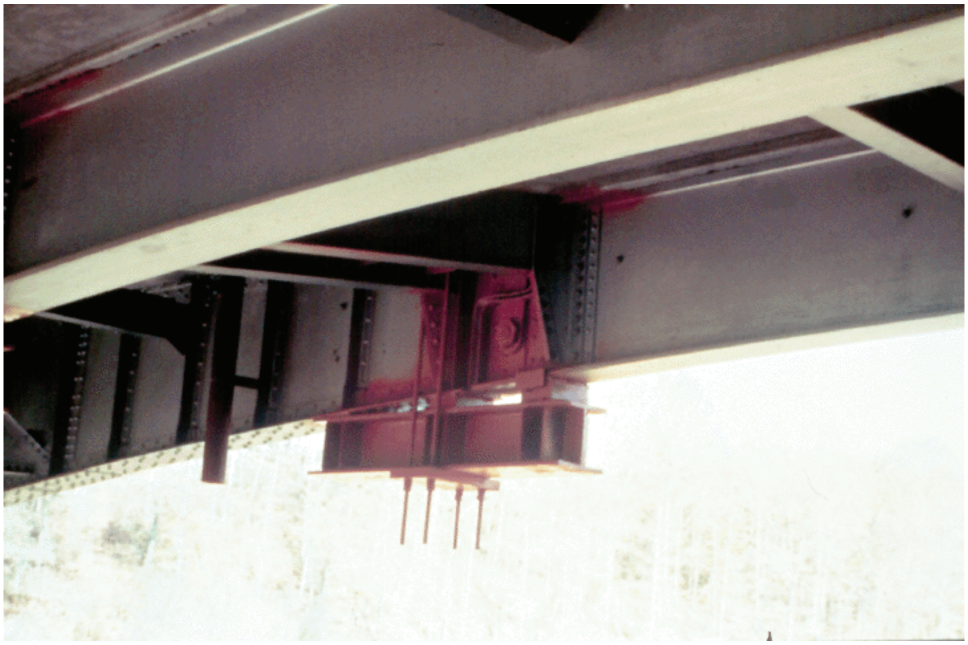

**Figure 8.4.24** Underslung Catcher Retrofit

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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# Table of Contents

# Topic 8.5 Steel Box Girders

## 8.5.1 Introduction

A box girder bridge is supported by one or more welded steel box girders. The rectangular or trapezoidal cross section of the box girder consists of two or more web plates connected to a single bottom flange plate.

Box girder bridges are used in simple spans of 75 feet or more (see Figure 8.5.1) and in continuous spans of 100 feet or more. They are frequently used for curved bridges due to their high degree of torsional rigidity (see Figure 8.5.2).

**Figure 8.5.1** Simple Span Box Girder Bridge

**Figure 8.5.2** Curved Box Girder Bridge

## 8.5.2 Design Characteristics

**Configuration**

A box girder bridge can use a single box configuration (see Figure 8.5.3) or have multiple (spread) boxes in its cross section (see Figure 8.5.4). Several factors such as deck width, span length, terrain and even aesthetics can all play a role in determining which configuration will be used.

**Figure 8.5.3** Box Girders With Cantilevered Supports for Deck

**Figure 8.5.4** Spread Box Girders

**Primary and Secondary Members**

The primary members of a box girder bridge are the box girders (including all internal bracing) and, on a curved bridge, the diaphragms. On a straight bridge, the diaphragms are secondary members. Diaphragms can be solid plates, rolled shapes (e.g., I-beams and channels), or cross frames constructed with angles, tee shapes, and plates (see Figure 8.5.5). Diaphragms may be on the interior or exterior of the box.

**Figure 8.5.5** Diaphragms – K Bracing and Plate

**Function of an Internal Stiffener**

The webs and bottom flange of large box shapes must be stiffened in areas of compressive stress. This is accomplished in part by stiffeners located inside the box member. The stiffeners are designed to help the box girder resist buckling due to torsional and shear forces. The stiffeners limit the unsupported length of the web and bottom flange, which result in increased stability of the box girder. Box girders may also incorporate both diaphragm and top flange lateral bracing systems. External diaphragms may be used between box girders (see Figure 8.5.6). Box girders typically have an opening or access door to allow the bridge inspector to examine the inside of the box (see Figure 8.5.7).

