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Section 8

Inspection and Evaluation of Common Steel Superstructures

Topic 8.7 Steel Eyebars

8.7.1 Introduction

Eyebars are tension only members that require pins to make their end connections. Eyebars are predominantly found on older truss bridges, but can also be found on suspension chain bridges and as anchorage bars embedded within the substructures of long span bridges (see Figures 8.7.1 to 8.7.4).

Figure 8.7.1 Typical Eyebar Tension Member on a Truss

Figure 8.7.2 Eyebar Cantilevered Truss Bridge (Queensboro Bridge, NYC)

Figure 8.7.3 Eyebar Chain Suspension Bridge

Figure 8.7.4 Anchorage Eyebar

Heat treated steel eyebars have been used in bridges all over the world. One of these eyebars failed on December 15, 1967, sending the Point Pleasant Bridge (Silver Bridge), built in 1928, into the Ohio River between Point Pleasant, West Virginia and Kanauga, Ohio (see Figure 8.7.5). Forty-six people died and nine were injured due to the fracture of an eyebar in the north suspension chain on the Ohio side.

Figure 8.7.5 Collapsed Silver Bridge

Since the collapse of the Silver Bridge, there has been considerable public and professional concern over the safety of existing bridges, especially those containing eyebars. Many of these structures have been inspected and analyzed (see Figure 8.7.6). As a result, costly structural modifications and retrofits were made to many of these bridges (see Figure 8.7.7), while some others have been demolished. Eyebars are rarely used in new bridge designs but are present on many existing bridges.

Figure 8.7.6 Inspection of Eyebars

Figure 8.7.7 Retrofit of Eyebars to Add Redundancy

The design of the eyebar connections does not allow for inspection by common techniques. These connections collect water and promote corrosion at the critical point on the eyebar head (see Figure 8.7.8).

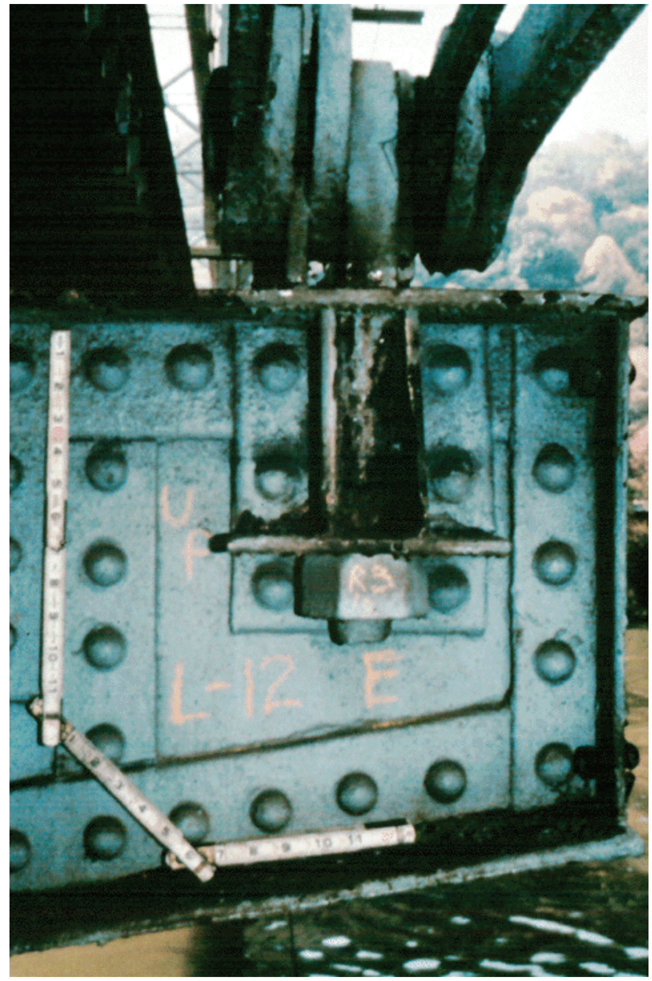


Figure 8.7.8 Eyebar Connection with Corrosion

8.7.2 Design Characteristics

Development of Steel Eyebars

In the late 1800's and early 1900's bridge spans began to increase in length, providing a need for higher strength steel. Prior to this time eyebars were made of wrought iron. The Eads Bridge in St. Louis, completed in 1874, was the first major steel bridge in America and the first in the world to use alloy steel. (see Figure 8.7.9).

Figure 8.7.9 Eads Bridge, St. Louis

Nickel alloy steel eyebars were developed around 1900. Nickel steel showed high physical properties with a yield point of 380 MPa (55,000 psi) and an ultimate strength of 620 MPa (90,000 psi). The major disadvantage of this steel was that it cost 2-1/2 cents per pound more than common carbon steel. Nickel steel was also difficult to roll without surface defects.

Sometime around 1915 mild grade heat treated steel eyebars (basically a "1035" steel) were developed with an ultimate strength of 550 MPa (80,000 psi) and a yield point of 345 MPa (50,000 psi). These eyebars were only 1 cent more per pound than common carbon steel.

In 1923 a high tension, mild grade heat treated steel eyebar was developed. The guaranteed minimum ultimate strength of 725 MPa (105,000 psi) and minimum yield point of 515 MPa (75,000 psi) made these bars equal to wire cable with added stiffness but no added cost. These "1060" steel eyebars were used on the Silver Bridge.

These heat treated alloy steels were extremely strong and contributed to substantial cost savings, but they could not be easily welded.

Forging

The ends of the eyebar shanks are connected by forging. Forging is a method of hot working to form steel by using hammering or pressing techniques.

Hammering

Hammering was the first method employed in shaping metals. An early form of the eyebar, shaped in this manner, is known as a loop rod (see Figures 8.7.10 and 8.7.11). Loop rods were first made of wrought iron, and later steel, by forging a heated bar around a pin, pounding the bar until a closed loop was formed.

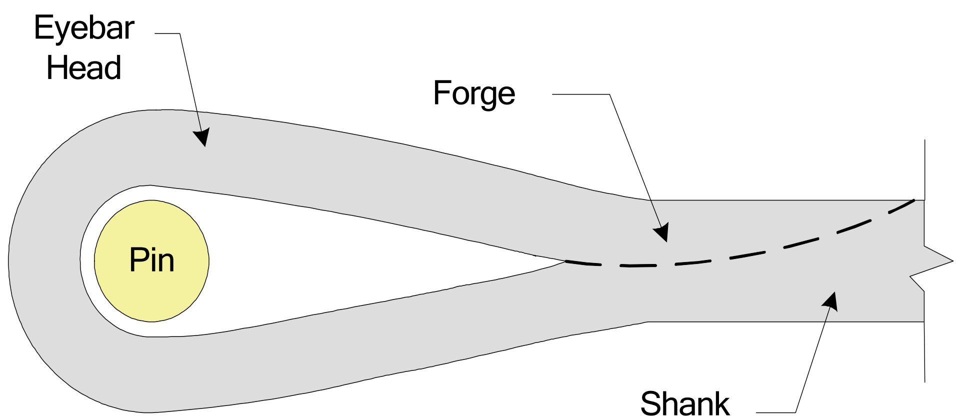


Figure 8.7.10 Forged Loop Rod

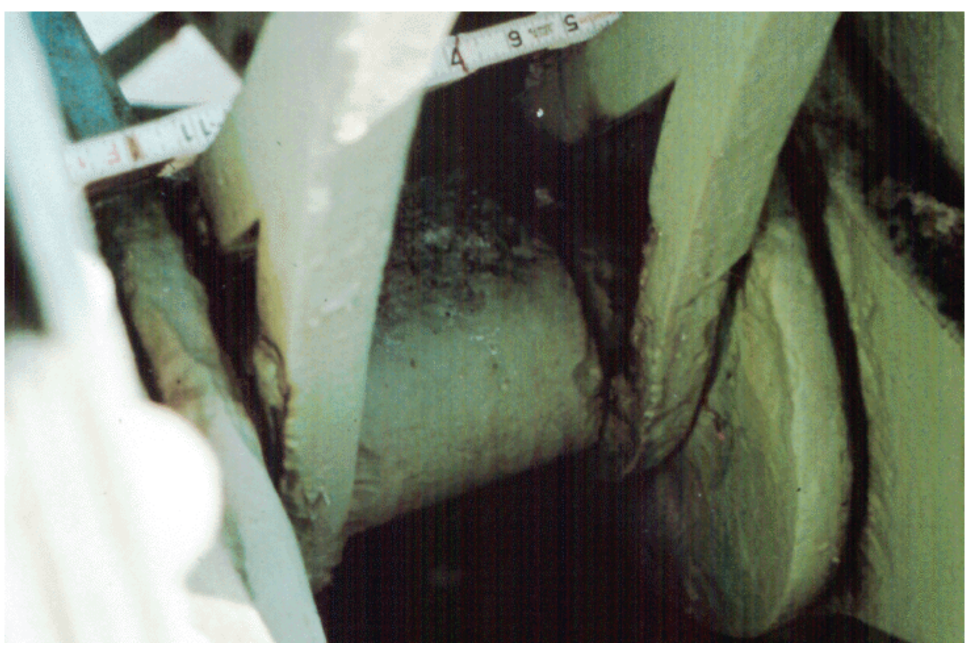


Figure 8.7.11 Close-up of the End of a Loop Rod

Pressing

Steel eyebars were also formed with a special type of mechanical forge press called an upsetting machine. The eyebar consists of the two heads (formed by casting) joined to the ends of the shaft. The upsetting machine clamps the eyebar pieces between two dies with vertical faces. The eyebar is then forged and shaped by the horizontal action of a ram operated by a crankshaft (see Figure 8.7.12). Most other forging presses operate with vertical rams.

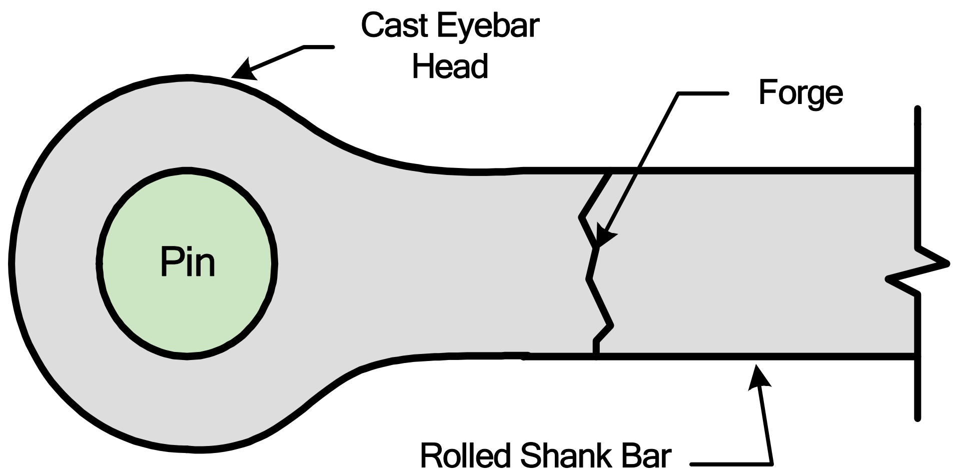


Figure 8.7.12 Forged Eyebar

Pin Hole

The pin hole in the enlarged head of the eyebar is commonly formed by boring. To fabricate the hole, flame cutting is permitted to within 50 mm (2 inches) of the pin diameter (see Figure 8.7.13).

