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Section 8

Inspection and Evaluation of Common Steel Superstructures

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

8.2.1 Introduction

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

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

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

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

8.2.2 Design Characteristics

Rolled Multi-beam

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

Figure 8.2.1 Simple Span Rolled Multi-beam Bridge

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

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

Figure 8.2.3 Rolled Multi-beam Bridge with a Cover Plate

Fabricated Multi-girder

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

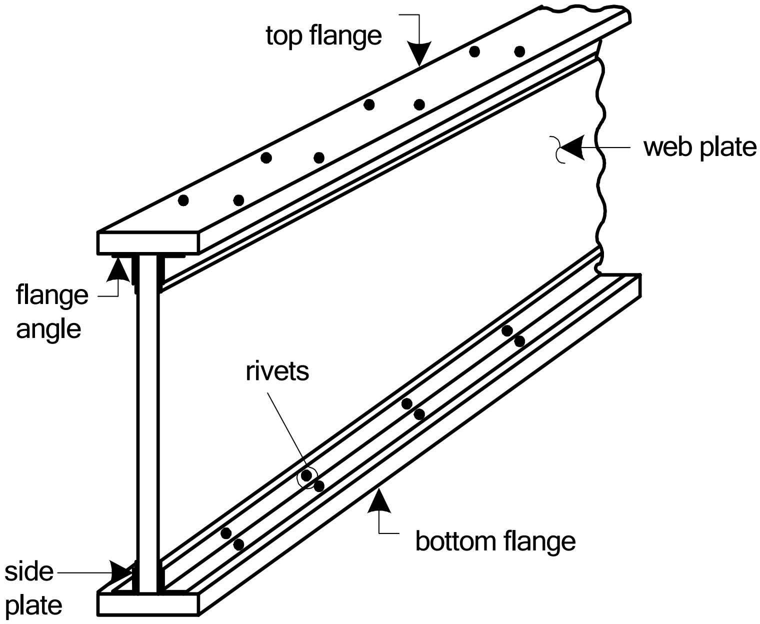

Figure 8.2.4 Built-up Riveted Plate Girder

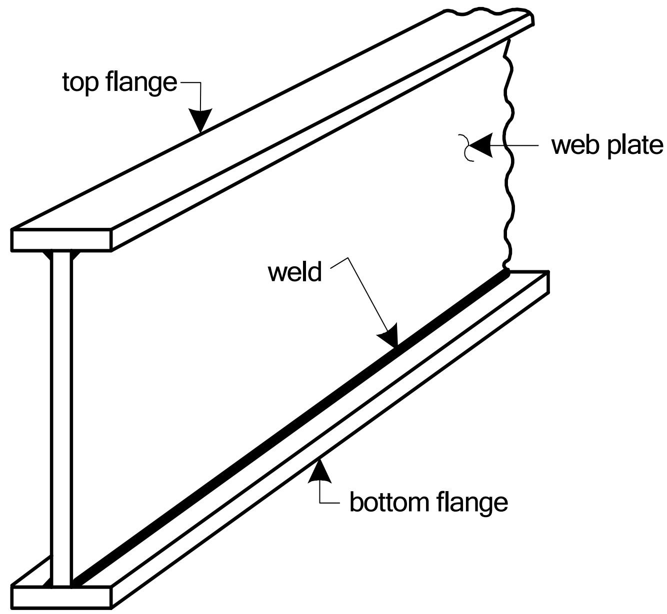

Figure 8.2.5 Welded Plate Girder

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

Figure 8.2.6 Single Span Fabricated Multi-girder Bridge

Figure 8.2.7 Continuous Span Fabricated Multi-girder Bridge

Figure 8.2.8 Curved Fabricated Multi-girder Bridge

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

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

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

Figure 8.2.10 Combination Rolled Beams and Fabricated Girders

Haunched Girder Design

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

Three methods have been used to haunch girders.