**Figure 8.5.6** External Diaphragm

**Figure 8.5.7** Box Girder Access Door

**Fatigue Prone Details**

Some common areas for fatigue prone details are:

- Welded attachments inside the box

- Attachment welds in the tension zone
- Butt welds in adjacent longitudinal stiffeners
- Intersecting welds between webs and flanges
- Field Splices
- Fatigue Cracks can also occur due to web-gap distortion and out-of-plane distortion

Inspection of these areas will be discussed in further detail later in this section.

**Fracture Critical Areas**

Box girder bridges may be fracture critical depending on the number of box girders in the span. If the span has two or less box girders, then the structure is nonredundant and the box girders are fracture critical members.

**Deck Interaction**

The top flange may consist of individual plates welded to the top of each web plate. If the top flange plates incorporate shear connectors, the superstructure is composite with the concrete deck. A composite deck is one in which the deck and the superstructure work together to carry the live load (see Figure 8.5.8). Alternatively, the top flange may consist of a single plate extending the width of the box. This configuration is classified as an orthotropic steel plate deck (see Figure 8.5.9). A wearing surface is then placed on the top flange as the riding surface.

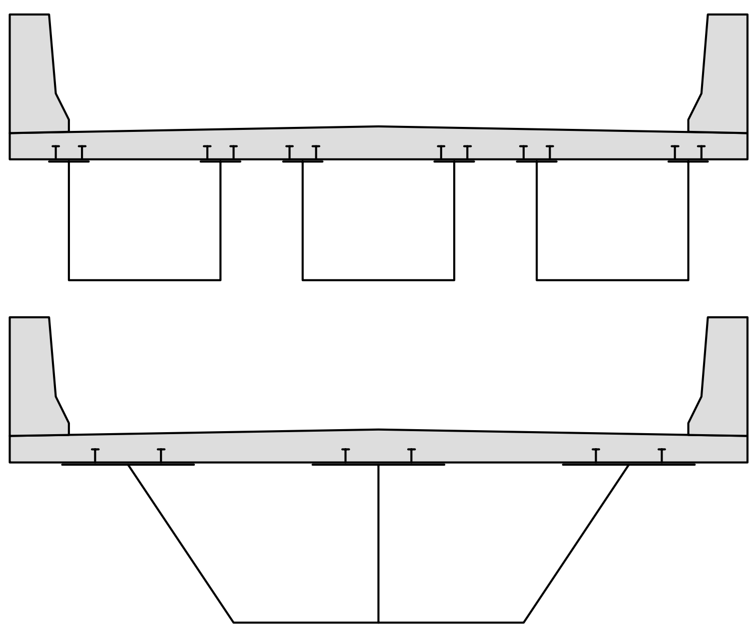

**Figure 8.5.8** Box Girder Cross Section with Composite Deck

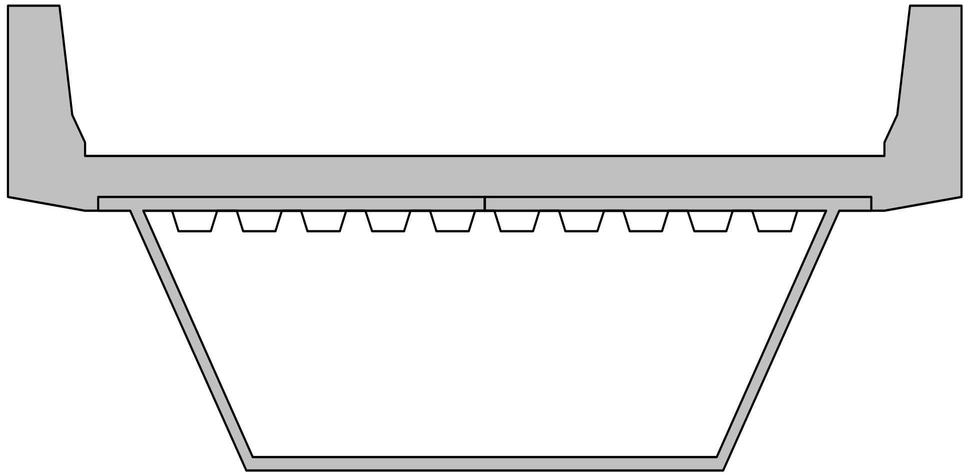

**Figure 8.5.9** Box Girder Cross Section with Orthotropic Steel Plate Deck

## 8.5.3 Overview of Common Defects

Common defects that occur on steel box girder bridges are:

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

See to 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.