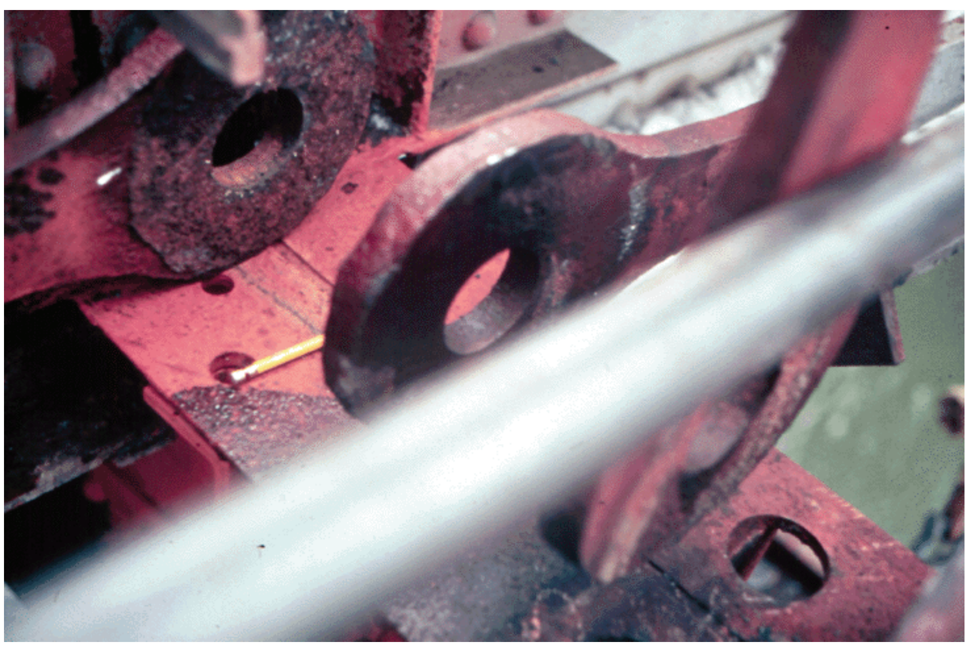


Figure 8.7.13 Eyebar Pin Hole (Disassembled Connection)

Heat Treating and Annealing

The inspector may find the terms "heat treated" and "annealed" on bridge plans to describe eyebars. Heat treating of steel is an operation in which the steel is heated and cooled, under controlled conditions according to a predetermined schedule, for the purpose of obtaining certain desired properties.

Through heat treatment various characteristics of steel can be enhanced. If steel is to be formed into intricate shapes, it can be made very soft and ductile by heat treatment. On the other hand, if it is to resist wear, it can be heat treated to a very hard, wear-resisting condition.

Annealing is a term used to describe several types of heat treatment which differ greatly in procedure yet all accomplish one or more of the following effects:

- Remove internal stresses
- "Soften", by altering mechanical properties
- Redefine the grain structure
- Produce a definite microstructure

More than one of these effects can often be obtained simultaneously.

Dimensions

The dimensions of a typical eyebar are as follows:

- Thickness - usually 25 to 50 mm (1 to 2 inches)
- Width - usually 200 to 400 mm (8 to 16 inches)
- Length - varied with bridge design

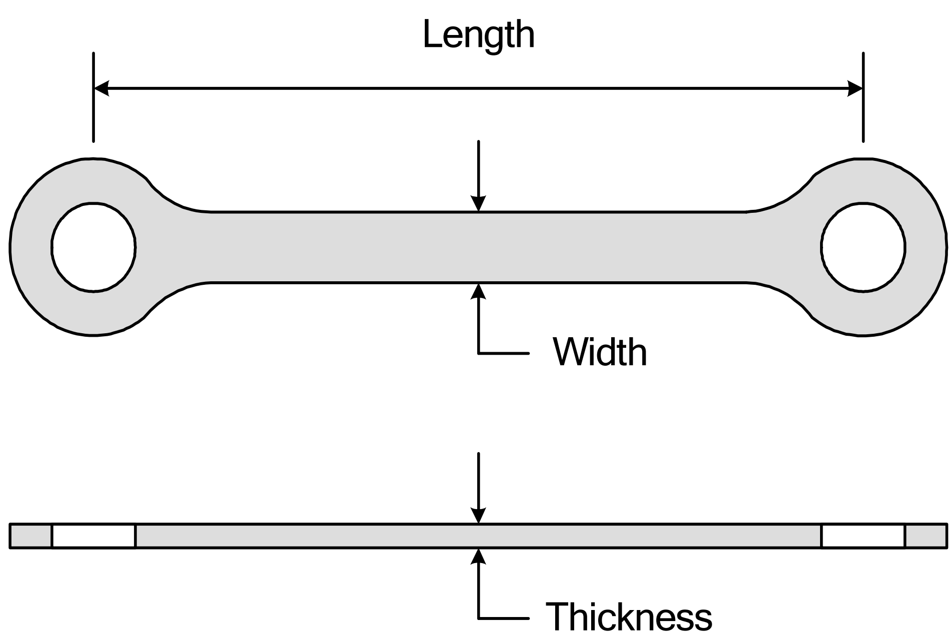


Figure 8.7.14 Eyebar Dimensions

The eyebars on the Silver Bridge were between 13.7 to 16.8 m (45 and 55 feet) in length, 300 mm (12 inches) wide, and varied in thickness.

Packing

Packing is the term used to describe the arrangement of all the eyebars at a given point. Eyebars may be tightly packed together or spread apart (see Figures 8.7.15 and 8.7.16). The packing should be symmetrical about the center-line of the member.

Figure 8.7.15 Loosely Packed Eyebar Connection

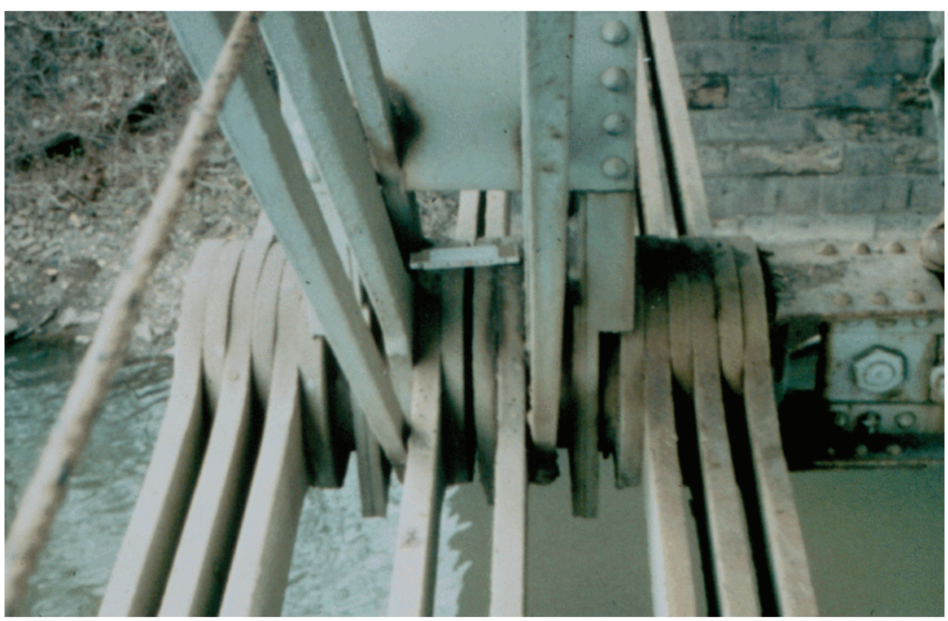


Figure 8.7.16 Tightly Packed Eyebar Connection

Spacers

Spacers or steel filling rings are often wrapped around the pin to prevent lateral movement within the eyebar pack (see Figure 8.7.17).

Figure 8.7.17 Steel Pin Spacer or Filling Ring

Redundancy

An internally redundant eyebar member will consist of three or more eyebars. Many eyebar members are internally non-redundant, having only one or two eyebars per member (see Figure 8.7.18).

The collapse of the Silver Bridge is attributed to the failure of an eyebar within a nonredundant eyebar member. When the first eyebar failed, the second eyebar was unable to carry the load due to lack of internal redundancy. The Silver Bridge was also not load path redundant which contributed to the complete collapse of the structure. Load path and internal (member) redundancy are discussed in detail in Topics P.2 and 8.1.

Figure 8.7.18 Nonredundant Eyebar Member

8.7.3 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.

8.7.4 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

Locations

Forge Zone

Inspect carefully the forged area around the eyebar head and the shank for cracks. Check the loop rods for cracks where the loop is formed (see Figure 8.7.19). Most eyebar failures are likely to occur in the forge zone.

Figure 8.7.19 Close-up of the Forge Zone on an Eyebar (Arrow denotes crack)

Tension Zone

Since an eyebar carries axial tension, the entire length must be closely examined for deficiencies that can initiate a crack. These deficiencies include notch effects due to mill flaws, corrosion or mechanical damage. The area around the eye and the transition to the shank where stress is the highest is the most critical.

Alignment

Check the alignment of the shank along the full length of the eyebar. Since the eyebar is a tension member, it should be straight. A bowed eyebar indicates that a compressive force has been introduced (see Figure 8.7.20).

Figure 8.7.20 Bowed Eyebar Member

Misalignment due to buckling can also be caused by movement at the substructure or changes in loading during rehabilitation (see Figures 8.7.21and 8.7.22). The eyebars of the same member should be parallel and evenly loaded.

Figure 8.7.21 Buckled Eyebar due to Abutment Movement

Figure 8.7.22 Non-parallel Eyebar Member

Areas That Trap Water and Debris

Areas that trap water and debris can result in active corrosion cells that can cause notches susceptible to fatigue or perforation and loss of section. On eyebar members, check the area between the eyebars especially if they are closely spaced.

Spacers

Examine the spacers on the pins to be sure they are holding the eyebars in their proper position (see Figure 8.7.23).

Figure 8.7.23 Corroded Spacer

Examine closely spaced eyebars at the pin for corrosion build-up (packed rust). These areas do not always receive proper maintenance due to their inaccessibility. Extreme pack rust can deform retainer nuts or cotter pins and push the eyebars off the pins.

Verify the eyebars are symmetrical about the central plane of the spacer (see Figure 8.7.24).