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

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

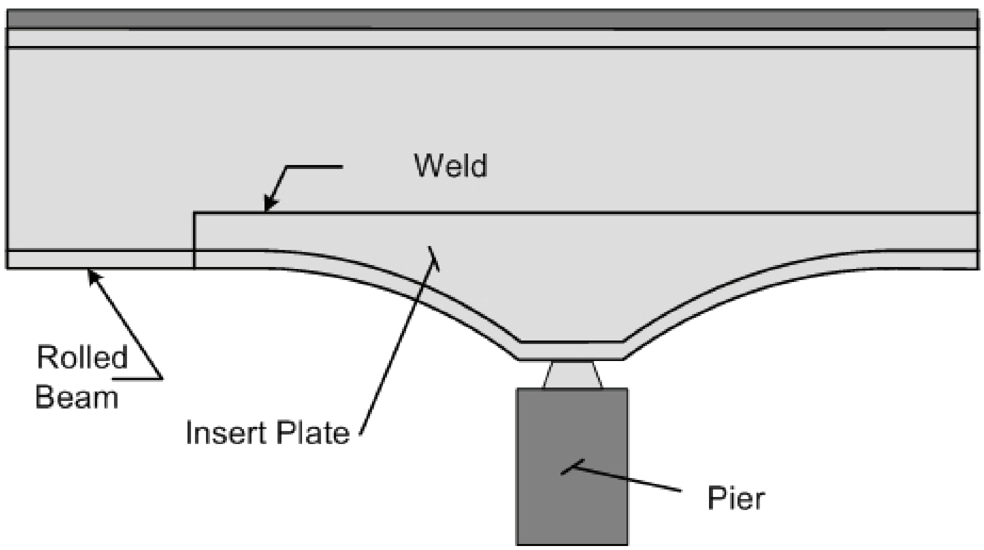

Figure 8.2.11 Web Insert Plate for Multi-beam

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

Figure 8.2.12 Fabricated Variable Depth Girder Bridge

Function of Stiffeners

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

Primary and Secondary Members

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

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

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

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

Figure 8.2.14 Curved Multi-girder Bridge

Figure 8.2.15 Straight Multi-girder Bridge

Fatigue Prone Details

Some common areas for fatigue prone details are:

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

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

Fracture Critical Areas

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

8.2.3 Overview of Common Defects

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

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

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

8.2.4 Inspection Procedures and Locations

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

Procedures

Visual

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

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

Physical

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

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

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

Shear Zones

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

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

Flexure Zones

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

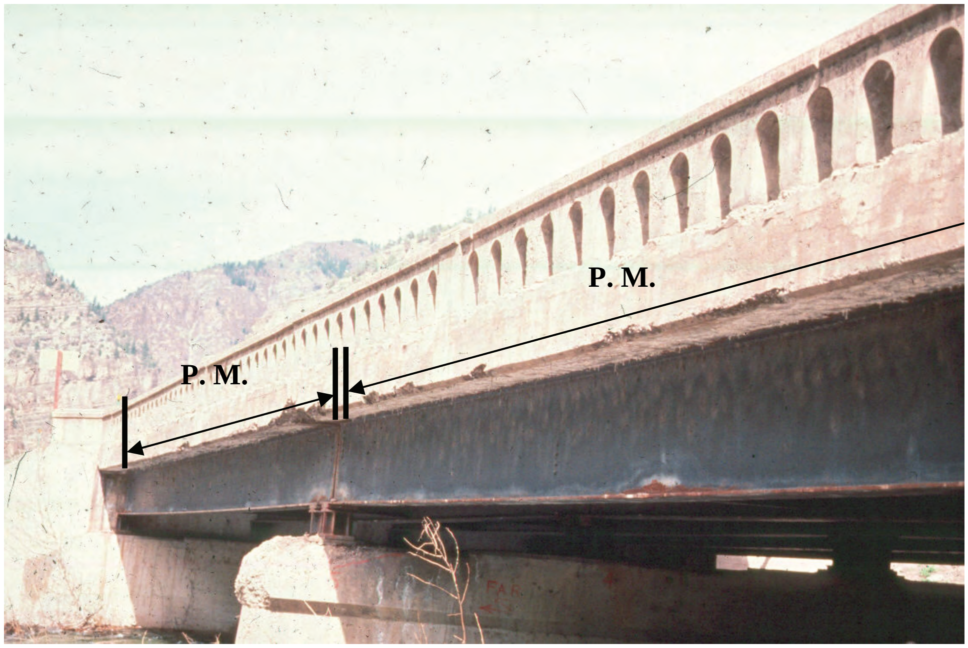

Figure 8.2.17 Flexural Zone on a Simple Rolled Multi-beam Bridge is Entire Length of Beams Between Supports

