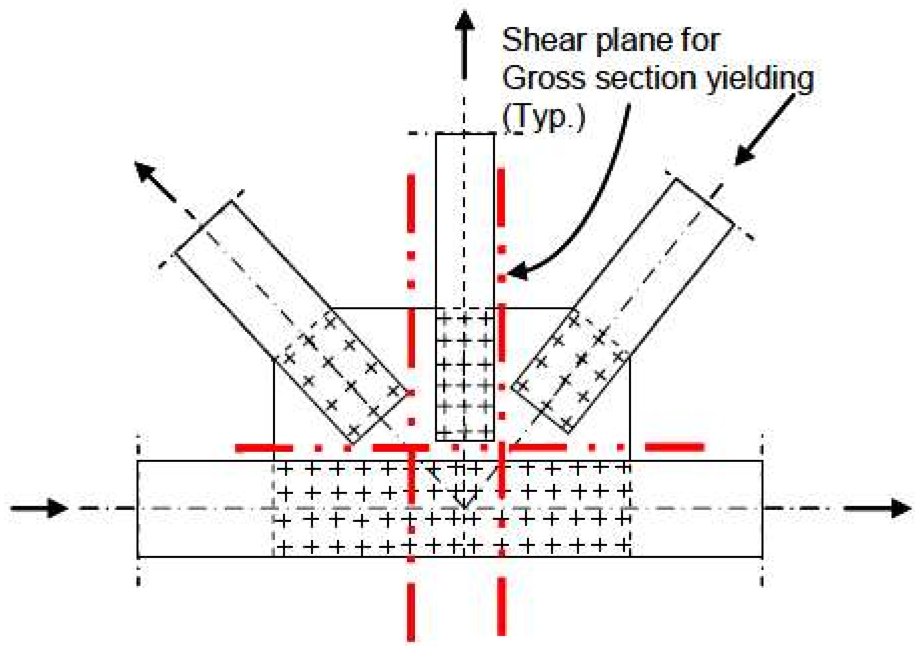

Figure 10.8.10 Examples of Gross Section Shear Yielding Planes

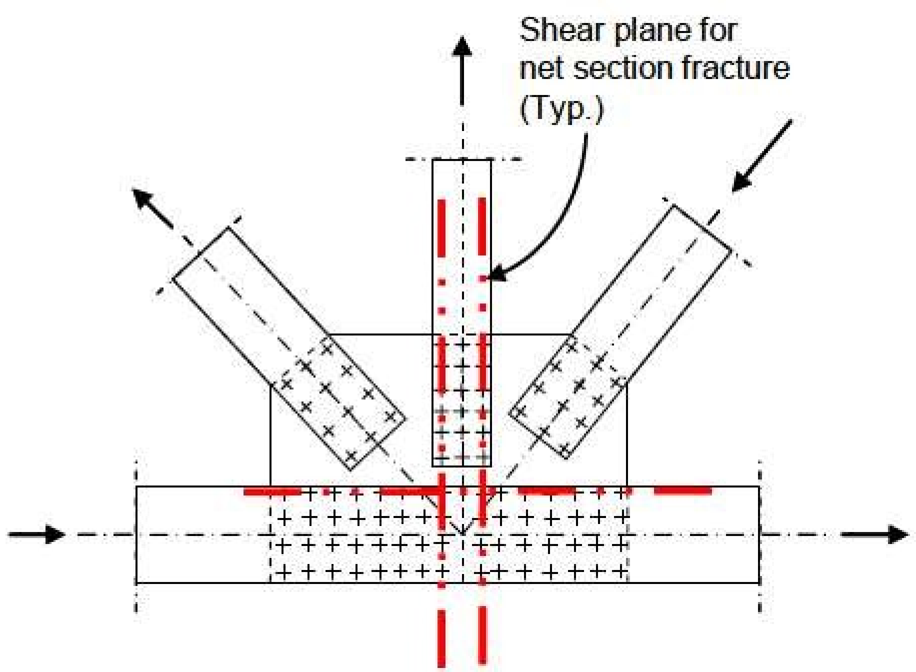

Figure 10.8.11 Examples of Net Section Shear Fracture Planes

Gusset Plates in Compression

Given the complex innerconnectivity that gusset plates provide, gusset plates subject to compression are evaluated against the compressive resistance, which considers the modes of buckling, the effective width of the compression member, and the unbraced length of the compression member, among other factors. The unbraced length may be determined as the distance between the last row of fasteners in the compression member under consideration and the first row of fasteners in the closest adjacent member measured along the line of action of the compressive axial force (see Figure 10.8.12).

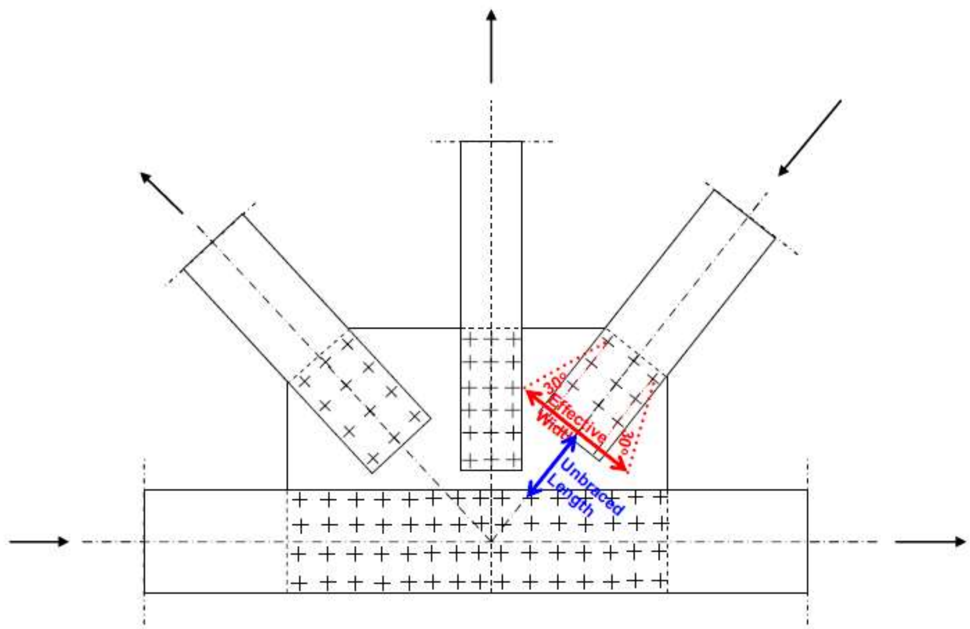

Figure 10.8.12 Example Showing the Unbraced Length and Effective Width for a Gusset Plate in Compression

Gusset Plates under Combined Flexural and Axial Loads

Gusset plates subject to combined flexural and axial stresses on the gross area of the plate are investigated for the critical section and consider the specified minimum yield strength of the plate. Examples of combined flexural and axial load planes are illustrated in Figure 10.8.13.

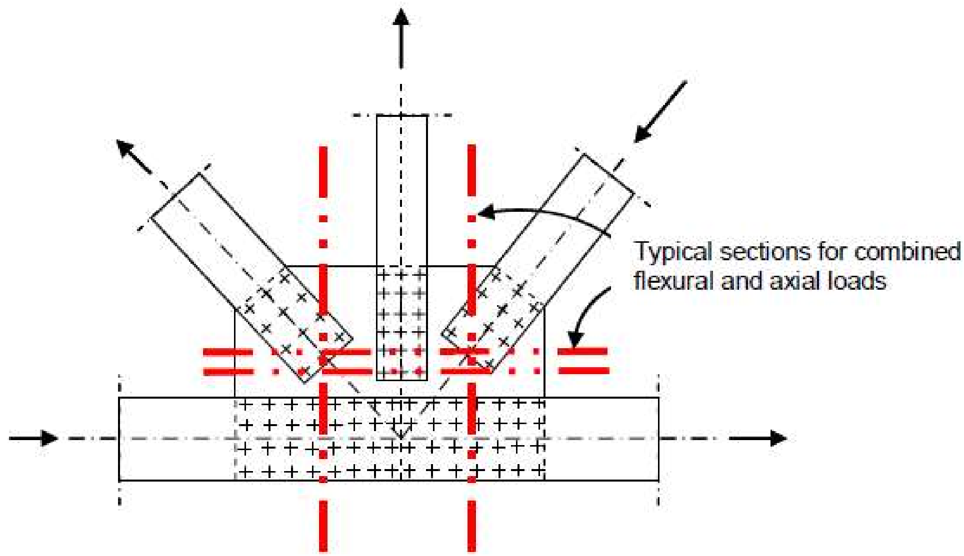

Figure 10.8.13 Examples of Combined Flexural and Axial Load Planes

Gusset Plates Connecting Secondary Members

Gusset plates are also used in connecting secondary (bracing) members together for various superstructure types. The secondary members may be connected to primary members at panel points or may be connected to other secondary members (see Figures 10.8.14 and 10.8.15). Gusset plates connecting secondary members are generally not as complex in design due to the inherent nature of the secondary members.

Figure 10.8.14 Gusset Plate Connecting Secondary (Bracing) Members to a Primary Load-Carrying Truss Member

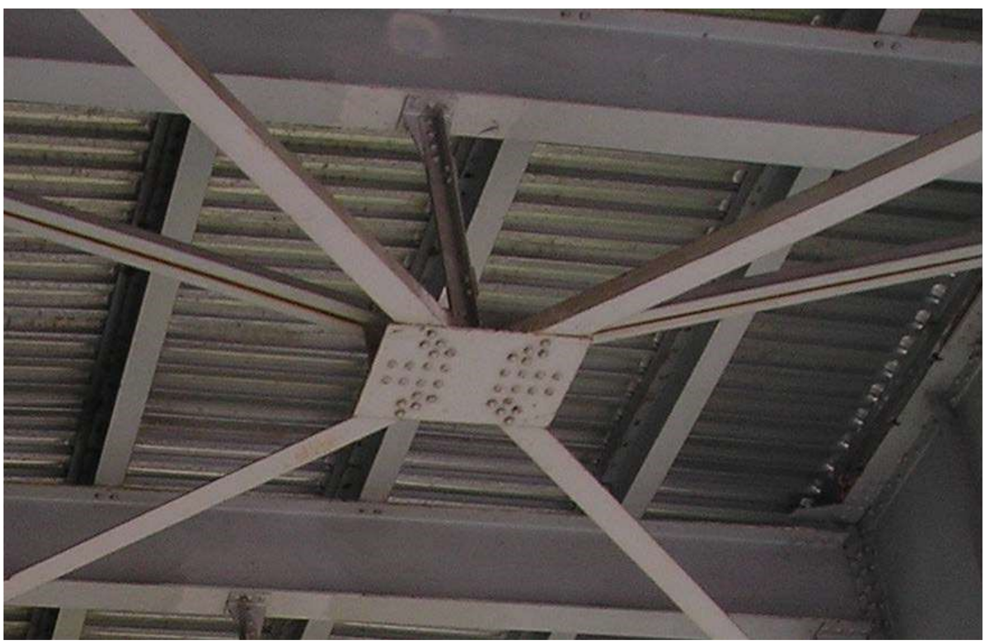

Figure 10.8.15 Gusset Plate Connecting Secondary (Bracing) Members on a Steel Two-Girder Bridge

10.8.3 Overview of Common Deficiencies

Common deficiencies that occur on steel gusset plates include:

- Corrosion
- Fatigue cracking
- Tack welds
- Overloads
- Coating failures
- Loose, missing or deteriorated fasteners
- Repairs or retrofits
- Out-of-plane distortion (including buckling)

See Topic 6.3.5 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges

10.8.4 Inspection Methods and Locations

Inspection methods to determine other causes of steel deterioration are discussed in detail in Topic 6.3.7.