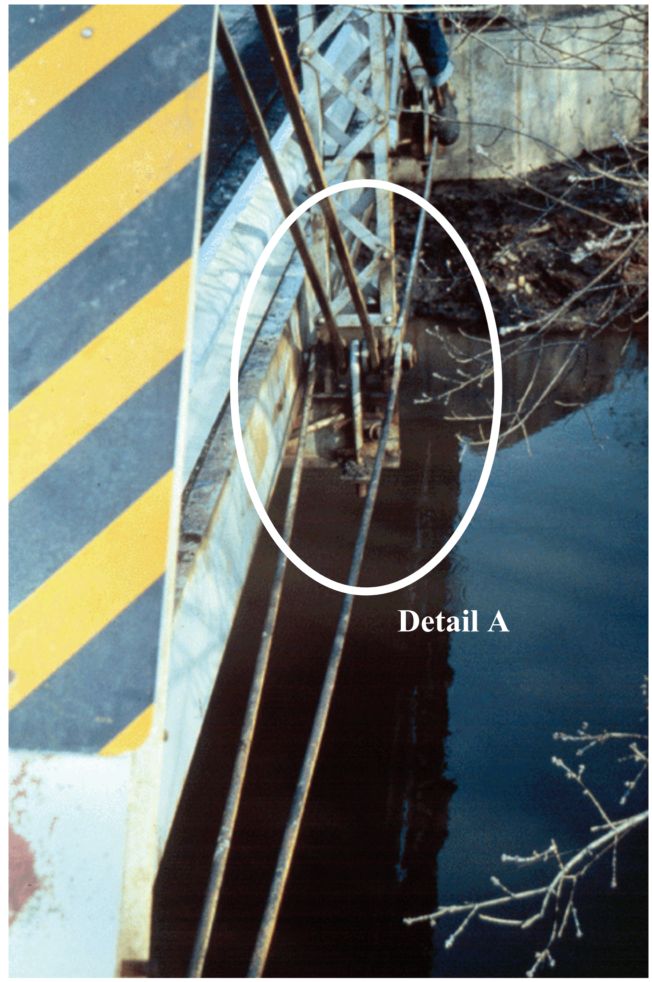
Figure 8.7.24 Symmetry at an Eyebar Connection

Areas Exposed to Traffic

Check underneath the bridge for collision damage if the bridge crosses over a highway, railway, or navigable channel. Document any cracks, section loss, or distortion found. On a suspension bridge using eyebars, investigate the eyebars along the curb lines and at the ends for collision damage.

Load Distribution

Check to determine if any eyebars are loose (unequal load distribution) or if they are frozen at the ends - preventing free rotation (see Figure 8.7.25). Check for panel point pins or eyebar twisting.



Detail A

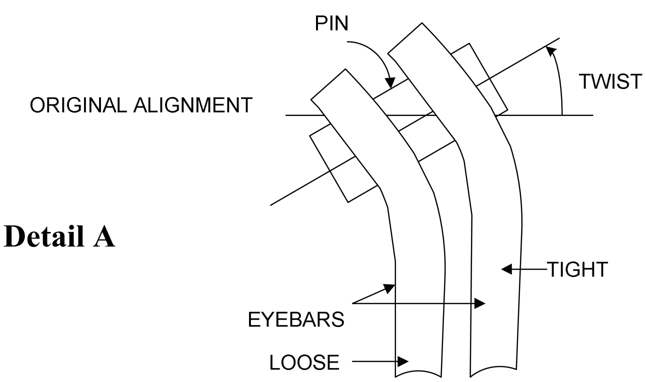


Figure 8.7.25 Eyebar Member with Unequal Load Distribution

Weldments

Evaluate the integrity of any welded repairs to the eyebar (see Figure 8.7.26). Check for any unauthorized welds and include their locations in your report so that the engineer can analyze the severity of their effect on the member (see Figure 8.7.27). Most of these bridges are old and constructed of steel which is considered "unweldable" by today's standards. It is difficult to obtain a high quality "field" weld.

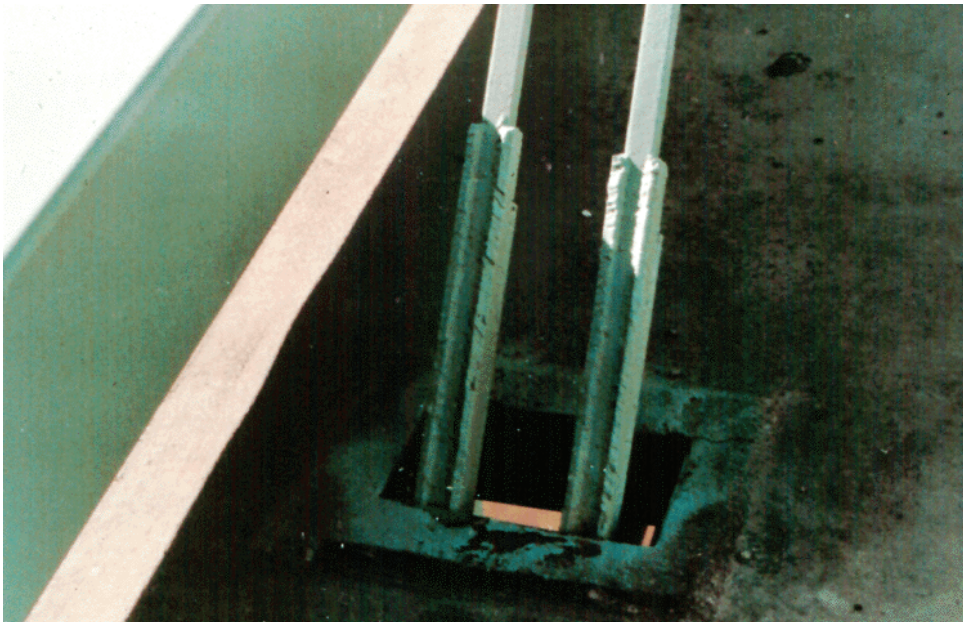


Figure 8.7.26 Welds on Loop Rods

Figure 8.7.27 Welded Repair to Loop Rods

Turnbuckles

Examine any threaded rods in the area of the turnbuckle for corrosion, wear and repairs. Turnbuckles are often located in counter diagonals (see Figures 8.7.28 and 8.7.29).

Figure 8.7.28 Turnbuckle on a Truss Diagonal

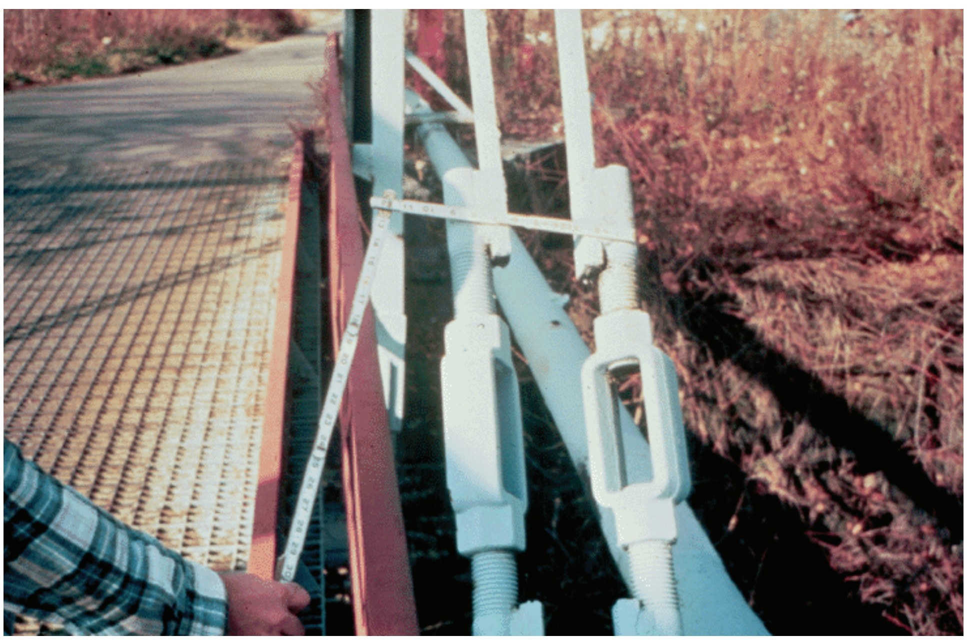


Figure 8.7.29 Welded Repair to Turnbuckles

Pins

Pins should be inspected for signs of wear and corrosion. Nondestructive methods such as ultrasonic inspection are recommended since visual inspection cannot reveal internal material flaws that may exist (see Figure 8.7.30).

Figure 8.7.30 Ultrasonic Inspection of Eyebar Pin

Fracture Critical Members

Eyebars are normally used on truss or suspension bridges. Since these bridge types normally only have two load paths between substructure supports, the bridges are considered non-load path redundant. If a steel eyebar member in tension fails causing a total or partial collapse of the bridge, that eyebar is considered a fracture critical member. Truss members that have three or more eyebars between panel points may be considered internally redundant (see Figures 8.7.31 and 8.7.32). See Topic 8.1 for a detailed discussion on fracture critical members and types of redundancy.

Figure 8.7.31 Fracture Critical Bottom Chord Truss Member: Internally Non-redundant Eyebar

Figure 8.7.32 Fracture Critical Top Chord Truss Member: Internally Redundant Eyebar

8.7.5 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

Under the NBI rating guidelines, the steel eyebars are considered part of the superstructure and do not have an individual rating. The rating for the superstructure should take into account the condition of the steel eyebar assembly and may be lowered due to a deficiency in the steel eyebars. The superstructure is still rated as a whole unit but the steel eyebars 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

Element level evaluation does not have specific CoRe elements for steel eyebars. Due to this fact, individual states may choose to create their own non-CoRe elements or use the AASHTO CoRe elements that "best describe" the steel eyebars. In an element level condition state assessment of steel eyebars, the AASHTO CoRe elements that relate closest to a steel eyebar include:

Element No.	Description
	Truss
121	Thru Truss (Bottom Chord) – Painted Steel
126	Thru Truss (Excluding Bottom Chord) – Painted Steel
131	Deck Truss – Painted Steel
	Cable
147	Cable Coated *(for suspension bridges using eyebars)*

The unit quantity for steel eyebars in truss bridges is meters or feet, and the total length must be distributed among the five available condition states for painted steel depending on the extent and severity of deterioration. The unit quantity for steel eyebars used as cables in suspension bridges is each and must be placed in one of the five available condition states for coated steel cables. 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 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 eyebars 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

Topic 8.8 Steel Arches

8.8.1 Introduction

Arches are a unique form of bridge in that they look like a half circle or ellipse, turned upside down. Arch bridges have been built since Roman times, but steel arch bridges have only been constructed since the late 1800's. Arch bridges generally need strong foundations to resist the large concentrated diagonal loads.

Arches are divided into three types: deck, through, and tied (see Figures 8.8.1, 8.8.2, and 8.8.3).

Figure 8.8.1 Deck Arch Bridge

Figure 8.8.2 Through Arch Bridge

Figure 8.8.3 Tied Arch Bridge

8.8.2 Deck Arch Design Characteristics

General

Arches are considered to be "simple span" because of the basic arch function, even though many bridges of this type consist of multiple arches. The arch reactions, with their massive horizontal thrusts, are diagonally oriented and transmitted to the foundation.

Like its concrete counterpart, the steel open spandrel arch is designed to resist a load combination of axial compression and bending moment. The open spandrel steel arch is considered a deck arch since the roadway is above the arches (see Figure 8.8.4). The area between the arches and the roadway is called the spandrel.

Open spandrel steel arches receive traffic loads through spandrel bents that support a deck and floor system. Steel deck arches can be used in very long spans, measuring up to 518 m (1700 ft).

Figure 8.8.4 Deck Arch

The arch members are called ribs and can be fabricated into I-girders, boxes, or truss shapes. The arches are classified as either solid ribbed, braced ribbed, or spandrel braced (see Figures 8.8.5 and 8.8.6). The members are fabricated using riveted, bolted, or welded techniques. Most steel deck arches have two arch rib members, although some structures have three or more ribs (see figure 8.8.7).

Figure 8.8.5 Solid Ribbed Deck Arch

Figure 8.8.6 Braced Rib Deck Arch, New River Gorge, WV

Figure 8.8.7 Spandrel Braced Deck Arch with Six Arch Ribs

An arch with a pin at each end of the arch is called a two-hinged arch (see Figure 8.8.8). If there is also a pin at the crown, or top, of the arch, it is a three-hinged arch. One-hinged and fixed arches may exist, although these are very rare. Foundation conditions, in part, dictate the requirements for hinges. Three-hinged arches, for example, are not significantly affected by small foundation movements.