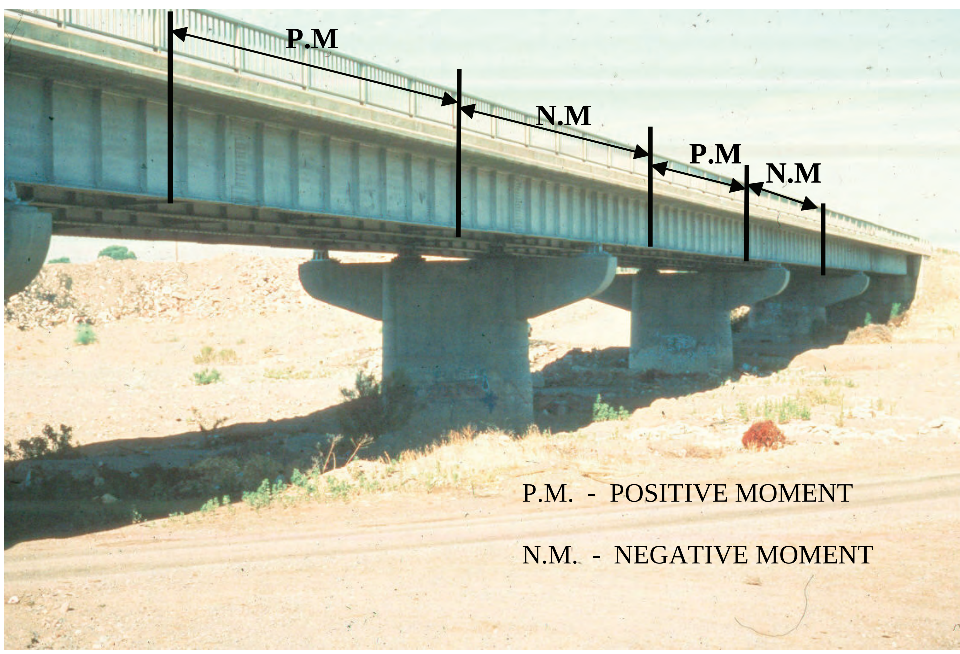

Figure 8.2.18 Flexural Zone on a Fabricated Multi-girder Bridge

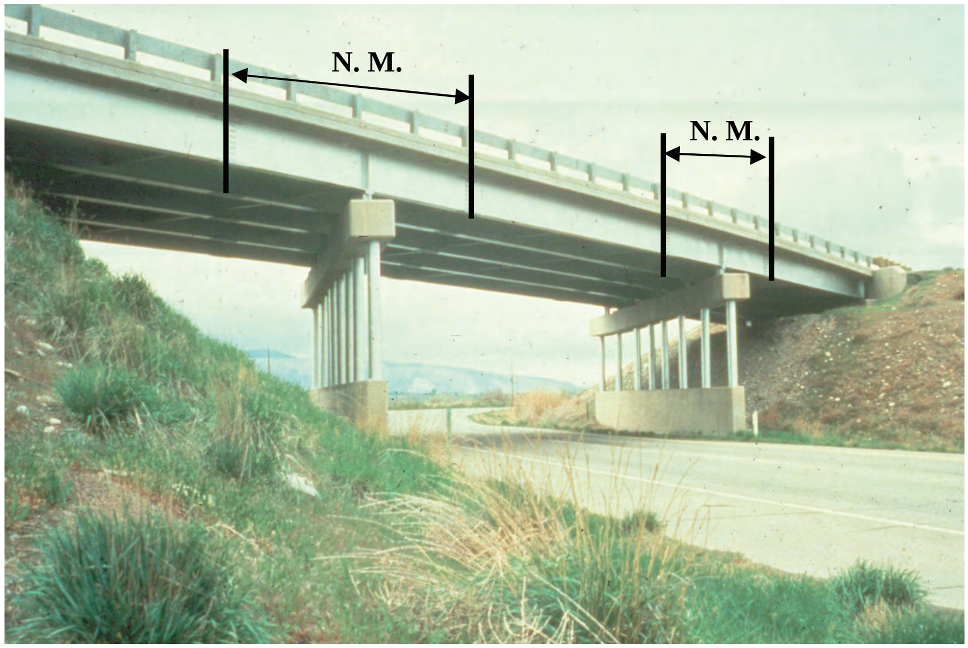

Figure 8.2.19 Negative Moment Region on a Continuous Rolled Multi-beam Bridge

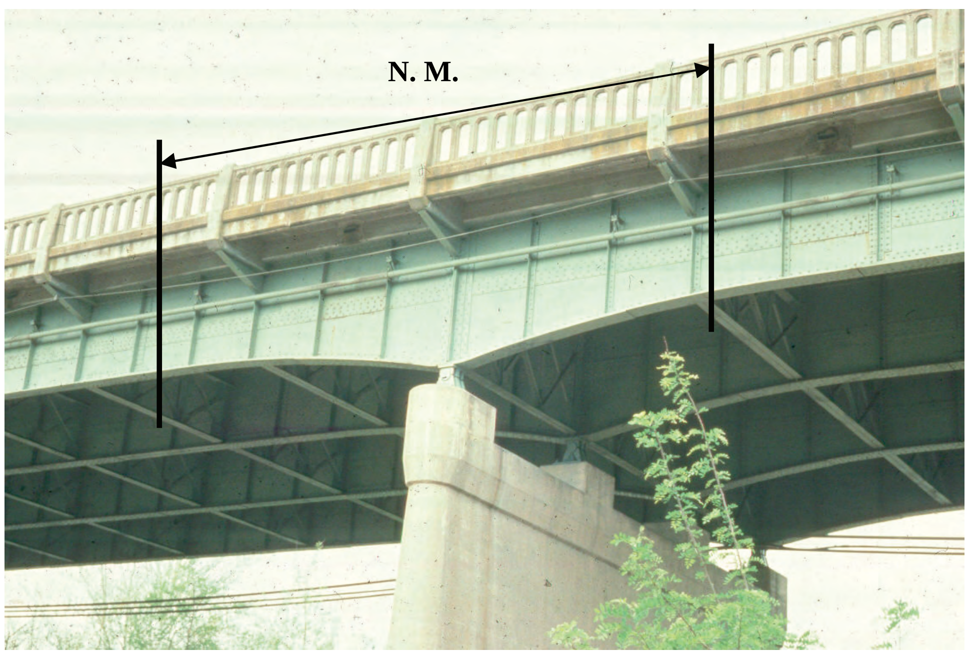

Figure 8.2.20 Negative Moment Region on a Continuous Fabricated Multi-girder Bridge