Methods

Visual

Many deficiencies in gusset plates 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 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 deficiency. The use of degreasing spray before and after removal of the paint may help in revealing the deficiency.

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 and the original drawings.

Hammer sounding may be performed on suspect bolts and rivets to detect loose or broken fasteners.

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, examine any other similar locations and details.

Advanced Inspection Methods

Several advanced methods are available for steel inspection. Nondestructive methods, described in Topic 15.3.2, include:

- Acoustic emissions testing
- Corrosion sensors
- Smart coatings
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Computed tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Electrochemical fatigue sensor (EFS)
- Magnetic flux leakage

Other methods, described in Topic 15.3.3, include:

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

Locations **General**

The basic requirement for all fracture critical members is visual inspection conducted within arm's length (or a "hands-on" inspection). Gusset plate connections sometimes require special tools to aid in visual inspection. Simple "mirrors and a stick" are good aids, though other tools such as bore scopes may be needed if internal areas are too confined for physical access and require close evaluation. Remote cameras connected to a viewing screen and a recording device can be used for otherwise inaccessible areas. If remote cameras are not available, simple pole-mounted video cameras may suffice. Prototype digital imaging equipment (to obtain dimensions) and robotic climbers (to access difficult to reach areas) have also been developed.

As with inspecting any other bridge member, the inspector is responsible for practicing good and thorough documentation during the inspection of gusset plates. Gusset plates and their fasteners are measured to an accuracy of 1/16 of an inch in the field. Measurements of the gusset plates and fasteners are recorded and compared to the design or as-built drawings, along with any deficiencies that were detected during the assessment (see Figure 10.8.16). Deficiencies that are recorded include corrosion (section loss), fatigue cracking, tack welds, paint failures, fastener condition, presence of repairs or retrofits, and out-of-plane distortions.

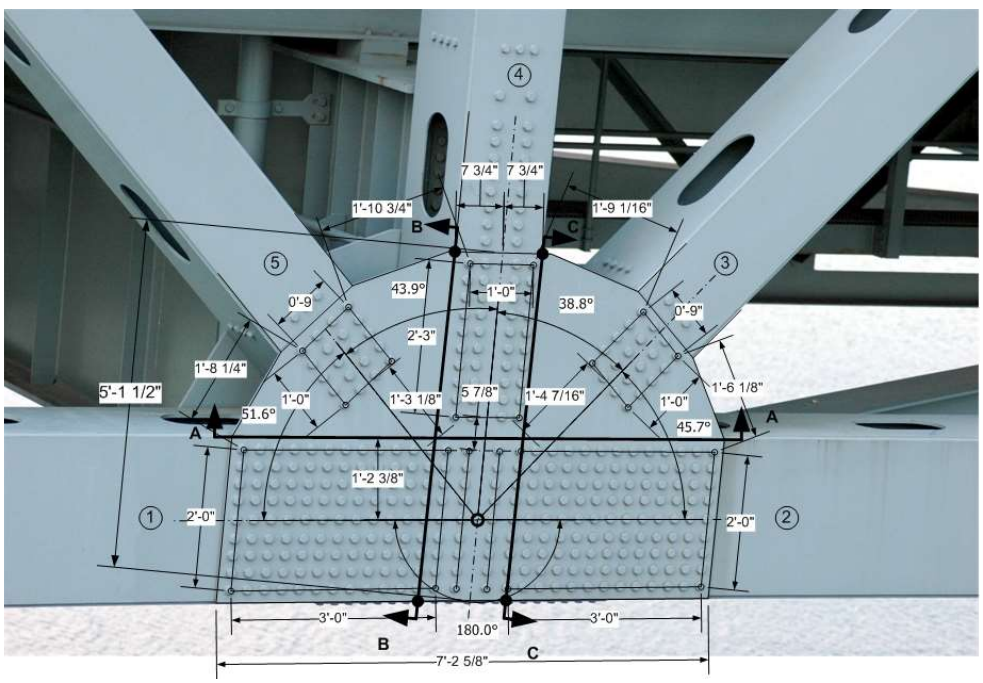

Figure 10.8.16 Gusset Plate Field Measurements

Areas with Corrosion

Surface corrosion may occur on gusset plates and can lead to section loss (see Figure 10.8.17). Corrosion may also occur on the surfaces between the gusset plate and connecting truss or arch member. This type of corrosion, known as “scaling corrosion,” can lead to section loss on the interior surface of the gusset plate and the connecting member.

Document the primary gusset plates if they contain any corrosion that is evident. Visual inspections that use traditional measurement devices (such as calipers, tape measure or depth probe) may not be able to detect or quantify section loss caused by corrosion for the entire plate. Locations where corrosion is discovered are documented and placed in the bridge file for future inspections. When conducting an inspection, review information that is in the bridge file from previous inspections. Nondestructive testing may also be required to determine the condition of the gusset plate.

Figure 10.8.17 General Corrosion of Gusset Plates

Areas with Section Loss

Significant section loss can occur due to corrosion where the horizontal members frame into the gusset plates (see Figure 10.8.18). Proper visual inspection may be impeded due to debris built-up on the member or from heavy rusting or corrosion.

Clean areas that trap debris or hold water in order to evaluate the remaining section at these locations. Areas containing corrosion are also cleaned and then evaluated. The use of a chipping hammer (geologist or masonry hammer), angle grinder, or drill fitted with a flexible paint stripping wheel is recommended. Necessary safety precautions (gloves, glasses or goggles, and respirator) are followed when these tools are being used. Refer to Topic 2.2.3 for more information on personal protection.

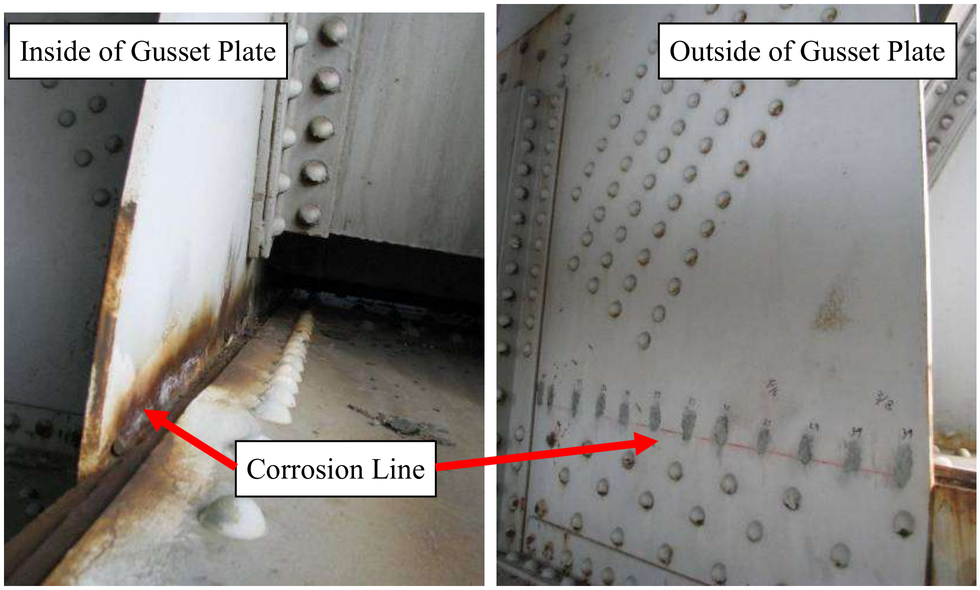

Figure 10.8.18 Corrosion Line Viewed from Inside and Outside of Gusset Plate

An ultrasonic thickness gage (D-meter) is preferred to measure the remaining thickness of a gusset plate (see Figure 10.8.19). Using a D-meter requires the transducer to be placed on a relatively flat surface. This will generally require the corroded surface to be ground smooth so that the D-meter transducer and couplant can obtain an accurate measurement. Paint will typically need to be removed to obtain accurate and proper readings. If not removed, account for the thickness of the paint, since it can significantly affect the reading. For major section loss and heavy pitting, the inspector may be required to take measurements from the opposite side of the plate or "clean side." For a single transducer ultrasonic thickness gage, measuring the clean side is recommended.

When taking measurements from the clean side of the plate, the inspector carefully locates the areas of section loss by visual examination, followed by properly preparing the surface, taking several readings along the line of corrosion and thoroughly documenting the remaining plate thickness using notes, sketches and photographs.

Figure 10.8.19 Inspector Using a D-meter to Measure the Thickness of the Gusset Plate

At locations or situations where a D-meter cannot be used or is not available, a vernier caliper with a depth probe is another tool that can be utilized to determine section loss (see Figure 10.8.20). A straight edge is required in conjunction with the probe to obtain the amount of section loss.

The use of the caliper or depth probe and a straight edge can be cumbersome. In lieu of this method, a tape measure may be used to measure the amount of section loss. This is accomplished by measuring the distance from the steel to the straight edge (see Figure 10.8.21).

For either method, multiple measurements along the line of section loss are recommended so that an adequate evaluation of the potential shear and tension failure planes for each connected member can then be performed.

Figure 10.8.20 Inspector Using Calipers Measure the Thickness of the Gusset Plate

Figure 10.8.21 Inspector Using a Straightedge and Tape to Measure the Section Loss of the Gusset Plate

In addition to the D-meter, caliper or depth probe, and tape measure, a visual weld acceptance criteria (V-WAC) gage may also be used. The V-WAC is used to measure section loss and then subtracted from the total thickness to determine the thickness of the plate that is left (see Figure 10.8.22). It can only measure up to one-quarter inch section loss. The V-WAC is also used to determine the severity of pitting undercutting, porosity and crown height.