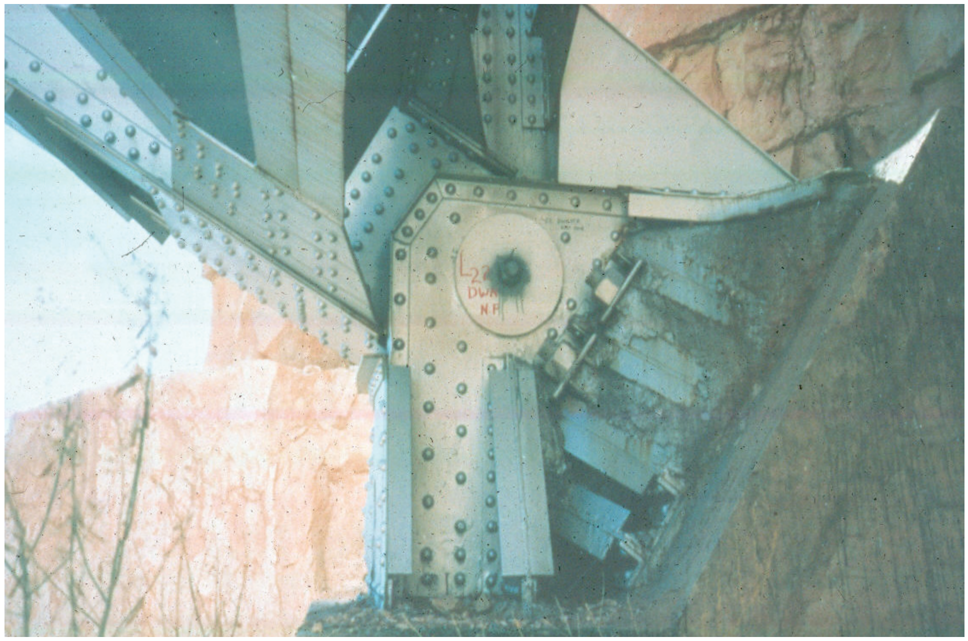

Figure 8.8.8 Hinge Pin at Skewback for Spandrel Braced Deck Arch (Navajo Bridge)

Primary and Secondary Members

The primary members of a deck arch bridge consist of the arches or ribs, spandrel columns or bents, spandrel girders and the floor system. The floor system consists of floorbeams and stringers (if present) (see Figure 8.8.9).

The secondary members of a deck arch bridge consist of the sway bracing and the upper lateral and lower lateral bracing of the arch or floor system (see Figure 8.8.10).

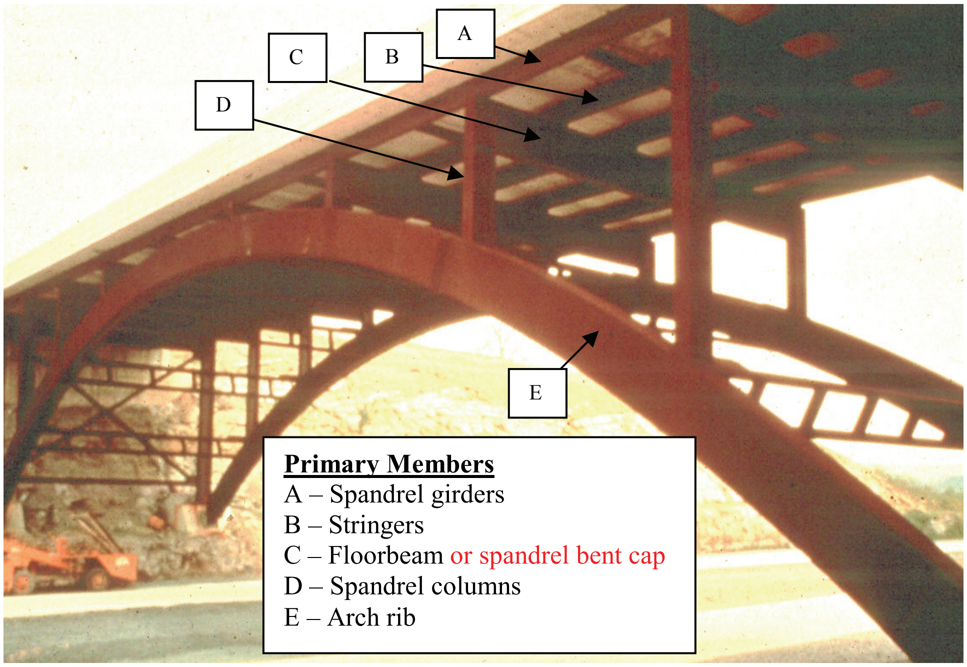

Figure 8.8.9 Solid Ribbed Deck Arch Primary Members

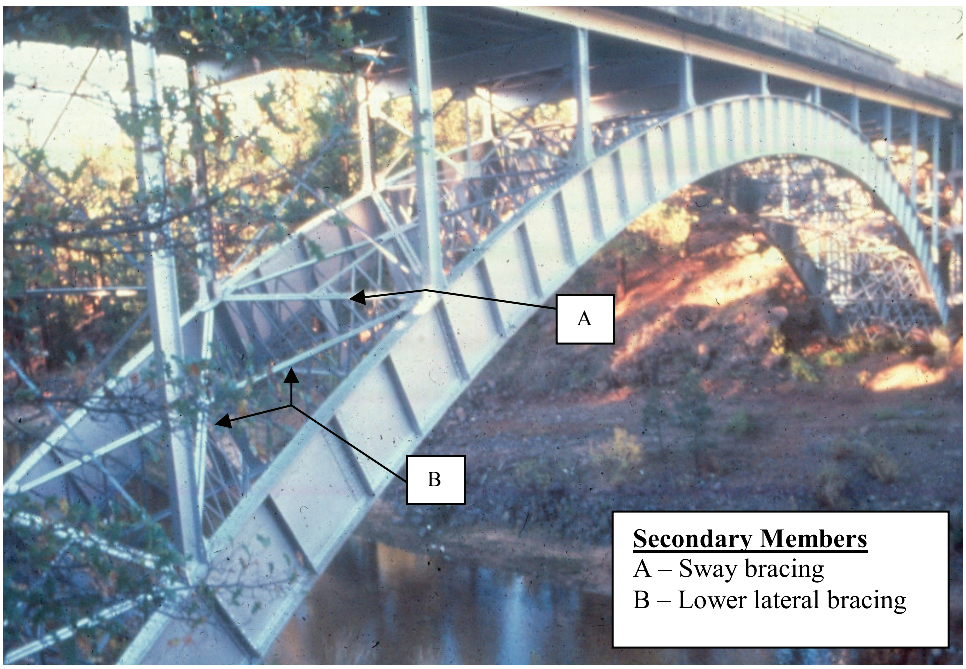

Figure 8.8.10 Solid Ribbed Deck Arch Secondary Members

Load Transfer

Traffic loads are supported by a deck. The load from the deck is transmitted to the stringers (if present) and then the floorbeams. The stringer and floorbeams resist the traffic load in bending and shear. The load is transferred to the spandrel bents and spandrel columns, which are in compression or bending. The arch supports the spandrel column and transfers the compressive load to the ground at the supports.

Fracture Critical Members

The deck arch bridge has two or more main members. However, the arch is not a tension member and is therefore not considered fracture critical. Some members of the floor system and spandrel bent may be considered fracture critical (see Topic 8.3)

8.8.3 Through Arch Design Characteristics

General

Arch bridges are considered simple spans because of the basic arch function, even though many bridges of this type consist of multiple arches. Through arches are typically two or three hinged. The arch reactions, with their massive horizontal thrusts, are diagonally oriented and transmitted to the foundations.

The steel through arch is constructed with the crown of the arch above the roadway and the arch foundations below the roadway (see Figure 8.8.11). The deck is hung from the arch by wire rope cables or eyebars.

Figure 8.8.11 Elevation View of a Braced Ribbed Through Arch

The arch members are called ribs and are usually fabricated box-type members. Steel through arches are known as either solid ribbed or braced ribbed. The solid ribbed arch, which can be any type of arch, has a single curve defining the arch shape, while the braced ribbed arch has two curves defining the arch shape, braced with truss webbing between the curves. The lower curve is the bottom rib chord, and the upper curve is the top rib chord. The rib chord bracing consists of posts and diagonals. The braced ribbed arch is sometimes referred to as a trussed arch and is more common than the solid ribbed through arch.

Primary and Secondary Members

The primary members of a through arch bridge consist of arch ribs (consisting of top and bottom rib chords and rib chord bracing), rib chord bracing, hangers and floor system including floorbeams and stringers (if present) (see Figure 8.8.12).

The secondary members of a through arch bridge consist of sway bracing, lateral bracing (top and bottom rib chords and floor system) (see Figure 8.8.13).

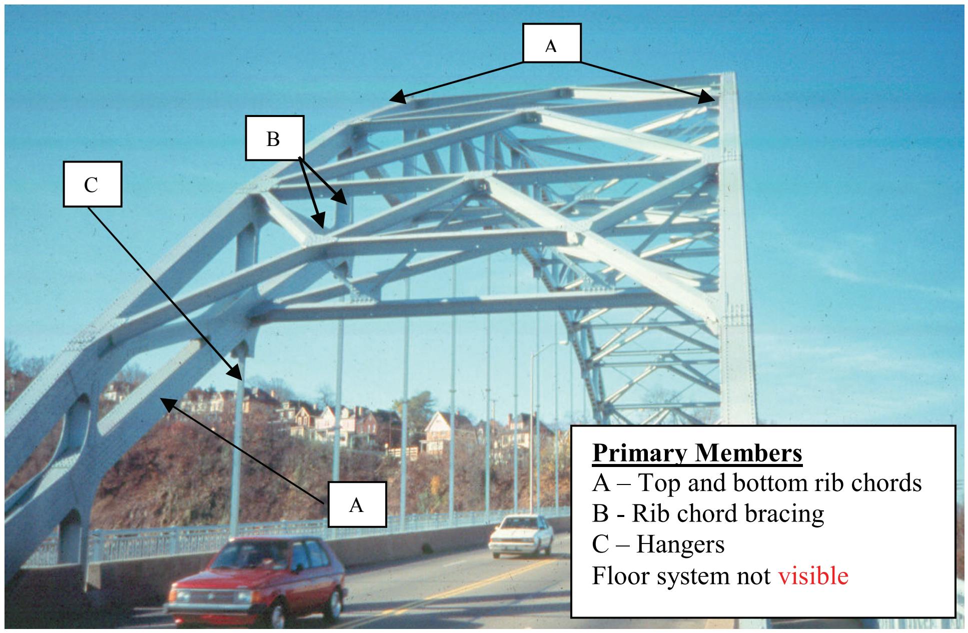

Figure 8.8.12 Through Arch Primary Members

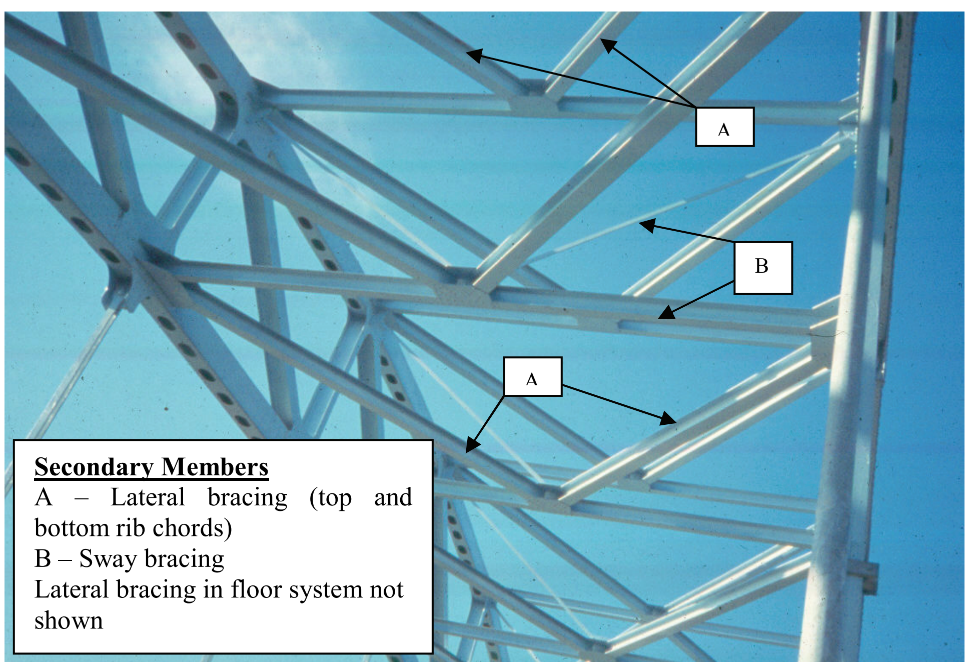

Figure 8.8.13 Through Arch Bracing (Secondary Members)