Secondary Members

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

Figure 8.2.21 End Diaphragm

Figure 8.2.22 Intermediate Diaphragm

Areas That Trap Water and Debris

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

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

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

Areas Exposed to Traffic

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

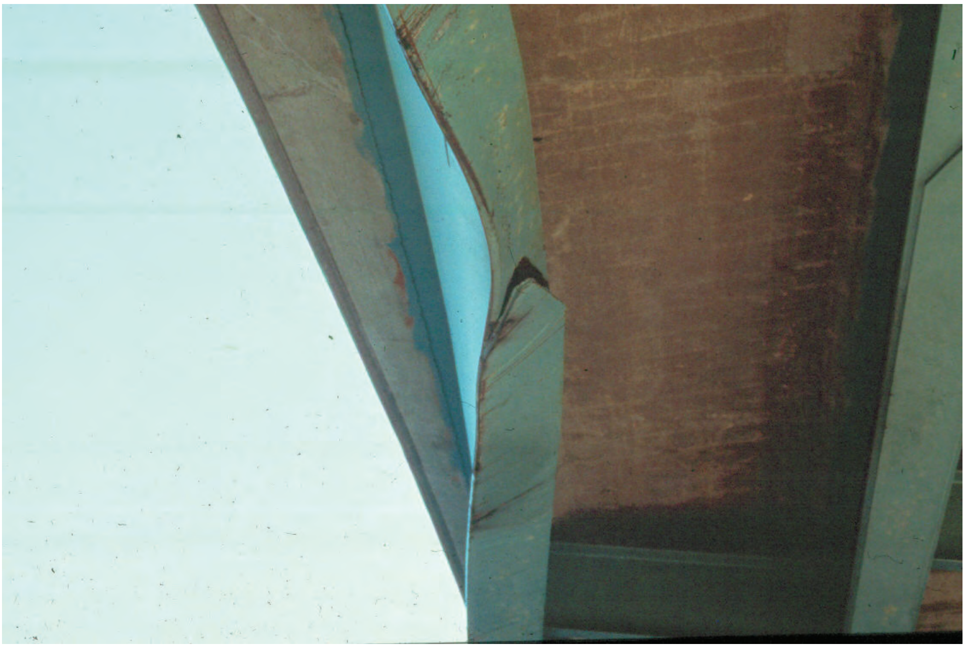

Figure 8.2.23 Collision Damage on a Rolled Multi-beam Bridge

Figure 8.2.24 Collision Damage on a Fabricated Multi-girder Bridge

Fatigue Prone Details

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

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

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

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

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

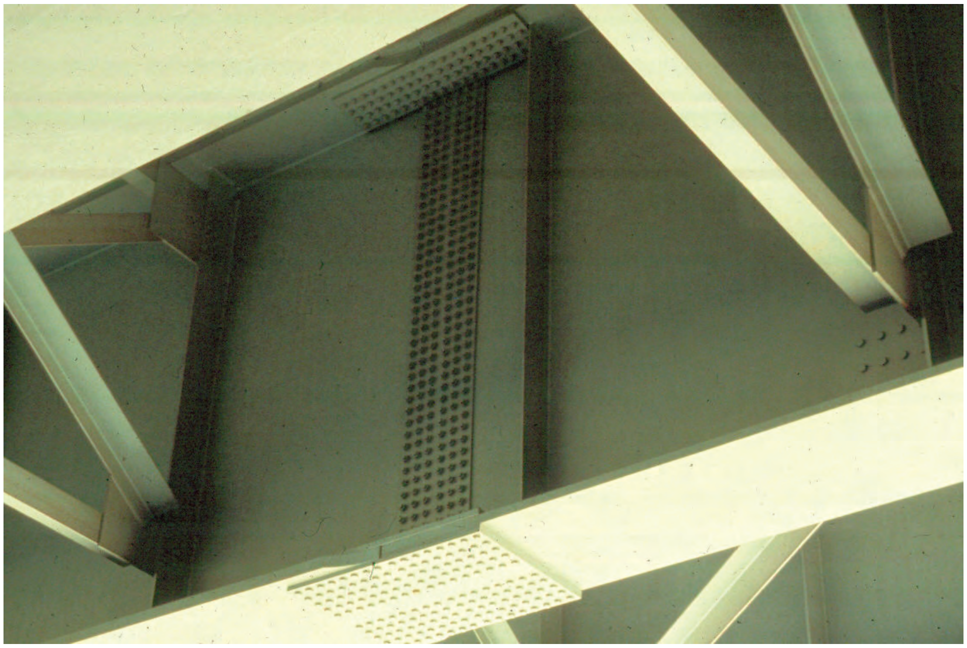

Figure 8.2.25 Field Splice

Figure 8.2.26 Welded Attachment in Tension Zone of a Beam

Fracture Critical Members

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

Out-of-plane Distortion

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

Girder Webs at Diaphragm Connections

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