Figure 10.8.22 V-WAC Gage and Inspector Using the V-WAC in the Field to Measure the Section Loss of the Gusset Plate

Portable ultrasonic testing (UT) inspection systems may be used to document cracks, flaws, corrosion and internal anomalies in steel gusset plates. These systems may use single element transducers for scanning along single lines or multi-transducer (phased array) probes that can scan multiple lines simultaneously. Both system types have the capability to display their data in both B-scan and C-scan formats to display defects (see Figures 10.8.23 and 10.8.24). The images can also be downloaded and saved in an electronic file. B-scan is a nondestructive inspection method that utilizes ultrasonic waves to image a cross-section (thickness) of an element (plate, flange or web), including the location of the defects. C-scan is a nondestructive inspection method that utilizes short pulses of ultrasonic energy that determine both flaw size and location within a plan view (two-dimensional plane perpendicular to the thickness) of the element tested.

When corrosion is evident, ultrasonic methods are often the most appropriate methods to measure the thickness of a single gusset plate. Research is being directed toward help identify a technology suitable for multi-gusseted connections. Currently, a combination of visual inspection and ultrasonic testing is the most efficient and accurate method.

Regardless of the instrument used to quantitatively and qualitatively evaluate gusset plate corrosion, all deterioration of the gusset plate is thoroughly documented in the inspection report using notes, sketches and photographs. Compare the measured thickness with the original thickness determined from as-built drawings or a portion of the gusset plate with no section loss. Reference to previous inspections documenting the remaining section is required.

Figure 10.8.23 Inspector Using a Portable Ultrasonic Testing Inspection System

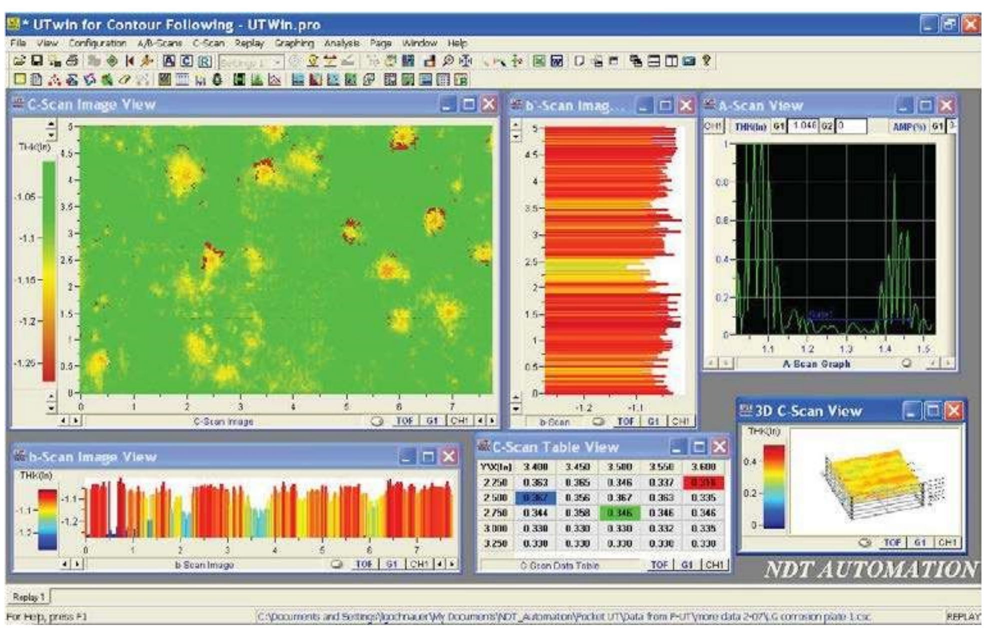

Figure 10.8.24 Ultrasonic Testing Inspection Acquisition Software

Areas Susceptible to Fatigue Cracking

Inspect gusset plates for fatigue cracking. Common locations for fatigue cracks to develop include bolt holes, classified as AASHTO Fatigue Category B, and rivet holes, classified as AASHTO Fatigue Category D. Rivet holes are especially susceptible to fatigue cracking (hence the “D” rating) since these holes may have been punched but not properly reamed during the fabrication process. The rough edges are sources for crack initiation points in tension members due to stress concentrations. Plate cracking can be visually detected by a thin line of corrosion beginning at the fastener (under the head) and propagating from the fastener hole.

Other areas with sharp corners or edges are also inspected for fatigue cracking, as these areas often represent areas with high stress concentrations. Note that if rivets are replaced by high strength bolts, the fatigue category has the ability to change from “D” to “B.”

Cracking of tension members is of particular concern. Any crack found in a gusset plate is considered critical, with the Bridge Owner notified immediately. With any cracking, thoroughly document the exact location and dimensions of the cracks in the gusset plates using notes, (location, length, width and growth history), sketches and photographs. Try to determine the point of crack initiation (see Figure 10.8.25).

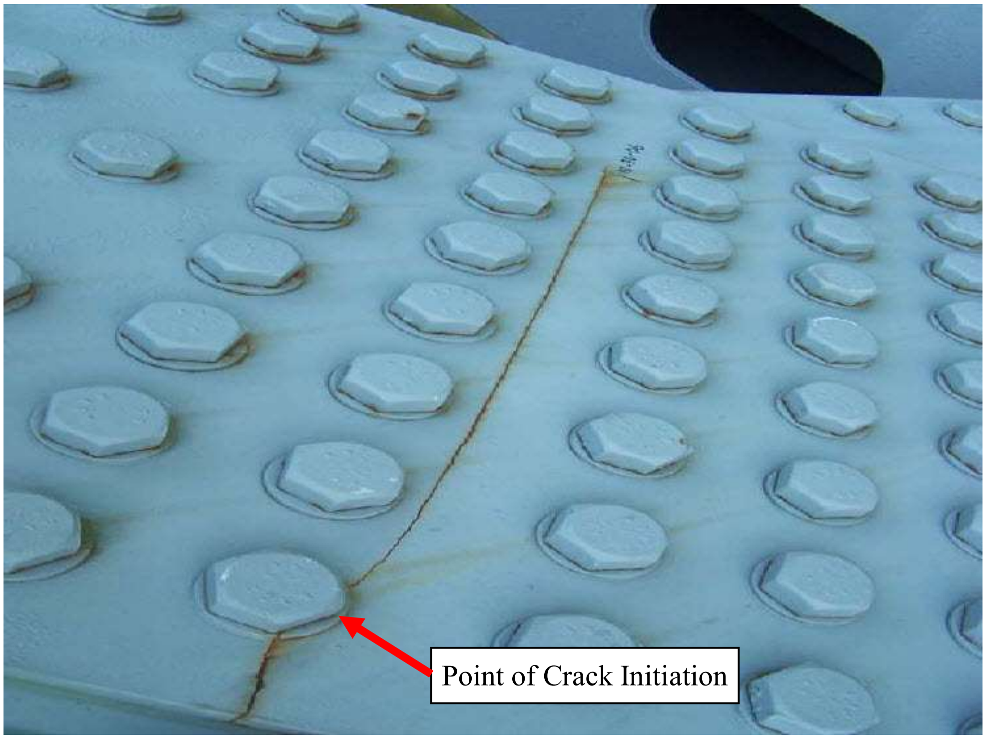

Figure 10.8.25 Cracked Gusset Plate and Point of Crack Initiation

Areas with Tack Welds

During the 1950s and 1960s, fabricators commonly used tack welds to hold members together during riveting operations. Because this type of weld does not provide structural strength, cracks in these welds do not directly represent a problem with respect to the structural integrity of the bridge. However, a tack weld on a tension element is considered a problematic detail because when or if a tack weld cracks, the potential for the crack to propagate into the base metal of the tension element exists (see Figure 10.8.26).

Tack welds exhibiting a full length crack with no evidence of base metal cracking generally do not present a problem. Partial length cracked tack welds, however, still have the potential for the crack to propagate into the base metal when exposed to tension. Crack propagation into fracture critical elements, such as gusset plates, has the potential to cause partial or total bridge collapse. These cracks can also propagate into other tension elements such as a truss chords, vertical or diagonal members, or arch members.

Inspect all cracked tack welds for propagation using methods such as visual observation, dye penetrant, magnetic particle, eddy current and ultrasonic testing. If required, carefully clean the welds using a flexible paint stripping wheel in a grinder or drill. Remember, do not grind tack welds since the grinding tends to smear the metal and can then hide a crack. Thoroughly document the results of the investigation. Removal of partially cracked tack welds may be considered.

Figure 10.8.26   Partial Length Cracked Tack Weld

Areas Subject to Overstress

Gusset plates that are subject to overstress may exhibit either yielding of the section (tension) or buckling of the section (compression). If section loss is present, gusset plates will be more susceptible to overstress due to a reduced capacity (see Figure 10.8.27). The capacity is reduced because less material is available to distribute the tension or compression loads. Review previous inspection reports to see if any distortion or section loss was documented.

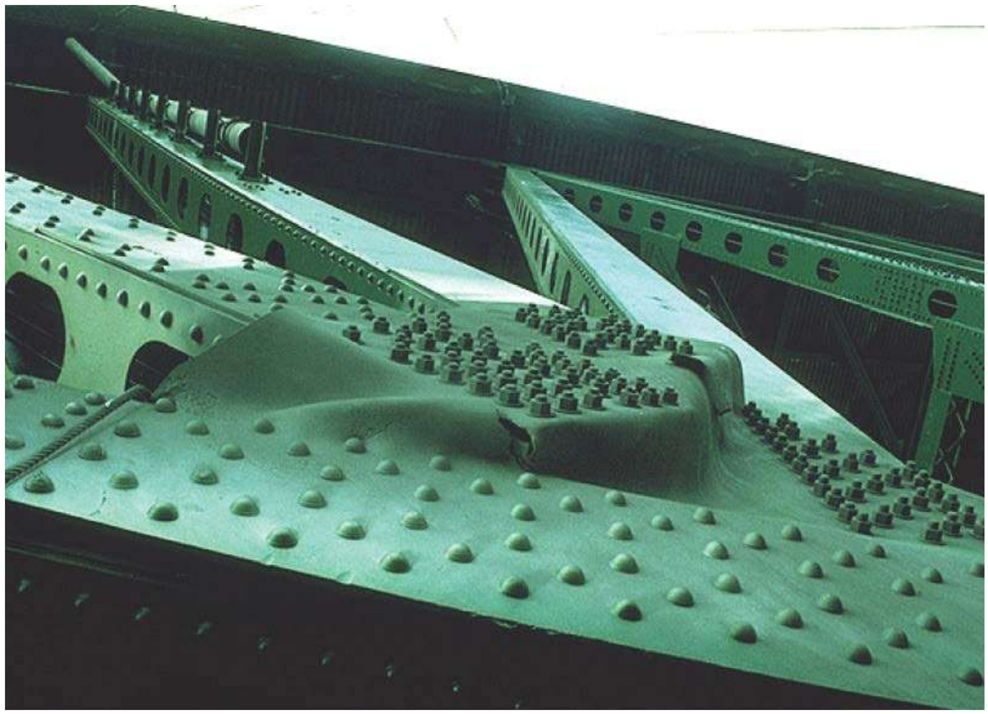

Figure 10.8.27 Gusset Plate Buckling (Compression) Failure due to Major Gusset Plate Section Loss

Areas with Paint Failure

Steel gusset plates are normally protected from corrosion by painting or using weathering steel. The failure of a coating system can eventually lead to corrosion and section loss on the gusset plate (see Figure 10.8.28).