Load Transfer

Traffic loads are supported by a deck. The load from the deck is transmitted to the stringers (if present) and then the floorbeams. The stringer and floorbeams resist the load in bending and shear. The load is transferred to the hangers, which are in tension. The arch supports the hangers and transfers the compressive load to the ground at the supports.

Fracture Critical Members

The through arch is the main load-carrying member. Since there are typically only two arch ribs, the structure is nonredundant. However, the bridge is not classified as fracture critical because the arches are not tension members. The hangers may be fracture critical, depending on the results of a detailed structural analysis. Some members of the floor system may be fracture critical (see Topic 8.3).

8.8.4 Tied Arch Design Characteristics

General

The tied arch is a variation of the through arch with one significant difference. In a through arch, the horizontal thrust of the arch reactions is transferred to large rock, masonry, or concrete foundations. A tied arch transfers the horizontal reactions through a horizontal tie which connects the ends of the arch together, like the string on an archer's bow (see Figure 8.8.14). The tie is a tension member. If the string of a bow is cut, the bow will spring open. Similarly, if the arch tie fails, the arch will lose its compression and will collapse.

Design plans are generally needed to differentiate between through arches and tied arches. Another guide in correctly labeling through and tied arches is by examining the piers. Since tied arch bridges redistribute the horizontal loads to the tie girders, the piers for tie arch bridges are smaller than the piers for through arch bridges.

Figure 8.8.14 Tied Arch

Arch members are fabricated with either solid rib members, box members or braced ribs.

The tie member is a fabricated box member or consists of truss members. The tie is also supported by hangers, which usually consist of wire rope cable, but can also be eyebars or built-up members.

Primary and Secondary Members

The primary members of a tied arch bridge consist of arch ribs, tie members, rib bracing truss (if present), hangers, and floor system including floorbeams and stringers (if present) (see Figure 8.8.15).

The secondary members of a tied arch bridge consist of sway bracing, lateral bracing (arch rib, top chord and floor system) (see Figure 8.8.16).

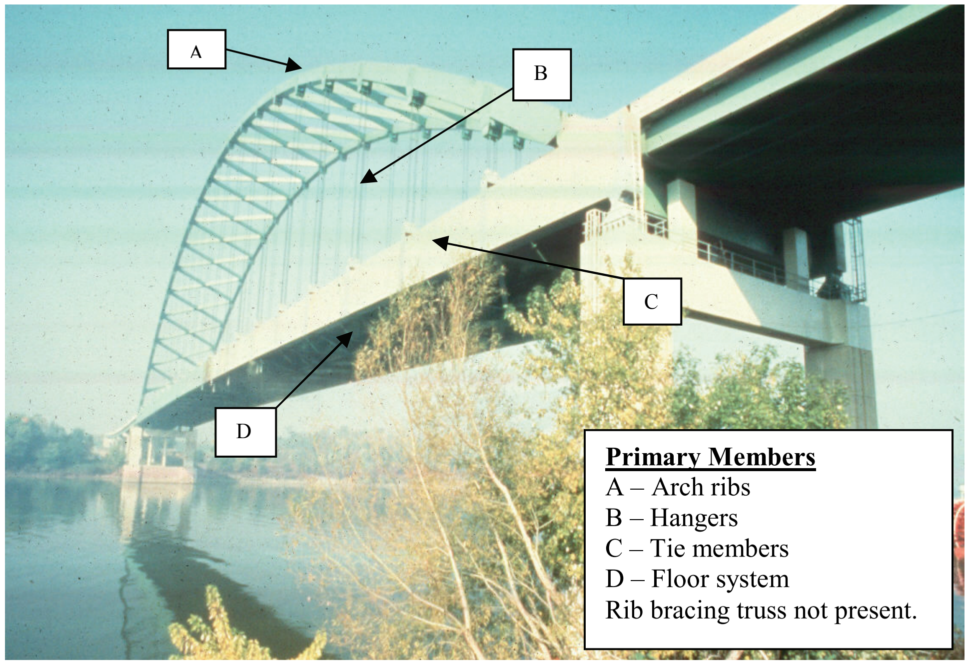

Figure 8.8.15 Tied Arch Primary Members

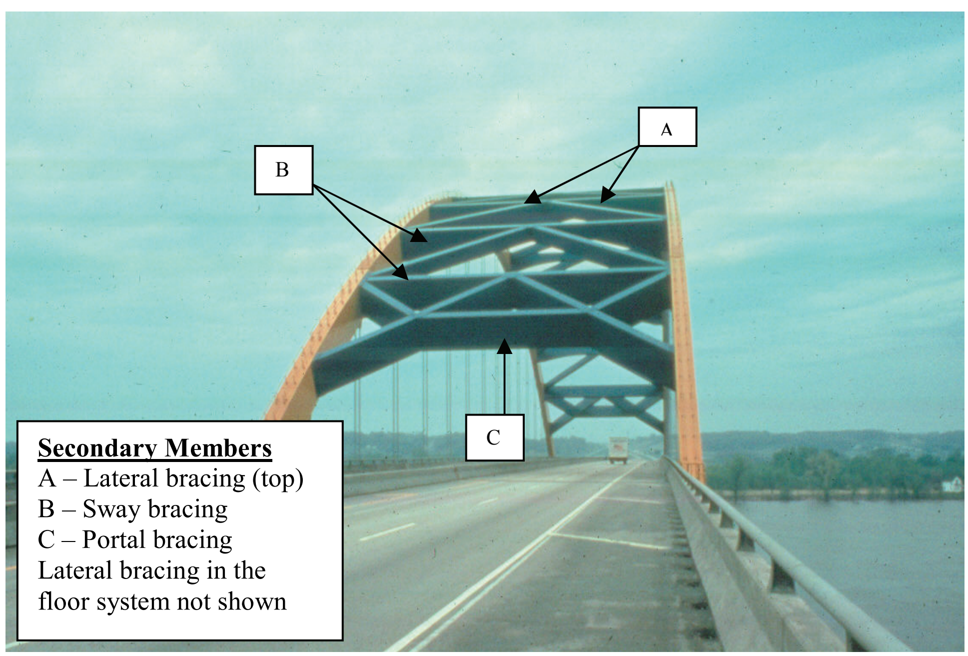

Figure 8.8.16 Tied Arch Secondary Members

Load Transfer

Traffic loads are supported by a deck. The load from the deck is transmitted to the stringers (if present) and then the floorbeams. The stringer and floorbeams resist the load in bending and shear. The load is transferred to the hangers, which are in tension. The arch supports the hangers and transfers the compressive load to the tie girder and the supports.

Fracture Critical Members

With only two load paths, arches are considered non-redundant structures. The arches are not fracture critical since they are subjected to axial compression. The tie girders, on the other hand, are axial tension members and are considered fracture critical.

8.8.5 Overview of Common Defects

Common defects that occur on steel arch 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.

8.8.6 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

Locations

Bearing Area

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 on each of the supports 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).

Arch Members

Inspect the alignment of the arch and look for signs of buckling and crippling in the arch ribs. Check for general corrosion and deterioration Examine any pins for corrosion and wear. Check the arch rib splice plates and the connections for hangers or spandrel bents.

Inspect steel arch girder type bridges as described in Topic 8.2. Inspect steel arch box girder type bridges as described in Topic 8.5. Inspect steel braced ribbed arch type bridges as described in Topic 8.6 (see Figure 8.8.17).

Figure 8.8.17 Through Truss Arch Members

Bracing (Through and Tied Arches)

Inspect the web members (posts and diagonals) in a manner similar to any other truss as described in Topic 8.6. Depending on the truss design (e.g., Pratt or Warren, etc.), the web members will either be designed for tension, compression, or both.

Spandrel Members (Deck Arch)

Examine the end connections of the spandrel bents, spandrel columns and spandrel girders for cracks and loose fasteners. Check the spandrel girders/caps/columns for flexure, section loss, and buckling damage (see Figure 8.8.18).

Figure 8.8.18 Solid Ribbed Deck Arch Showing Spandrel Columns

Hangers (Through and Tied Arches)

Check the connections at both ends of the hangers, and look for corrosion and cracks. Examine the alignment of the hangers; the hangers may be near traffic, so inspect for collision or fire damage (see Figures 8.8.19 and 8.8.20). Check the hangers for any welded attachment; examine the welds between the attachment and the hanger for cracks

Figure 8.8.19 Hanger Connection on a Through Arch

Figure 8.8.20 Performing Baseline Hardness Test on Fire Damaged Arch Cables

Floor System

The floor system, consisting of floorbeams and possibly stringers, should be inspected in the same manner as previously described in Topic 8.3, Steel Two-girder Systems & Steel Through Girder Systems (see Figure 8.8.21).

Tied Arches

Tied arches are subjected to axial compression in addition to bending caused by the hanger connections. Check floorbeam to tied member connection for distortion caused by fatigue or horizontal floorbeam displacement in the webs of the floorbeams when the stringers are placed above the floorbeams.