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

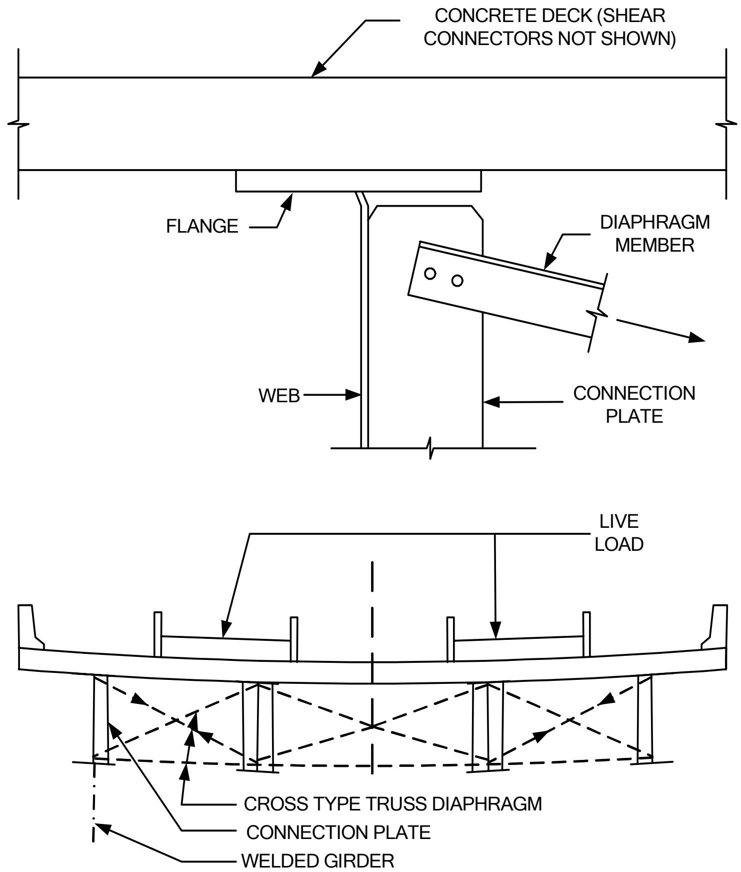

Figure 8.2.27 Out-of-plane Distortion in Web Gap at Diaphragm Connections

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

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

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

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

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

Ends of Diaphragm Connection Plates in Girder Bridges

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

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

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

Lateral Gussets on Plate Girder Webs at Connections

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

8.2.5 Evaluation

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

NBI Rating Guidelines

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

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

Element Level Condition State Assessment

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

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

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

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

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