Protective systems for gusset plates include:

- Protective coating
- Galvanizing
- Weathering steel

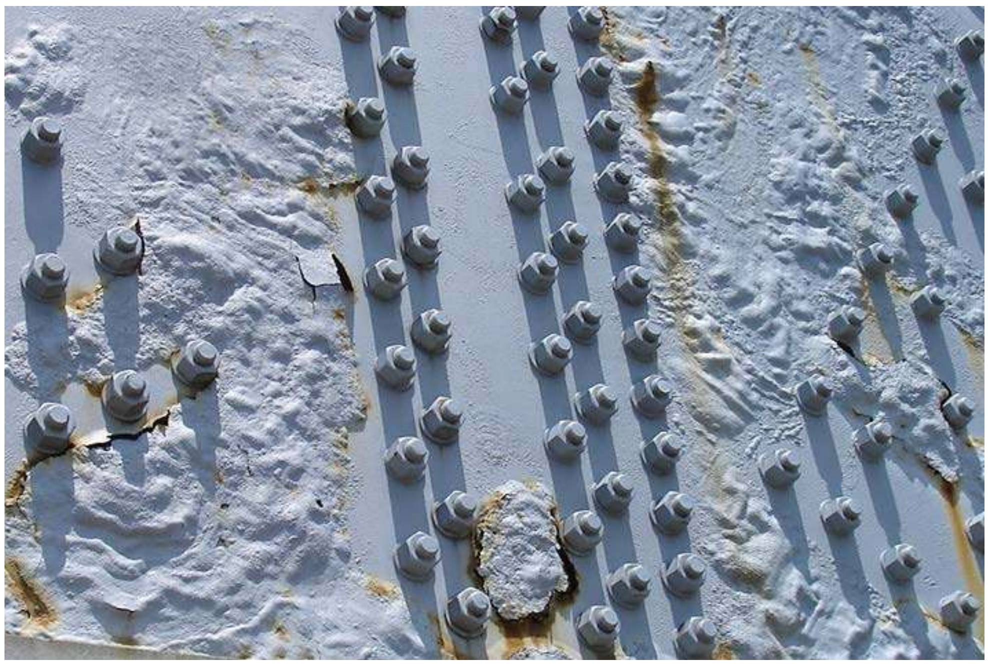

Figure 10.8.28 Gusset Plate with Paint Failure

Loose, Missing or Deteriorated Fasteners

Depending on the detail, pack rust (corrosion) may cause plate separation, which can lead to overstressed fasteners. Rivet or bolt heads can “pop” off (tension failure) under the extreme forces generated by pack rust (see Figure 10.8.29). If the head is still intact, this overstress can be visually observed as out-of-plane rotation of the rivet head.

Inspect the riveted or bolted connection for slipped surfaces and section loss around the individual bolts and rivets. Slipped surfaces occur when there is a break in the bond between the fastener and gusset plate, as exhibited by missing paint or scratched base material.

Loose or broken fasteners may be detected by hammer sounding. Check to assure the fastener number and pattern is consistent with the as-built or construction plans.

Figure 10.8.29 Missing Bolts on Gusset Plate

Areas with Repairs and Retrofits

Structural steel repairs and retrofits are used to strengthen deteriorated and distorted gusset plates. Repairs are normally made by bolting or welding. Riveting has been used in rare instances. Types of retrofits for gusset plates include:

- Plate thickening (see Figure 10.8.30)
- Free (unbraced) edge stiffening (see Figure 10.8.30)
- Stiffening within the plate

Welded retrofits are considered to be very problematic (see Figure 10.8.31). Many trusses and arches older than 1970 are constructed with steel that is more brittle than modern steel. Durable and high quality welds are difficult to obtain for these more brittle steels. Toughness requirements were generally not enforced until the late 1970s.

For welded gusset plate retrofits, closely examine the toe of the weld and base metal for signs of cracking. Visual inspection may need to be supplemented with more in-depth inspection using the proper tools.

Gusset connections with multiple plate layers, whether retrofits or part of the original construction, will often complicate the inspection and evaluation process. Due to the complexity of these gusset plates, extra care is taken with D-meter (or other thickness measurement) readings and distortion documentation.

Inspect all repairs and retrofits for distortion, deterioration, pack rust and tack welds as a means to verify that the repairs and retrofits are functioning as intended.

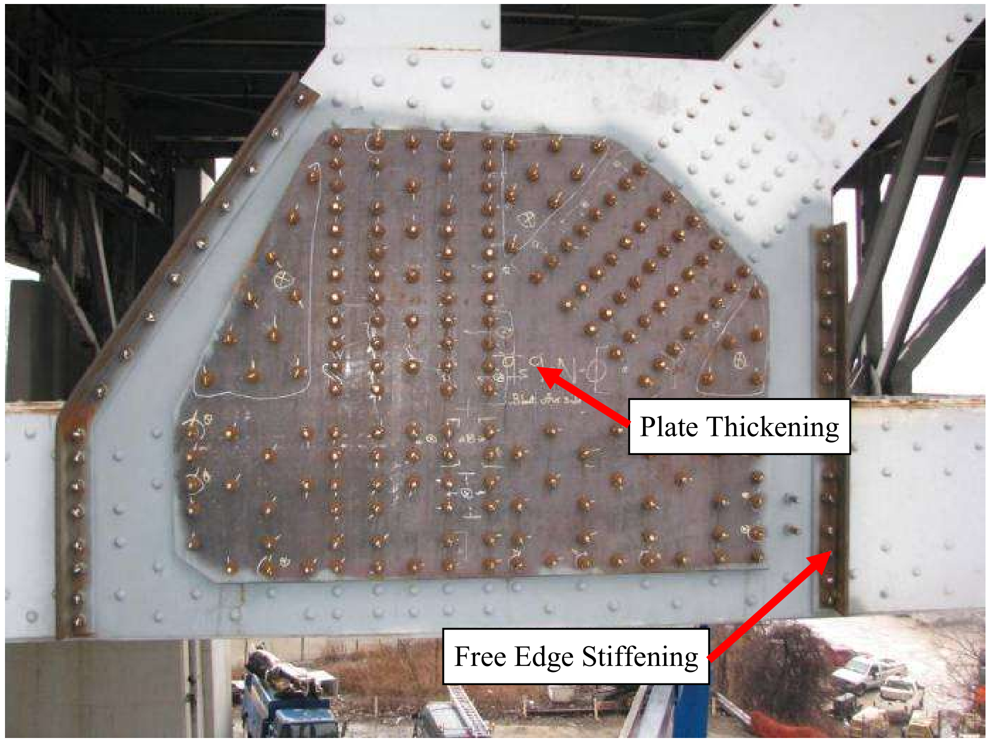

Figure 10.8.30 Plate Thickening and Free Edge Stiffening on Gusset Plate

Figure 10.8.31 Poorly Designed Welded Retrofit

Areas with Out-of-Plane Distortion

Gusset plate distortion may be caused by overstressing of the plate due to overloaded vehicles or inadequate bracing during the initial erection. Other causes include fit of the connected members, section loss due to corrosion, design error and increased dead load. These causes can be broken down into two categories: geometry driven and load driven.

Sight across the gusset plate surface looking for out-of-plane distortion of the plate. A straight edge is used to evaluate and quantify any distortion of the unbraced gusset plate edges between members (see Figure 10.8.32). If gusset plates exist on both sides of a given truss or arch member, check both gusset plates for out-of-plane distortion. Any distortion that is detected is documented with respect to a common reference (see Figure 10.8.32).

Measure and indicate the amount of plate distortion by measuring from the straight edge of the plate. Set up a reproducible reference system to record measurements against. Dimensions of the distorted gusset plates are measured from a reference point on the plate. This reference point is used in subsequent inspections to provide findings based on a common point of reference that can then be compared to previous measurements.

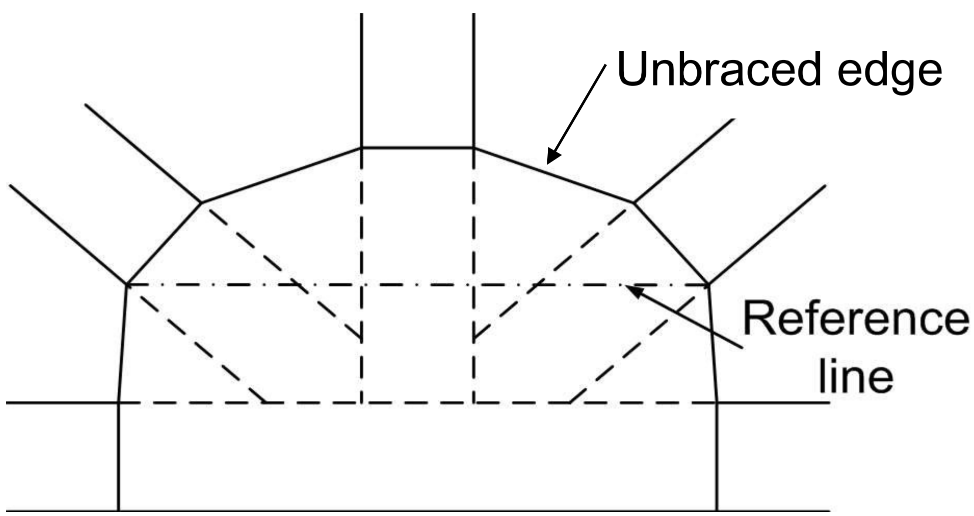

Figure 10.8.32 Unbraced Gusset Plate Edges and Reference Line

Gusset plate distortion may also be caused by pack rust (corrosion). Pack rust is formed between two mating steel surfaces when the correct combinations of moisture, oxygen and failure of the protective coating are present. As the steel corrodes, it expands and generates pressure between the steel surfaces, therefore forcing the surfaces to separate. Depending on the detail, this separation can sometimes cause plate distortion and lead to overstressed mechanical fasteners.

Gusset plate distortion caused by pack rust is generally observed to be directly proportional to the amount of pack rust observed between the plate and the member. The amount of distortion can be easily obtained by using a taut string line along the free edges of the plate and measuring the distance between the line and the inside edge of the plate (see Figure 10.8.33). As with any gusset plate deficiency, distortion due to pack rust is thoroughly documented using notes, sketches and photographs.