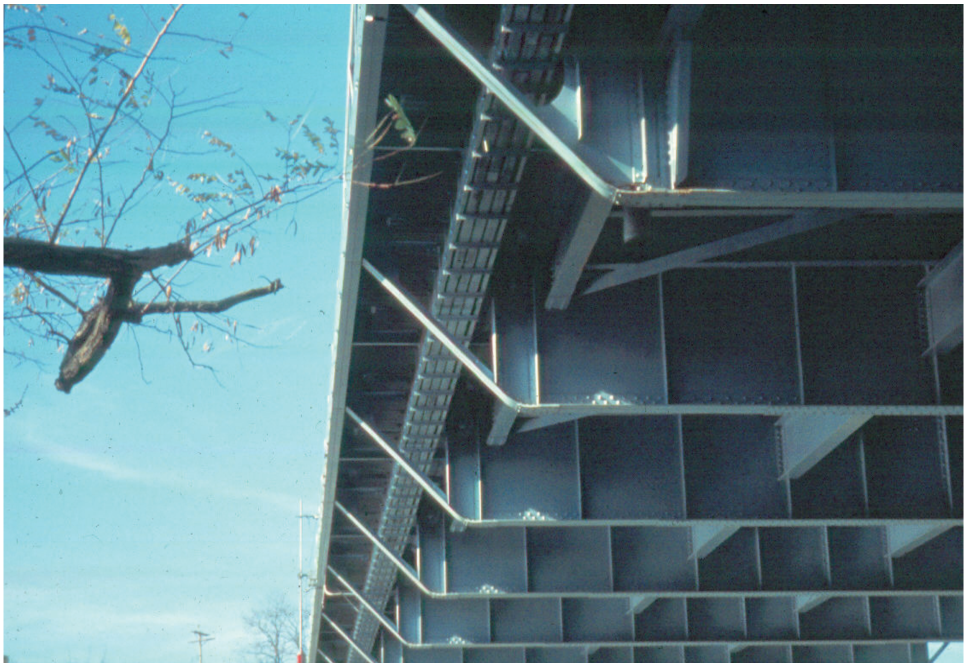

Figure 8.8.21 Floor System on a Through Arch

Tied Girder

Determine if back-up bars were used to make corner welds in the tie box. If so, the back-up bars should be carefully examined to determine if they are continuous. Weld cracks can occur at points where the bars are discontinuous. Check all welds, and examine the ends of any cover plates. Especially check welds connecting internal diaphragms to the tie box. Inspect the floorbeam connections and the corner welds of the tie box (see Figure 8.8.22).

Fatigue Prone Details

Check the welds on tension members such as tie girders, floorbeams or stringers. Look at welds for stiffeners/diaphragms/connection plates.

If cover plates are present, look at the welded ends for cracking. Back-up bars may be used between the webs and flanges and are susceptible to cracking.

Check the web gap area between the flanges and the connection plates. Closely inspect tack welds, intersecting welds and discontinuous or intermittent welds.

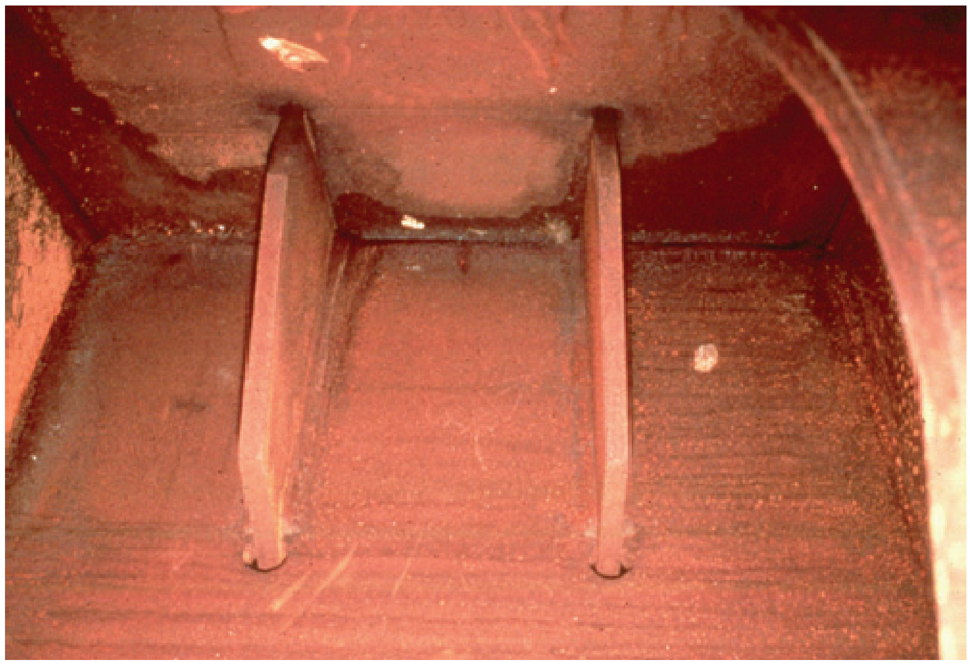

Figure 8.8.22 Tie Girder Interior

Out-of-plane Distortion

Investigate the girders at the floorbeam connection for cracks in the webs due to out-of-plane distortion. Investigate for fatigue cracks due to web-gap distortion. This is a major source of cracking when floorbeams frame into girders. For additional information on out-of-plane distortion, see Topic 8.3.

Out-of-plane distortion can also crack the tie girder webs at partial depth diaphragm connection with floorbeams (see Figure 8.8.23). The diaphragms stiffen the tie girders. However, the small, unstiffened regions of the webs do not have sufficient rigidity against lateral, out-of-plane distortion. The situation is analogous to that of webs at unattached ends of floorbeam or diaphragm connection plates. Fatigue cracks can develop at the ends of the diaphragm plates. The ends of all partial depth diaphragms in a box tie girder should be closely inspected.

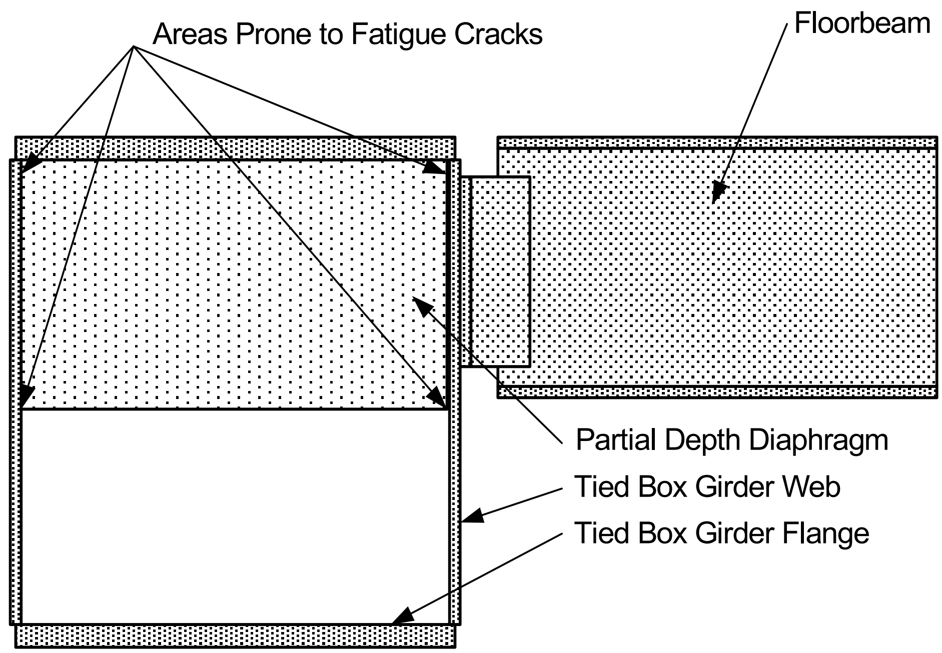

Figure 8.8.23 Partial Depth Diaphragm in a Tied Box Girder

Secondary Members

The secondary members should be inspected using methods similar to those detailed in Topics 8.3, 8.5, and 8.6 (see Figure 8.8.24). In addition:

Investigate the alignment of the bracing elements. Check horizontal connection plates, which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Examine the end connections for cracks, corrosion, and loose fasteners.

Misalignment of secondary members may be an indication of differential structure movement or substructure settlement.

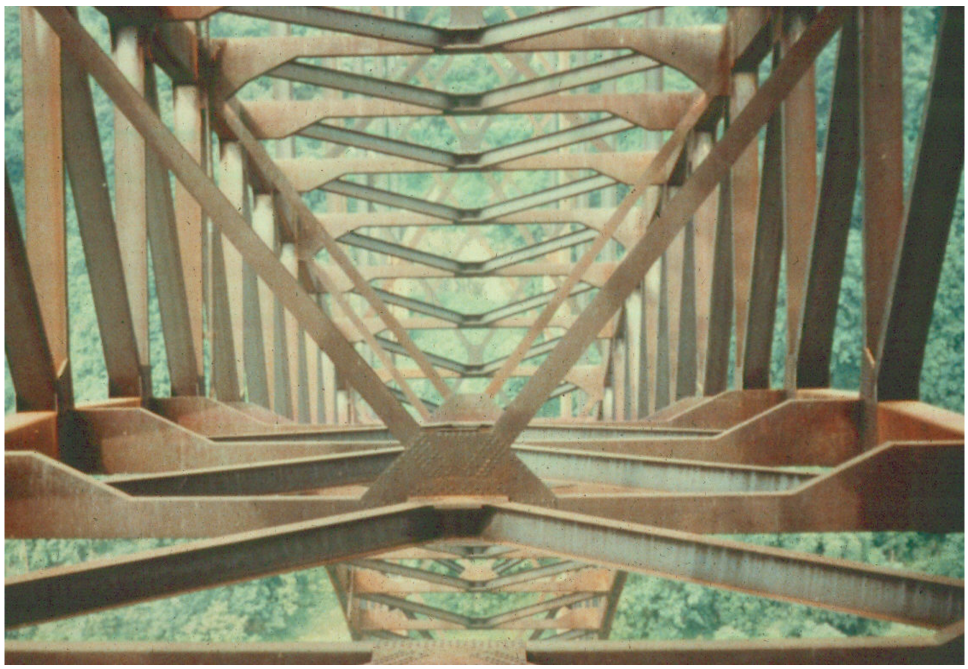

Figure 8.8.24 Bracing Members (New River Gorge Bridge)

Areas Exposed to Traffic

Inspect any areas exposed to traffic for collision damage (see Figure 8.8.25). If collision damage is found, document the location and dimensions and reference with photographs or sketches.

Figure 8.8.25 Areas Exposed to Traffic

Areas Exposed to Drainage

The areas that trap water and debris result in active corrosion cells, which cause loss of section and notches susceptible to fatigue. On arch bridges check lateral bracing gusset plates, pockets created by floor system connections, and areas exposed to drainage runoff.

8.8.7 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 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 arch bridge, the AASHTO CoRe elements may be:

Element No.	Description
	Box Girder
101	Unpainted Steel Closed Web/Box Girder
102	Painted Steel Web/Box Girder
	Floor System
106	Unpainted Steel Open Girder/Beam
107	Painted Steel Open Girder/Beam
112	Unpainted Steel Stringer (Stringer Floorbeam System)
113	Painted Steel Stringer (Stringer Floorbeam System)
	Steel Arch
140	Unpainted Steel Arch
141	Painted Steel Arch
	Hanger
146	Cable (not embedded in concrete) Uncoated
147	Cable (not embedded in concrete) Coated
	Floor System
151	Unpainted Steel Floorbeam
152	Painted Steel Floorbeam
	Pin and Hanger
160	Unpainted Steel Pin and Hanger Assembly
161	Painted Steel Pin and Hanger Assembly
	Spandrel Columns
201	Unpainted Steel Columns
202	Painted Steel Columns

The unit quantity for the arch is meters or feet and the total length of the arch ribs 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. The unit quantity for the floor system is meters or feet and the total length of floor beams and stringers must be distributed among the 4 or 5 available condition states. The unit quantity for columns, cables or hanger assemblies is each and the total quantity must be placed in one of the four available condition states for unpainted and five available condition states for painted. 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 rust 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 steel arches 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

Section 8
Inspection and Evaluation of Common Steel Superstructures

Topic 8.9 Steel Rigid Frames

8.9.1 Introduction

A frame is a multi-sided configuration in which the sides are rigidly connected in such a fashion that applied loads are distributed to each side (see Figure 8.9.1). Steel rigid frames are popular today in building construction because of their space-saving characteristics. The same principles that permit the omission of intermediate column supports in buildings are applied to bridge frames. In a steel rigid frame bridge structure, the frame sides or "legs" replace intermediate supports. Because the legs contribute to the structures overall capacity, increased span lengths and material savings can be realized.