## 8.5.4 Inspection Procedures and Locations

Box girders must be inspected on both the interior and the exterior. When examining the interior, the inspector should proceed with caution. Major concerns involved with inspecting a confined space include lack of sufficient oxygen, the presence of toxic or explosive gases, unusual temperatures and poor ventilation. Also, the distance between access hatches frequently exceeds the limit that rescue crews can reach in the event of an emergency (refer to Topic 3.2 for a more detailed description of these safety concerns).

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

**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 near substructure supports for cracks, section loss and buckling (see Figure 8.5.10). Be sure to include intermediate supports provided by piers (see Figure 8.5.11).

**Figure 8.5.10** Box Girder Shear Zone

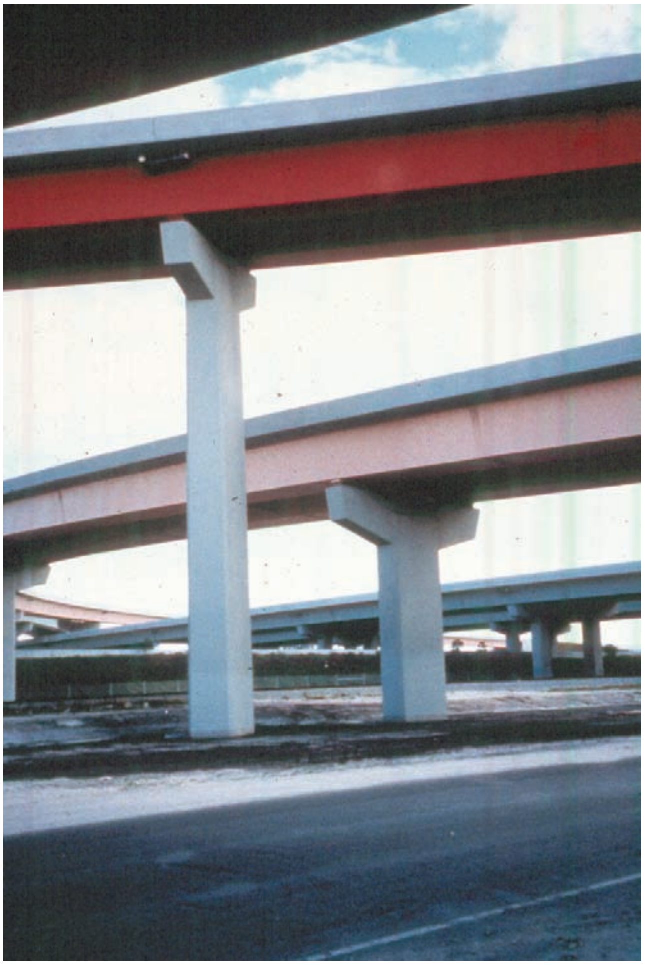

**Figure 8.5.11** Continuous Box Girders

**Flexure Zones**

The flexure zone of each box girder includes the entire length between the supports (see Figure 8.5.11). 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 box girders over the intermediate supports have high flexural stresses due to negative moment. If welded cover plates are present, check carefully at the ends of the cover plates for cracks.

**Secondary Members**

Examine the diaphragm and bracing connections for loose fasteners or cracked welds. 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.

### Areas That Trap Water and Debris

The areas that trap water and debris result in active corrosion cells and excessive loss in section. Check horizontal connection plates that can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Areas such as diaphragm to bottom flange connections can trap water, while external lateral bracing connection plates collect bird droppings and roadway debris. On box girder bridges, check the integrity of the drainage system. No water should be gaining access to the interior of the box(es).

Some steel box girders are designed or retrofitted with small drainage holes. If present, the drainage holes should be inspected for blockage and corrosion.

### Areas Exposed to Traffic

For box girders over a highgway, railway or navigable channel, check the box girder for signs of collision damage. Document any loss of section, cracking, scrape marks or distortion.

### 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 all welds and welded attachments inside the box. This includes web stiffeners, flange stiffeners, diaphragms, lateral bracing, and stay-in-place deck panels. Stiffener weld terminations in tension zones are a special concern. Butt welds joining adjacent longitudinal stiffeners on the bottom flange serve as potential sources for cracks to propagate into the bottom flange. Check all intersecting welds between the webs and flanges, particularly in tension areas. Welds are considered to be intersecting if they are within 6 mm (1/4 inch) from each other. Check field splice areas, especially where stiffeners are welded to the web or bottom flange (see Figure 8.5.12).