Distorted gusset plates connecting compression members are considered more critical than gusset plates connecting tension members. Any distortion can be considered critical and may warrant an analysis.

Figure 10.8.33 Inspector Measuring Out-of-Plane Distortion Using String Line and Tape Measure

10.8.5 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of gusset plates. 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 condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Under NBI component condition rating guidelines, gusset plates are considered part of the superstructure and do not have an individual rating. Take into account the condition of the gusset plates when rating the superstructure, which may be lowered due to gusset plate deficiencies. The superstructure is still rated as a whole unit, but gusset plates may be the determining factor in the given rating.

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition 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 component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

In an element level condition state assessment of a bridge with gusset plates, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Superstructure	
162	Gusset Plate

BME No.	Description
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating

The unit quantity for the gusset plate is each. Each gusset plate element is placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity of protective coating is square feet, with the total area distributed among the four available condition states depending on the extent and severity of the deficiency. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of gusset plates:

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
362	Superstructure Traffic Impact (load capacity)
363	Steel Section Loss
364	Steel out-of-plane (Compression Member)

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

10.8.6 Reasons to Inspect

On Wednesday, August 1, 2007, the Interstate 35W (I-35W) highway bridge over the Mississippi River in Minneapolis, Minnesota collapsed after experiencing a superstructure failure in the 1,000-foot long deck truss portion of the structure (see Figure 10.8.34). The result of this tragic event was the loss of 13 people and the injury of 145 people.

The ensuing National Transportation Safety Board (NTSB) inspection discovered the original design process led to a serious error in the sizing of some of the gusset plates in the main trusses. These gusset plates were roughly half the thickness required. This design error was not detected during the internal review process conducted by the design firm responsible for the original design in the early 1960s.

The NTSB concluded that the bridge was designed with undersized gusset plates and the riveted gusset plates consequently became the weakest link in the structural system. Although inspections conducted in accordance with the NBIS are not designed or expected to uncover such design-related problems, this bridge catastrophe has raised significant awareness in the safety inspection of gusset plates. Gusset plates connect primary load-carrying members and it is important that they are accurately inspected.

Figure 10.8.34 Collapsed I-35W Mississippi River Bridge

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Chapter 10

Inspection and Evaluation of Steel Superstructures

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Topic 10.9 Steel Eyebars

10.9.1 Introduction

Eyebars are tension only members consisting of a rectangular bar with enlarged forged ends having holes through them for engaging connecting pins to make their end connections. Eyebars are predominantly found on older truss bridges, but can also be found on suspension chain bridges, arch bridges, and as anchorage bars embedded within the substructures of long span bridges (see Figures 10.9.1 to 10.9.4).

Figure 10.9.1 Typical Eyebar Tension Member on an Arch

Figure 10.9.2 Eyebar Cantilevered Truss Bridge (Queensboro Bridge, NYC)

Figure 10.9.3 Eyebar Chain Suspension Bridge

Figure 10.9.4 Anchorage Eyebar

Heat treated steel eyebars have been used in bridges around 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 10.9.5). 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 10.9.5 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 10.9.6). As a result, costly structural modifications and retrofits were made to many of these bridges (see Figure 10.9.7), while some others have been demolished. Eyebars are rarely used in new bridge designs but are present on many existing bridges.

Figure 10.9.6 Inspection of Eyebars

Figure 10.9.7 Retrofit of Eyebars to Add Redundancy

The design of the eyebar connections does not allow for inspection by common methods. These connections collect water and promote corrosion at the critical point on the eyebar head (see Figure 10.9.8).

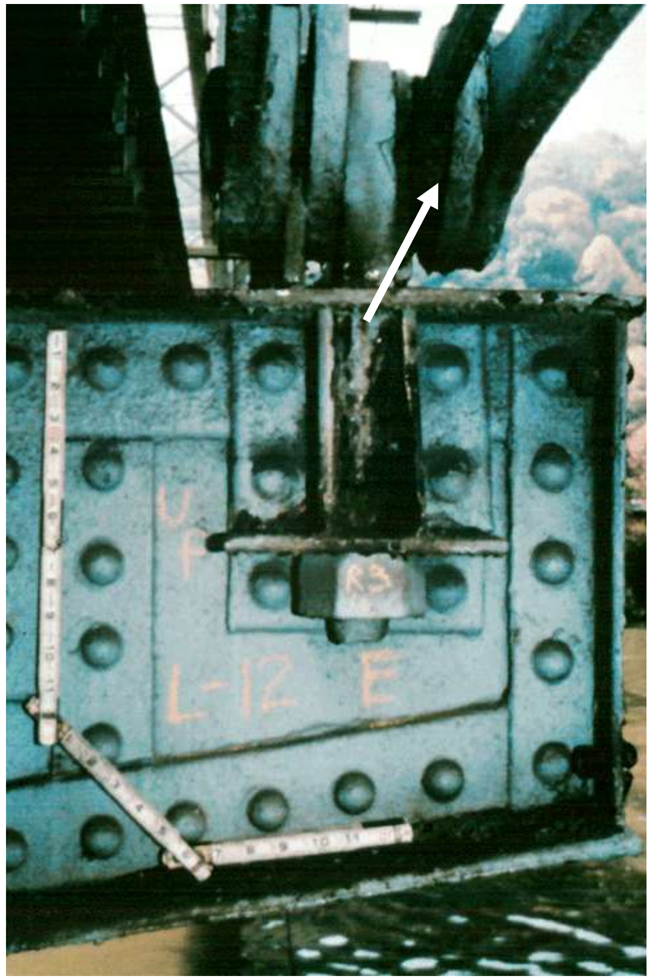

Figure 10.9.8 Eyebar Connection with Corrosion

10.9.2 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 10.9.9).

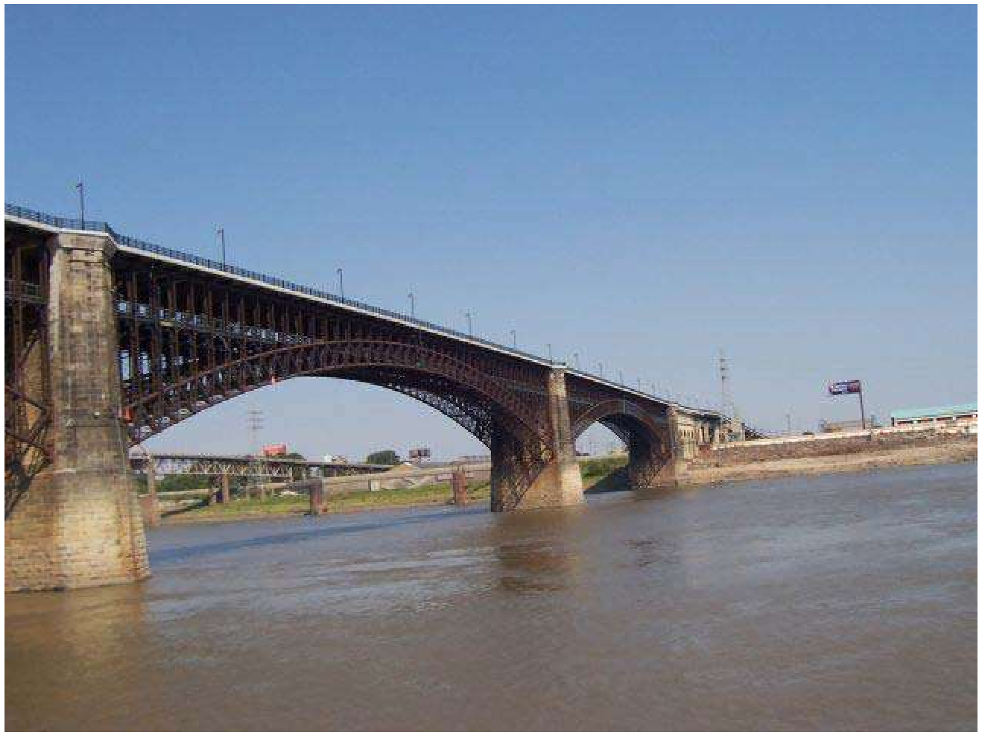

Figure 10.9.9 Eads Bridge, St. Louis

Nickel alloy steel eyebars were developed around 1900. Nickel steel showed high physical properties with a yield point of 55,000 psi and an ultimate strength of 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.

Around 1915, mild grade heat-treated steel eyebars were developed with a yield point of 50,000 psi and an ultimate strength of 80,000 psi. This steel was basically "1035" steel, or plain carbon steel with 35 percent carbon content. Eyebars manufactured from this steel 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 yield point of 75,000 psi and minimum ultimate strength of 105,000 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 10.9.10 and 10.9.11). Loop rods were first made of wrought iron (and later from steel) by forging a heated bar around a pin and pounding the bar until a closed loop was formed.

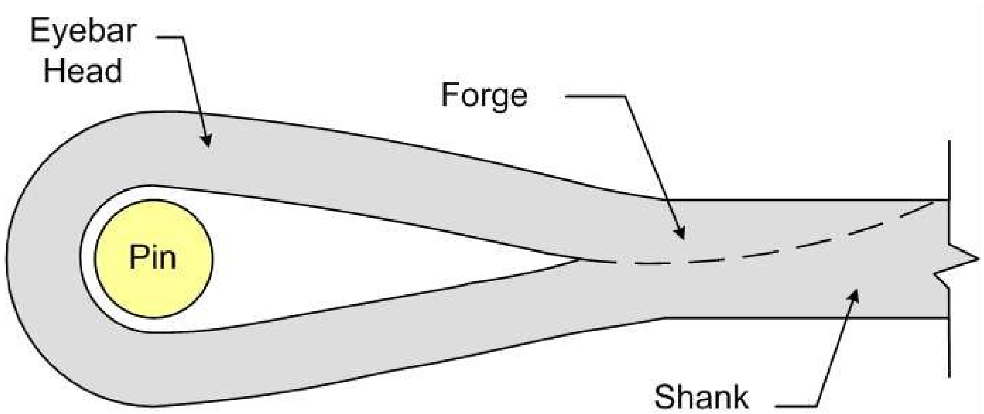

Figure 10.9.10 Forged Loop Rod

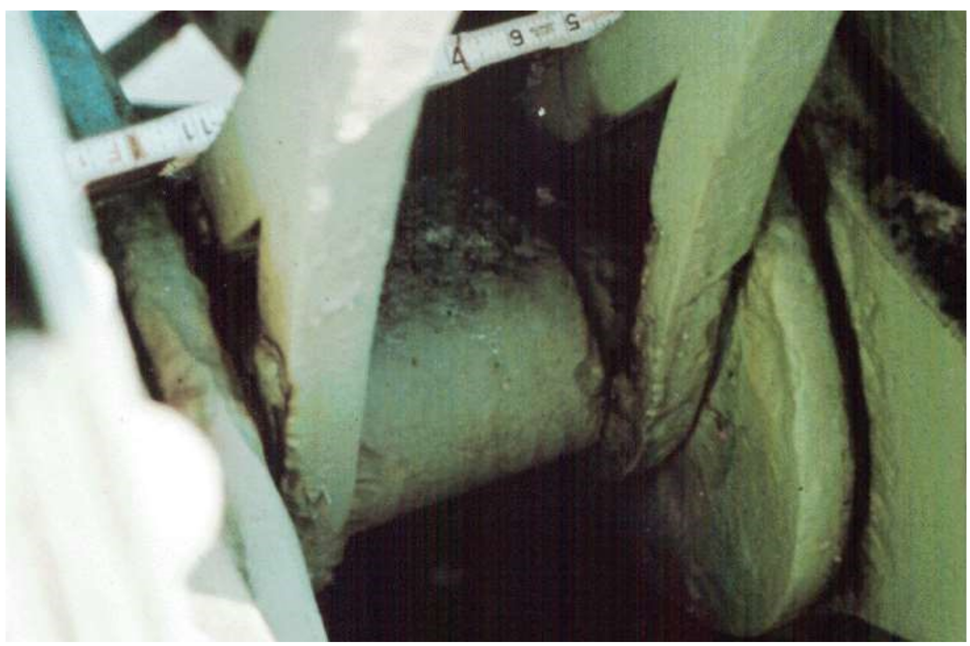

Figure 10.9.11 Close-up of the End of a Loop Rod

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 (see Figure 10.9.12). 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. Most other forging presses operate with vertical rams.