Figure 8.9.1 Typical Rigid K-frame Constructed of Two Frames

8.9.2 Design Characteristics

General

Frames are not referred to as having a single, simple, multiple, or continuous spans. Also, steel rigid frame structures are used only in straight horizontal applications.

Steel rigid frame bridges typically consist of welded plate girder construction with bolted field splices in low stress areas and welded stiffeners in high stress areas. The frames are spaced from about 2 to 6 m (7 to 20 feet) on centers, depending on loads, span lengths, and type of floor system. Steel rigid frames can be economical for spans from 15 m (50 feet) to over 61 m (200 feet). Standard abutments and expansion bearings support the ends of the frame girders.

The superstructure of a rigid frame bridge can be constructed of two frames similar to a two-girder bridge (see Figure 8.9.1) or of multiple frames in the same manner as a multi-girder bridge (see Figure 8.9.2). These frames can be thought of as fabricated girders with legs.

Figure 8.9.2 Typical Rigid Frame Constructed of Multiple Frames

K – Frames

Most steel rigid frame bridges are multi-span structures and are commonly referred to as "K-frame" or "grasshopper leg" bridges (see Figure 8.9.3). The sloping legs give the rigid frame a "K" shape, when looked at by rotating the frame counterclockwise 90°. K-frames are not economical for very short or very long span bridges. Because of their aesthetically pleasing appearance, sometimes an effort is made to use steel rigid frames whenever possible.

Figure 8.9.3 Typical K-frame

It is possible to think that the legs of the K-frame look very much like piers and consider them part of the substructure. This is not the case because there is no bearing between the legs and the girder portion of the frame (see Figure 8.9.3).

Since there are no bearings between the legs and girder portion of the frame, bending forces are transferred between the girder portion and the legs (see Figure 8.9.4).

Figure 8.9.4 Connection Between Legs and Girder Portion

Delta Frames

In some designs, a triangular frame configuration can be used. For very long spans, two K-frames can be connected together end-to-end (see Figure 8.9.5). Instead of one of the end spans bearing on an abutment, it is connected to the end span of another K-frame. The bottoms of the legs are also connected together and share the same bearing. This type of configuration is known as a delta frame. The leg connections form an inverted triangle with the girder portion of the frame. The Greek letter Delta (∇) is the symbol used for this triangle.

Figure 8.9.5 Delta Frame

Regardless of the frame configuration, the entire portion of the bridge, (legs and girders) constitutes the frame, and is considered the superstructure. The legs of rigid frames are supported by relatively small concrete footings and bearings which are essentially hinges (see Figure 8.9.6).

Figure 8.9.6 Bearing

Stiffeners

Steel rigid frames may have up to three different types of stiffeners (see Figure 8.9.7).

Transverse Stiffeners

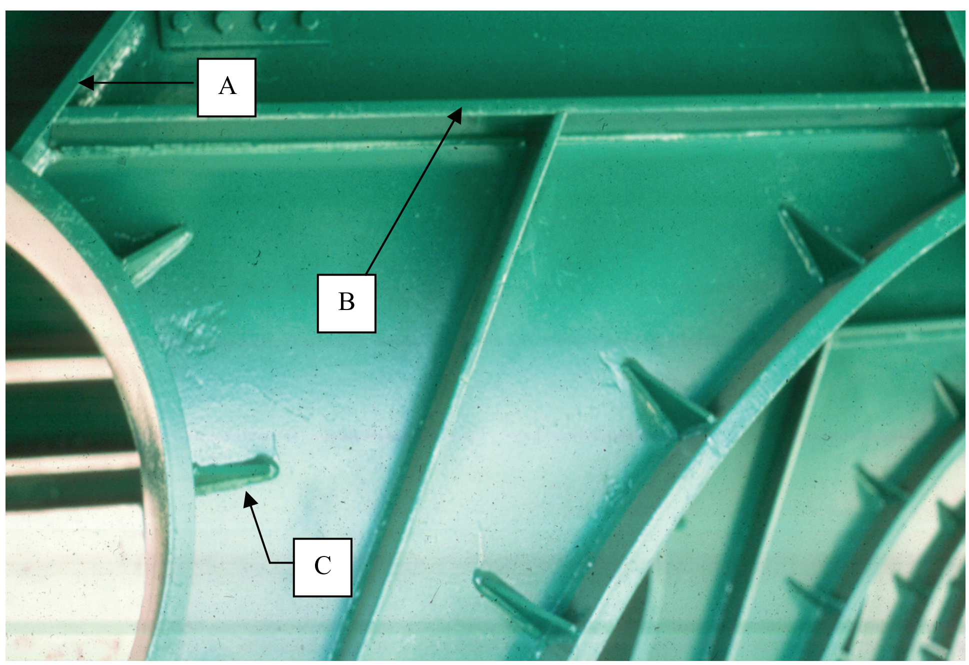
Transverse stiffeners are placed approximately perpendicular to the flanges and welded to the web and flanges of the frame at spacings required by design. Transverse stiffeners are used to prevent buckling in high shear regions.

Longitudinal Stiffeners

Longitudinal stiffeners are placed parallel to the flanges and welded to the web of the frame. They may extend the entire length of the frame girder or just in areas of high moment. Longitudinal stiffeners resist web buckling in the compression zone and therefore are closer to the top flange in areas of higher positive moment and closer to the bottom flange in areas of higher negative moment.

Radial Stiffeners

Radial stiffeners are placed perpendicular along the frame knee bottom flange radius. The radial stiffeners are welded to the flange and web at spacings required by design. This type of stiffener is used to resist shear and moment forces in the knee.



Stiffeners
A – Transverse
B – Longitudinal
C – Radial

Figure 8.9.7 Transverse, Longitudinal, and Radial Stiffeners on a Frame Knee

Floor System Arrangement

A rigid frame will have one of three floor systems:

Multiple Frame System

For a multiple frame system, the deck is supported only by the frames (see Figure 8.9.2).

Frame-Floorbeam System

Floorbeams are connected to the girder portion of the two frames. The floorbeams are much smaller than the girder portion of the frame and are perpendicular to the flow of traffic. The deck is supported by the floorbeams, which in turn transmit the loads to the frames. The floorbeams can be either rolled beams, fabricated girders, or fabricated cross frames.

Frame-Floorbeam-Stringer System

Longitudinal stringers, parallel to the frames, are connected to the floorbeams (see Figure 8.9.8). Floorbeams are connected to the girder portion of the two frames. The stringers are typically rolled sections and are connected to the web of the floorbeams.

Figure 8.9.8 Two Frame Bridge with Floorbeam-Stringer Floor System

Primary and Secondary Members

For steel rigid frame bridges, the primary members are the frames as a whole, including floorbeams and stringers (if present) (see Figure 8.9.8). However, for ease of discussion, the frame is commonly broken down into the following five elements:

- Frame girder - the horizontal sections
- Frame leg - the inclined sections
- Frame knee - the intersection between the frame girder and frame leg
- Floorbeams (if present)
- Stringers (if present)

The primary members consist of the frames and floor system. The frame girder, frame knee and frame leg make up the frame. If present, the floor system consists of floorbeams and stringers (see Figure 8.9.9).

Secondary members consist of lateral bracing, sway bracing and diaphragms.

In a two frame system, lateral bracing members are placed diagonally between the frames. This bracing restricts any differential and longitudinal movements between the frames. This bracing is in the plane of the bottom flange of the girder portion of the frame or between the legs of the frame (see Figure 8.9.10).

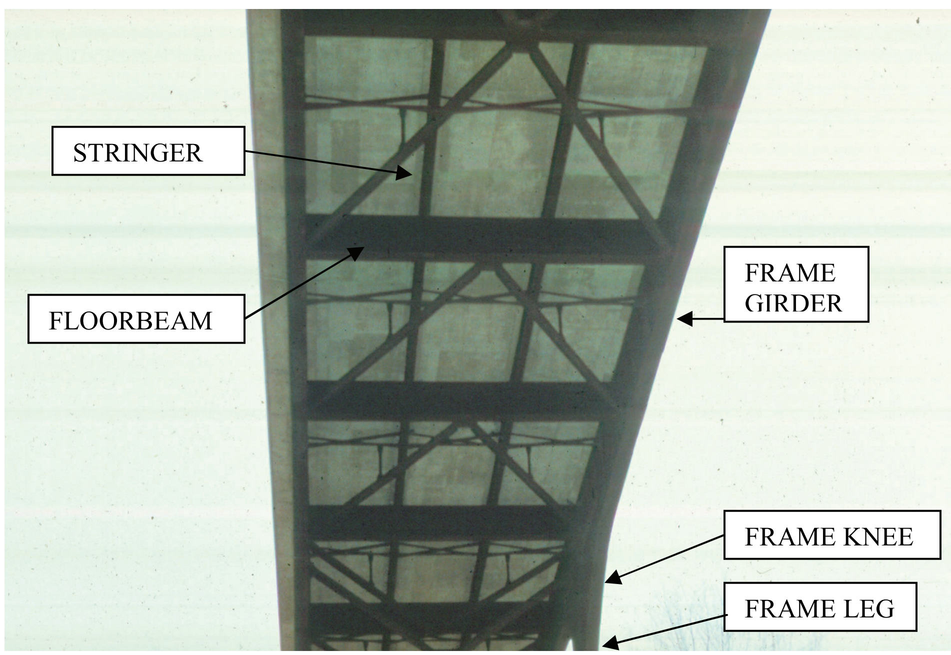


Figure 8.9.9 Frame Members, Floorbeams, and Stringers

In a two frame system, sway bracing is placed between the leg portions of the frame (see Figure 8.9.10). In a multiple frame system diaphragms are placed perpendicular between the frames. The sway bracing and diaphragms minimize any transverse movements of the frames (see Figure 8.9.11).