**Figure 8.5.12** Field Splice

Check back-up bars. These bars are sometimes used inside a box girder to help fabricate the corner welds between the web and the flange. A significant potential exists for cracking if these bars are discontinuous or are connected to the web or flange by tack welds or intermittent fillet welds. If such conditions exist, it should be brought to the attention of the bridge engineer.

Back-up bars were often spliced with non-NDT-inspected butt welds. These welds may be flawed and deserve special care in inspection if present in tension areas (see Figure 8.5.13).

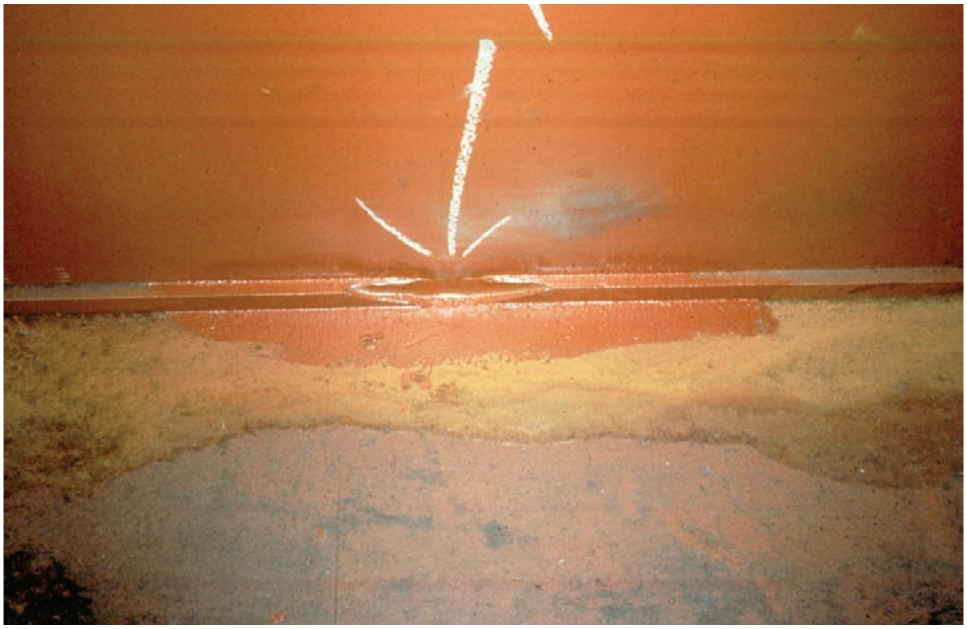

**Figure 8.5.13** Butt Welds in Back-up Bars Ground Out as Retrofit

**Fracture Critical Members**

The redundant nature of a box girder bridge depends primarily on the number of box girders in the span. If two or less box girders are used, the structure is considered nonredundant and the box girders are fracture critical members (see Figure 8.5.14). All previous inspection 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.

If three or more box girders are used, the structure is generally considered redundant (see Figure 8.5.15). However, if the spacing of the box girders is large, the structure may not be redundant.

**Figure 8.5.14** Non-redundant Box Girder Bridge

**Figure 8.5.15** Redundant Box Girder Bridge

**Out-of-Plane Distortions**

Box girder members with diaphragm connection plates are susceptible to similar out-of-plane distortions and fatigue cracking experienced in steel two girder systems (see Topic 8.3).

Out-of-plane distortion and fatigue cracking has developed in box girders of rectangular or trapezoidal cross section with k-frame, or plate diaphragms. Curved boxes and box girders which are subjected to high torsional loads are more likely to develop this type of crack at diaphragm connection plates. Frequent inspection should be made if truck traffic volume is high.

The web gap detail between the web and the transverse stiffeners is another area that is prone to cracking (see Figure 8.5.16). Similar to a girder and floorbeam connection, the top flange is restrained from lateral movement and cracks may develop in the region as a result of web plate bending.

**Figure 8.5.16** Box Girder Internal Diaphragm Not Attached to Flange

## 8.5.5 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 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 box girder bridge, the AASHTO CoRe element is:

| Element No. | Description |
|---|---|
| 101 | Unpainted Closed Web / Box Girder |
| 102 | Painted Closed Web / Box Girder |

The unit quantity for the steel box 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.

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 box girders with section loss, the "Section Loss" Smart Flag, Element No. 363, can be used and one of the four condition states assigned.