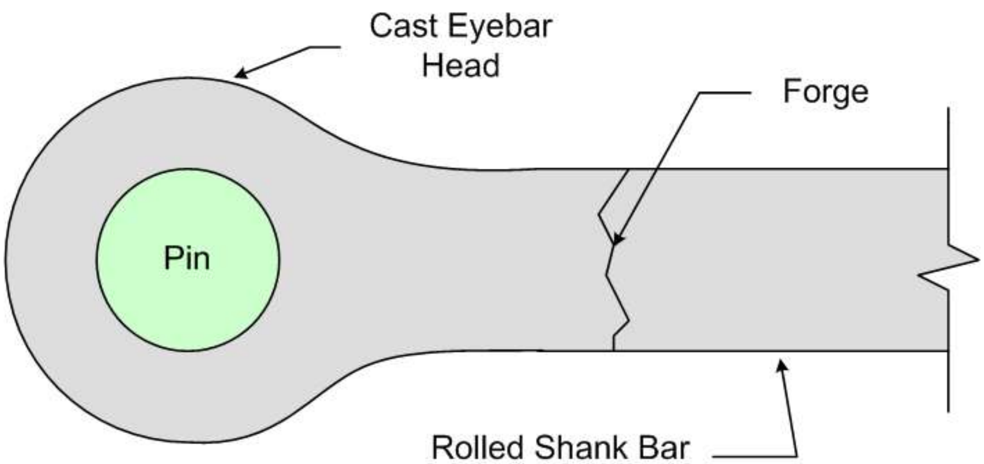

Figure 10.9.12 Forged Eyebar by Mechanical Forge Press

Pin Hole

The pin hole in the enlarged head of the eyebar is commonly formed by boring (see Figure 10.9.13). To fabricate the hole, flame cutting is permitted to within two inches of the pin diameter.

Figure 10.9.13 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 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 and Nomenclature

The dimensions of a typical eyebar are as follows:

- Thickness - usually one to two inches
- Width - usually 8 to 16 inches
- Length - varies with bridge design

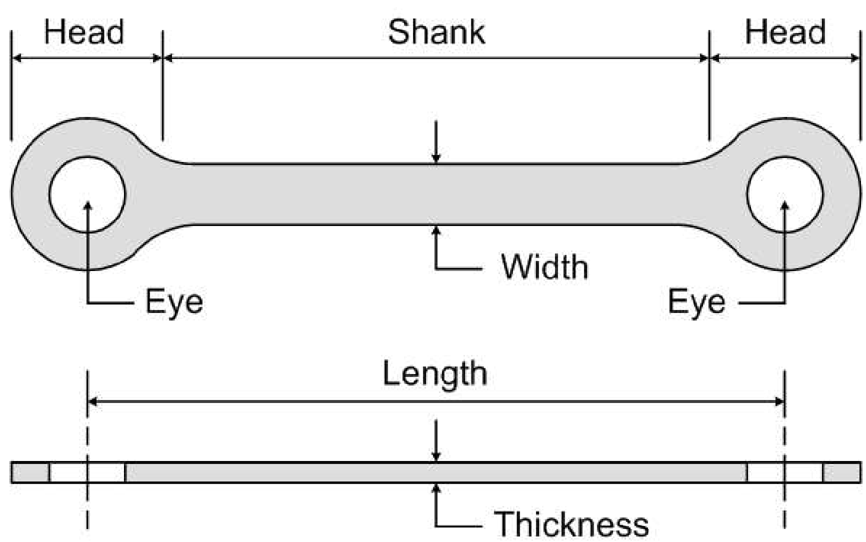

Figure 10.9.14 Eyebar Dimensions

The eyebars on the Silver Bridge were between 45 and 55 feet in length, 12 inches wide, and varied in thickness.

Packing

Packing is the term used to describe the arrangement of the eyebars at a given point. Eyebars may be spread apart or tightly packed together (see Figures 10.9.15 and 10.9.16). The packing is symmetrical about the center-line of the member.

Figure 10.9.15 Loosely Packed Eyebar Connection

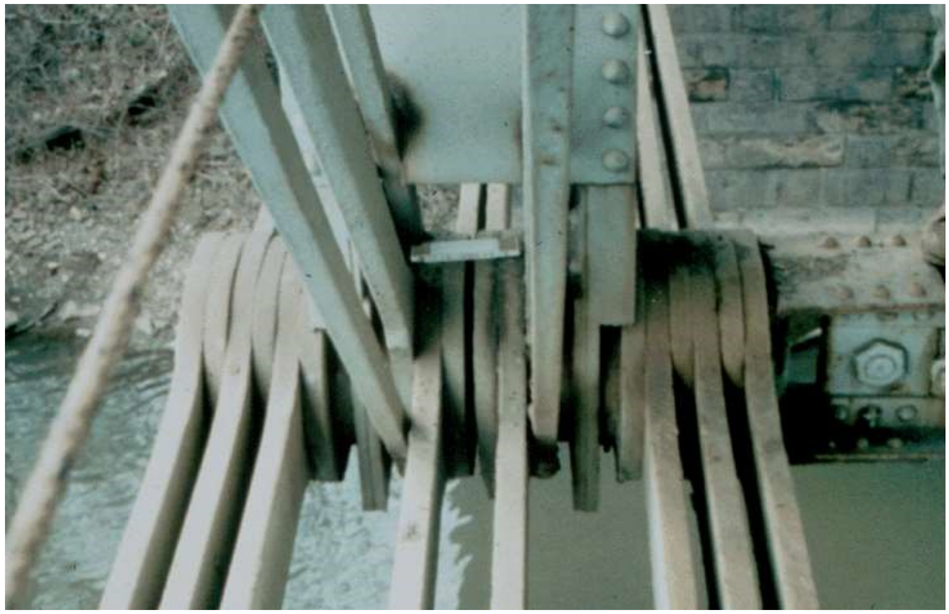

Figure 10.9.16 Tightly Packed Eyebar Connection

Spacers

Spacers or steel filling rings are often wrapped around the pin to prevent lateral movement within the eyebar pack (see Figure 10.9.17).

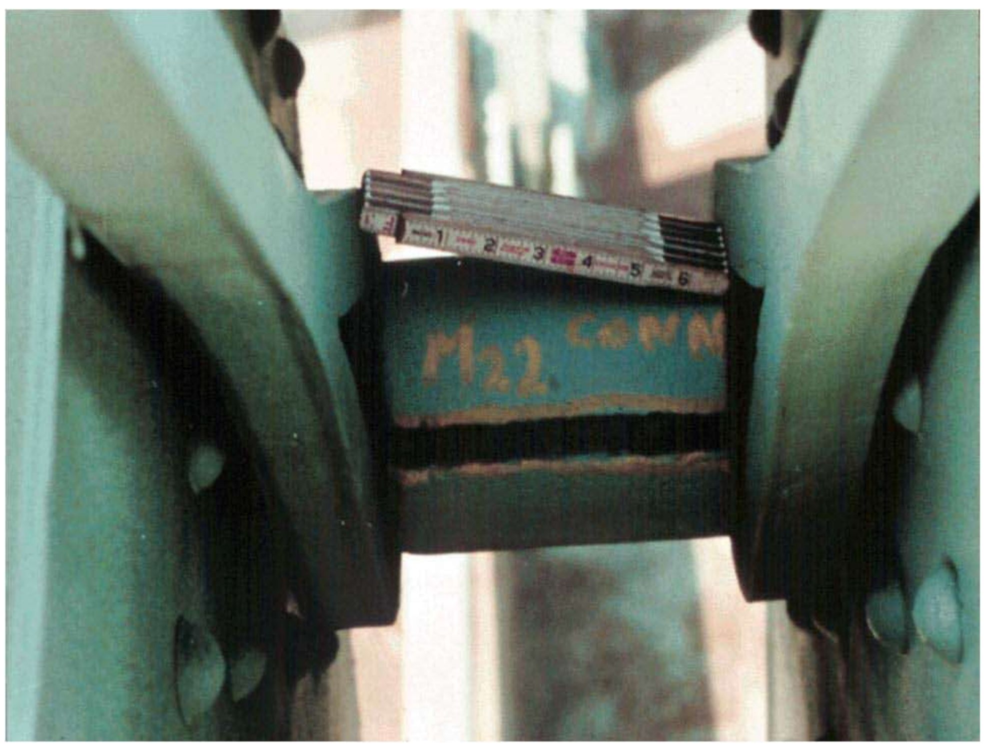

Figure 10.9.17 Steel Pin Spacer or Filling Ring

Redundancy

An internally redundant eyebar member consists of three or more eyebars. Many eyebar members are internally non-redundant, having only one or two eyebars per member (see Figure 10.9.18).

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 5.1 and 6.4.

Figure 10.9.18 Non-redundant Eyebar Member

10.9.3 Overview of Common Deficiencies

Common deficiencies that occur on eyebars and eyebar connections include:

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

See Topic 6.3.5 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges.

10.9.4 Inspection Methods and Locations

Inspection methods to determine other causes of steel deterioration are discussed in detail in Topic 6.3.7.

Methods

Visual

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

Most deficiencies 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 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 deficiency. The use of degreasing spray before and after removal of the paint may help in revealing the deficiency.

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, examine other similar details and similar locations on the bridge.