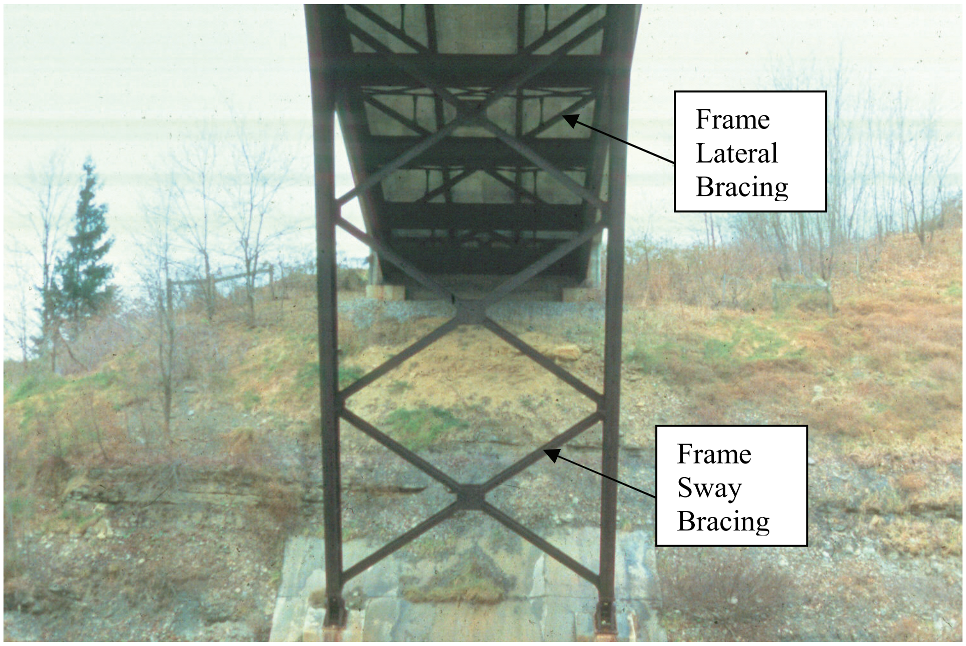


Figure 8.9.10 Lateral Bracing for the Frame Legs

Figure 8.9.11 Lateral Bracing and Diaphragms

Stress Zones

Each element of the frame resists various levels of stress due to moment and shear. Tension zones are similar to those for concrete rigid frames (see Figure 8.9.12).

Stress zones for the floor systems are similar to the two-girder floor systems discussed in Topic 8.3.

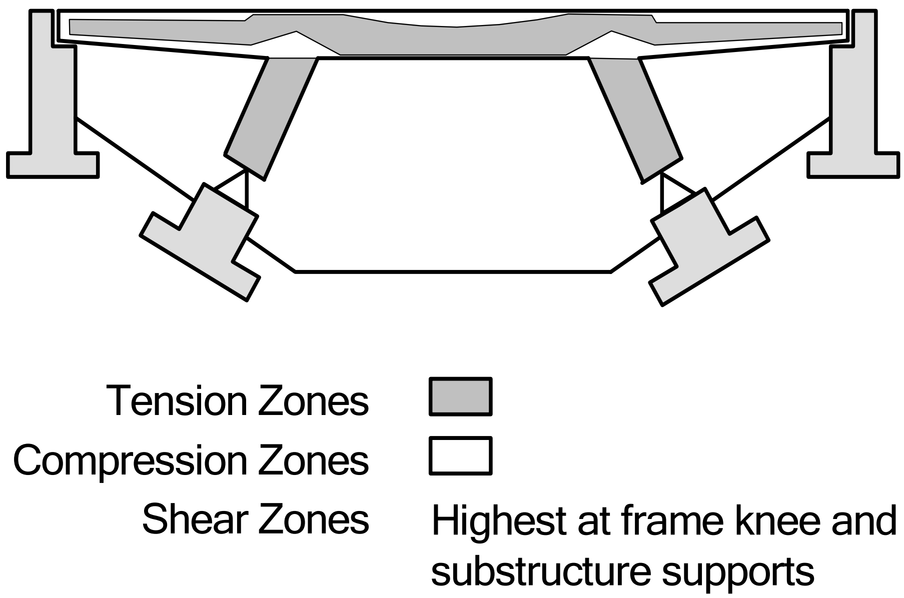


Figure 8.9.12 Stress Zones in a Frame

Fatigue Prone Details and Failure

Some common areas for fatigue prone details are:

- Fabrication welds
- 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 in Topic 8.9.4.

Fracture Critical Areas

A rigid frame consisting of two frames has no load path redundancy. This means that a two frame steel rigid frame is a fracture critical bridge type (see Figure 8.9.13).

Figure 8.9.13 Dual Frame Rigid Frame – A Fracture Critical Structure

A rigid frame bridge consisting of three or more frames has load path redundancy and is not fracture critical (see Figure 8.9.14).

Figure 8.9.14 Multiple Frame Rigid Frame – Not a Fracture Critical Structure

8.9.3 Overview of Common Defects

Common defects that occur on steel rigid frame bridges include:

- 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.

8.9.4 Inspection Procedures and Locations

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

Procedures

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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 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 floor system, frame knee area and 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 figure 8.9.15). See Topic 9.1 for a detailed presentation on the inspection of bearings.

Figure 8.9.15 Bearing Area of a Two Frame Bridge

Shear Zones

Examine the web area of the girder portion near the bearings and knee areas for section loss due to corrosion. Check the web area of the girder portion near the bearings and knee areas for buckling. Inspect floorbeams and stringers (if present) near their respective bearing areas for corrosion or buckling. Check the bottom of the frame legs for corrosion or buckling.

Flexure Zones

Check the tension and compression flanges for corrosion, section loss, cracks or buckling (see Figure 8.9.16). Special attention should be given to the flanges at the connection between the legs and girder portion of the beam. Bending moment is at its highest in this area.

Figure 8.9.16 Flexural Zones

Secondary Members

Check horizontal connection plates which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Investigate the areas beneath drainpipes and deck joints for corrosion from exposure to roadway drainage. Examine the connection areas of the lateral bracing or diaphragms for cracked welds, fatigue cracks, and loose fasteners. Check for distortion in the secondary members. Distorted secondary members may be an indication the primary members are overstressed of 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 can cause in notches susceptible to fatigue or perforation and loss of section. On rigid frame bridges check horizontal surfaces that include top of bottom flange, lateral bracing gusset plates and pockets created by floor system connections.

Areas Exposed to Traffic

Check underneath the bridge for collision damage to the frame sections and bracing if the bridge crosses over a highway, railway, or navigable channel. Document any cracks, section loss, scrapes or distortion found.

Fatigue Prone Details

Examine any attachment welds located in the tension areas of the frame or floor system. Check web stiffener welds. Pay close attention to the knee area. Vertical, longitudinal, and radial stiffeners may all be present, and there may be a possibility of intersecting welds (see Figure 8.9.17). Inspect welded flange splices, particularly where changes in thickness and/or width occur (see Figure 8.9.18).

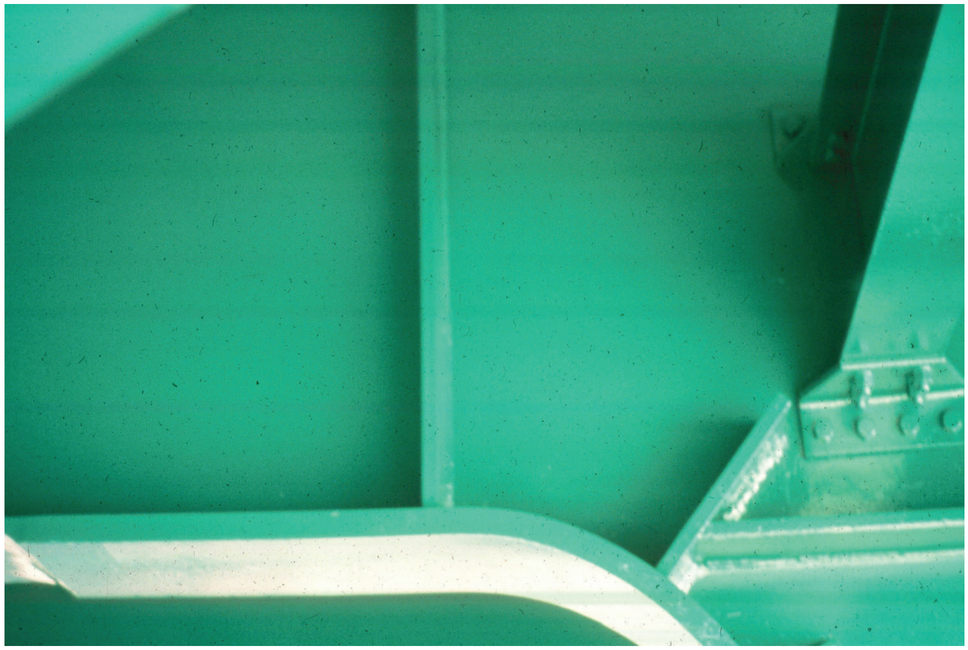

Figure 8.9.17 Knee Area: Fatigue Prone Details

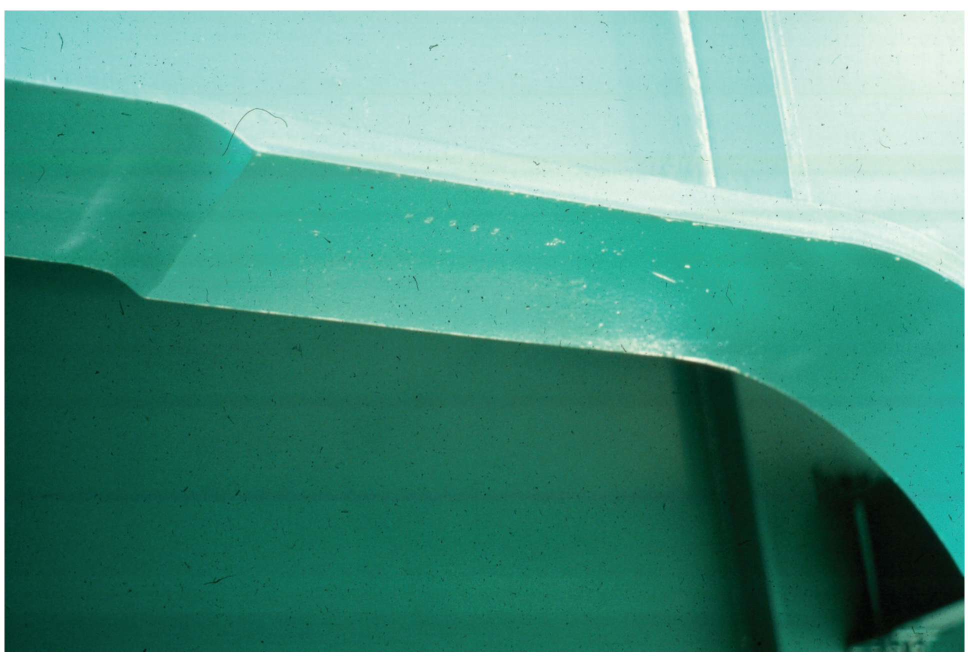

Figure 8.9.18 Welded Flange Splice

8.9.5 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 NBIS Rating Guidelines.

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

Element Level Condition State Assessment

The element level method does not have specific CoRe elements for steel rigid frames. Due to this fact, individual states may choose to create their own non-CoRe elements or use the AASHTO CoRe elements that "best describe" the rigid frame. In an element level condition state assessment of a steel rigid frame bridge, the AASHTO CoRe elements that relate closest to a rigid frame include:

Element No.	Description
106	Unpainted Steel Open Girder/Beam
107	Painted Steel Open Girder/Beam
112	Unpainted Steel Stringer (Stringer Floorbeam System)
113	Painted Steel Stringer (Stringer Floorbeam System)
151	Unpainted Steel Floorbeam
152	Painted Steel Floorbeam
201	Unpainted Steel Column or Pile Extension
202	Painted Steel Column or Pile Extension

The unit quantity for the rigid frame 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. The unit quantity for columns is each and the total quantity must be placed in one of the four available condition states for unpainted and five available condition states for painted. 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 states that create their own non-CoRe elements, the bridge inspector must use that particular state's Bridge Inspection Manual to determine the appropriate non-CoRe element(s) as well as the correct condition state(s).

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 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 steel rigid frames 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.

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 steel rigid frames 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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