Advanced Inspection Methods

Several advanced methods are available for steel inspection. Nondestructive methods, described in Topic 15.3.2, include:

- Acoustic emissions testing
- Corrosion sensors
- Smart coatings
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Computed tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Electrochemical fatigue sensor (EFS)

Other methods, described in Topic 15.3.3, 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 10.9.19 and 10.9.20). Most eyebar failures are likely to occur in the forge zone.

Figure 10.9.19 Close-up of the Forge Zone on an Eyebar (Arrow denotes crack)

Figure 10.9.20 Forge Loop is Completely Apart

Tension Zone

Since an eyebar carries axial tension, closely examine the entire length 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 and Load Distribution

Check the alignment of the shank along the full length of the eyebar. The eyebar will be straight since it is a tension member. A bowed eyebar indicates that a compressive force has been introduced (see Figure 10.9.21).

Figure 10.9.21 Bowed Eyebar Member

Misalignment due to buckling can also be caused by movement at the substructure or changes in loading during rehabilitation (see Figure 10.9.22). Eyebars of the same member are suppose to be parallel and evenly loaded.

Figure 10.9.22 Buckled Eyebar due to Abutment Movement

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 10.9.23).

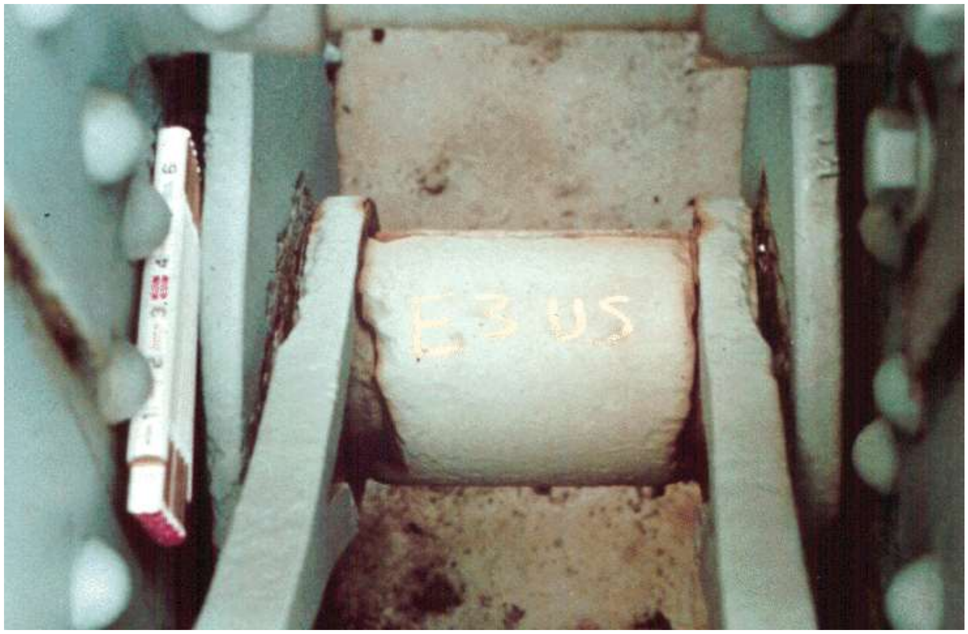

Figure 10.9.23 Corroded Spacer

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 10.9.24).

Figure 10.9.24 Asymmetry at an Eyebar Connection

Load Distribution

Check to determine if any eyebars are loose (unequal load distribution) or if they are frozen at the ends - preventing free rotation. Check for panel point pins or eyebar twisting (see Figure 10.9.25).

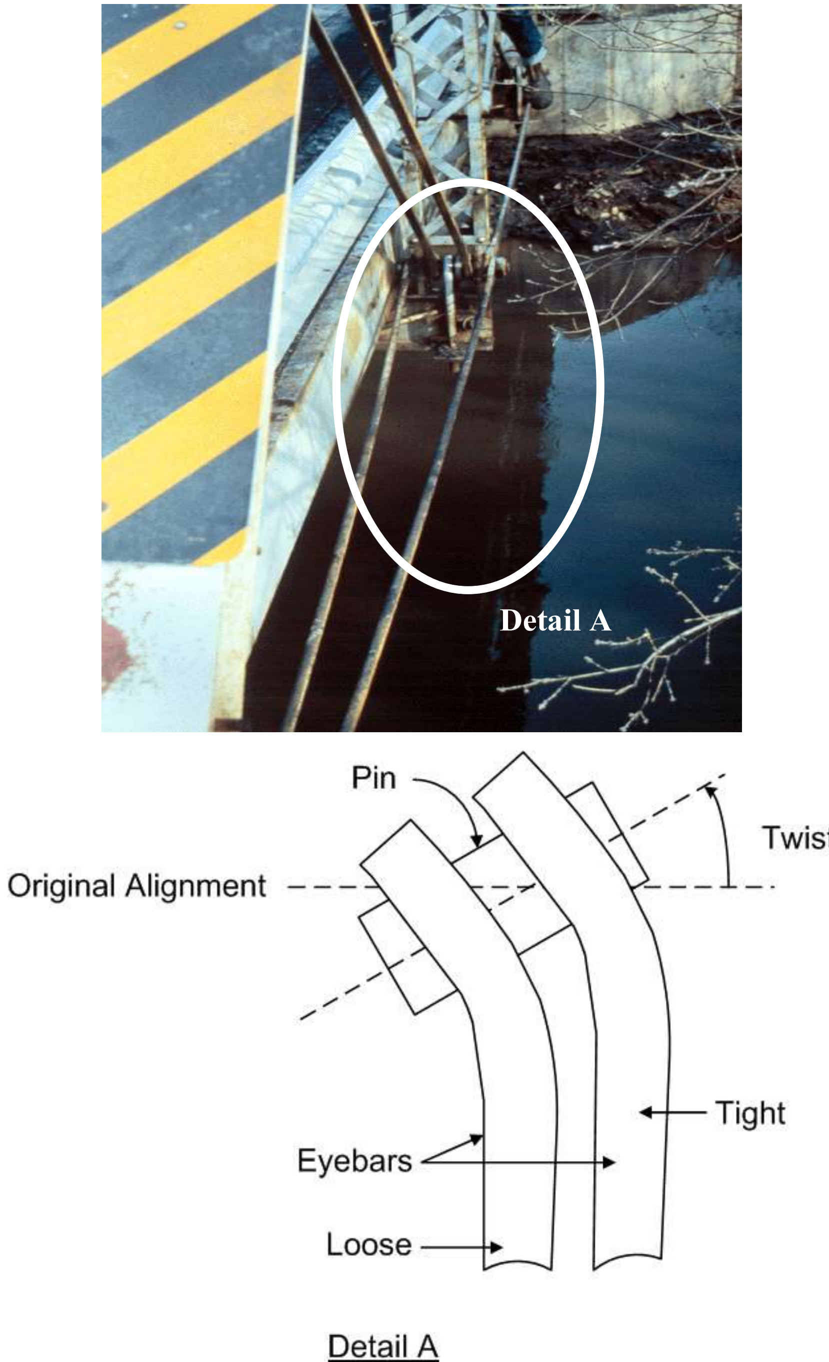

Figure 10.9.25 Eyebar Member with Unequal Load Distribution

Weldments

Evaluate the integrity of any welded repairs to the eyebar (see Figure 10.9.26). Check for any welds used in repairing or strengthening the eyebar, as well as field welds for utility supports (see Figure 10.9.26). Include weld locations in the inspection report so that the engineer can analyze the severity of their effect on the member (see Figure 10.9.27). 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.

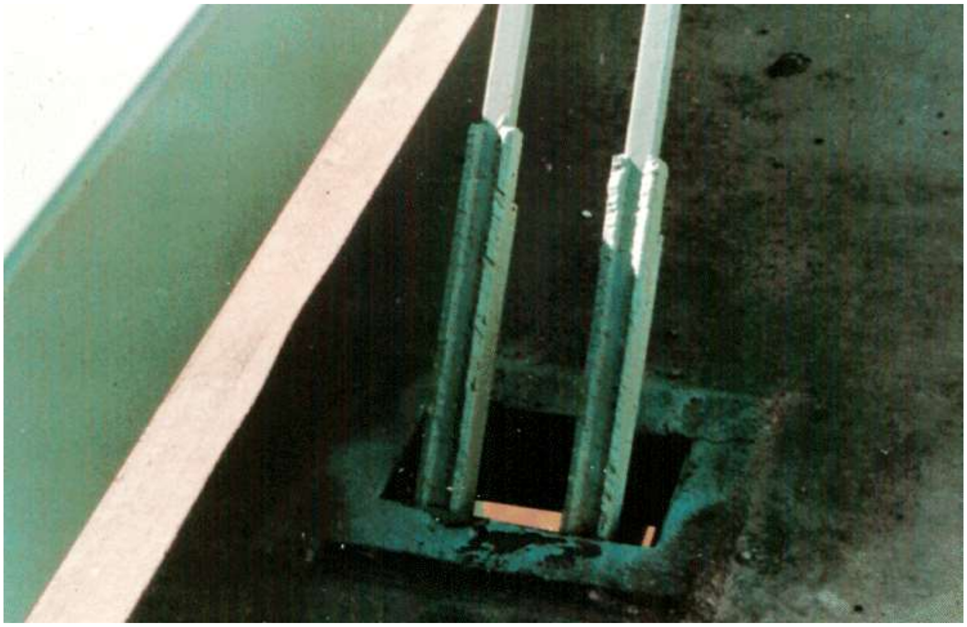

Figure 10.9.26 Welds on Loop Rods

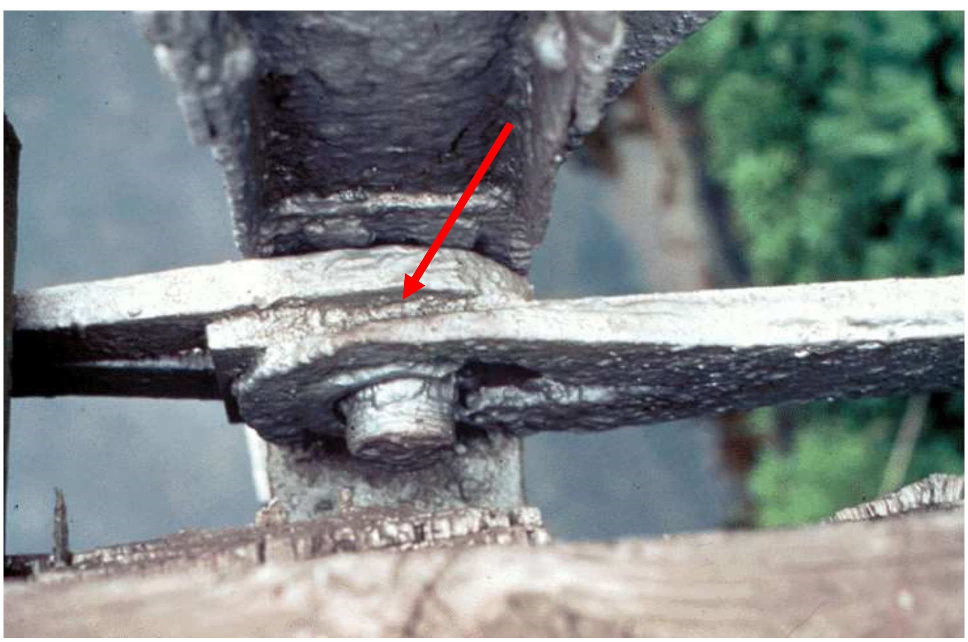

Figure 10.9.27 Welded Repair to Loop Rods

Turnbuckles

Examine any threaded rods in the area of the turnbuckle for corrosion, pack rust, tack welds, cracks, wear and repairs. Inspect the threaded portion of the rod for signs that the turnbuckle is loosening. Turnbuckles are often located in counter diagonals (see Figures 10.9.28 and 10.9.29).

Figure 10.9.28 Turnbuckle on a Truss Diagonal

Figure 10.9.29 Welded Repair to Turnbuckles

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.

Pins

Inspect pins 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 10.9.30).

Figure 10.9.30 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 failure would cause total or partial collapse of the bridge, then that eyebar is considered a fracture critical member. Truss members that have one or two eyebars between panel points are not considered internally redundant (see Figure 10.9.31). Truss members that have three or more eyebars between panel points may be considered internally redundant (see Figure 10.9.32). See Topic 6.4 for a detailed discussion on fracture critical members and types of redundancy.

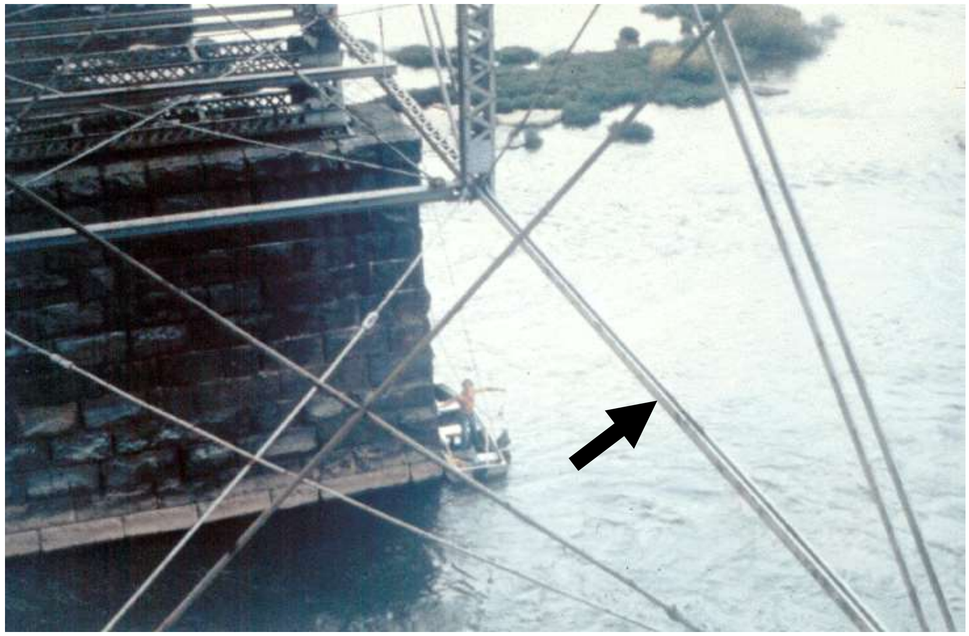

Figure 10.9.31 Fracture Critical Bottom Chord Truss Member: Internally Non-redundant Eyebar

Figure 10.9.32 Fracture Critical Top Chord Truss Member: Internally Redundant Eyebar

10.9.5 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 condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Under NBI component condition rating guidelines, the steel eyebars are considered part of the superstructure and do not have an individual rating. Take into account the condition of the steel eyebar assembly when rating for the superstructure, which 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 NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition 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 component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

Element level evaluation does not have specific National Bridge Elements or Bridge Management Elements for steel eyebars. Therefore, individual states may choose to create their own element for eyebars or use the AASHTO Bridge Management Elements that best describe the steel eyebars. In an element level condition state assessment of steel eyebars, possible AASHTO National Bridge Elements (NBEs) or Bridge Management Elements (BMEs) that relate closest to a steel eyebar include:

NBE No.	**Description**
Superstructure	
120	Steel Truss
141	Steel Arch
161	Pin, Pin and Hanger assembly, or both

BME No.	**Description**
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating

The unit quantity for steel trusses and arches is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for steel protective coating is square feet, with the total area distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of steel eyebar systems:

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
362	Superstructure Traffic Impact (load capacity)
363	Steel Section Loss

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

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Inspection and Evaluation of Bridge Bearings

Chapter 11
Inspection and Evaluation of Bridge Bearings

Topic 11.1 Bridge Bearings

11.1.1 Introduction

A bridge bearing is a superstructure element that provides an interface between the superstructure and the substructure. The three primary functions of a bridge bearing are:

- To transmit loads from the superstructure to the substructure
- To allow rotation caused by permanent (dead load) and transient (live load) deflection.
- To permit horizontal movement of the superstructure due to thermal expansion and contraction (expansion bearings only)

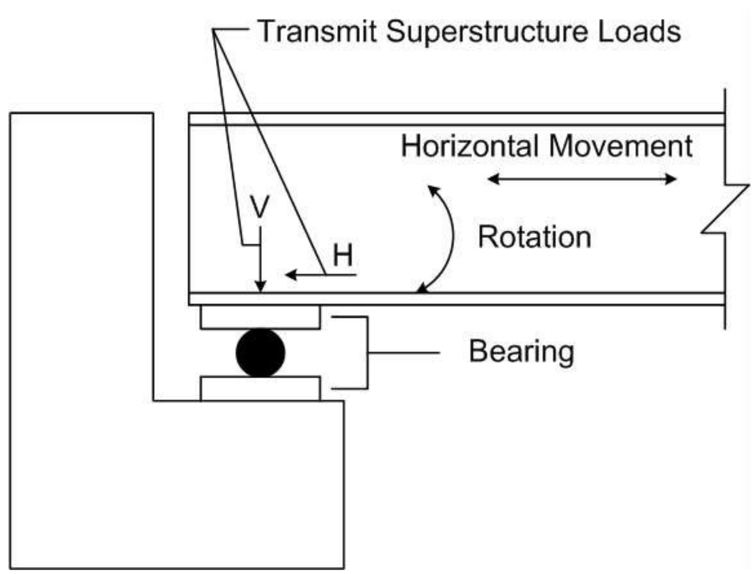

Figure 11.1.1 Three Functions of a Bearing

Fixed and Moveable Bearings

The operation of bridge bearings is critical to the safety and load-carrying capacity of a bridge. When bridge bearings do not operate properly:

- Expansion and contraction movements that are not accommodated by bearings cause internal axial stresses.
- End rotations that are not accommodated by bearings cause internal bending stresses, including high stresses in the substructure.
- Excessive forces may result in damage or instability of the superstructure or substructure.

Bearings that do not allow for horizontal translation or movement of the superstructure are referred to as fixed bearings. Bearings that do allow for horizontal translation or movement of the superstructure are known as moveable bearings. Both fixed and moveable bearings permit rotation that occurs as loads are applied or removed from the bridge (see Figure 11.1.2).

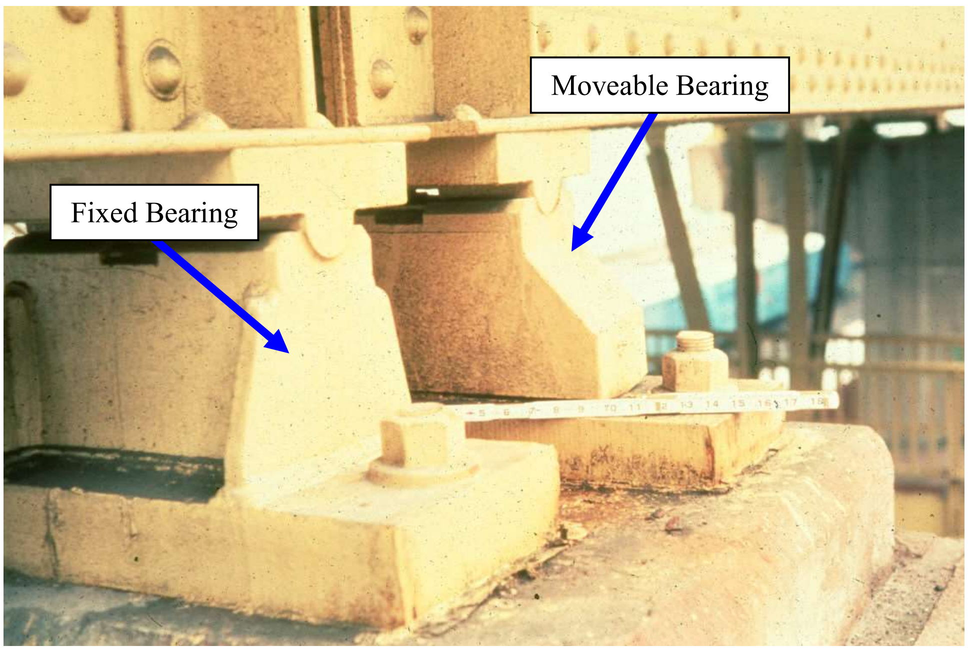

Figure 11.1.2 Fixed and Moveable Bearings

11.1.2 Four Basic Elements of a Bearing

A bridge bearing consists of four basic elements; sole plate, bearing or bearing device, masonry plate and anchor bolts (see Figure 11.1.3).

Sole Plate

The sole plate is responsible for distributing forces from the superstructure to the bearing device. It is a steel plate that is attached to the bottom of girders, beams or truss chords. A sole plate may also be embedded into the bottom flange of a prestressed concrete girder. With concrete beams, girders or slabs, the lower flange or bottom of the section may function as the sole plate.

Bearing or Bearing Device

The bearing or bearing device is secured to the sole plate and masonry plate and provides the function of transmitting the forces from the sole plate to the masonry plate.

Masonry Plate

The masonry plate is a steel plate that is attached to the bearing seat of an abutment or pier. The masonry plate serves to distribute vertical forces from the bearing to the substructure unit.

Anchor Bolts

The anchor bolts connect the bearing to the substructure unit. Anchor bolts are designed to restrain the masonry plate from horizontal translation. The anchor bolts can, however, pass through or alongside the moveable bearing element to provide restraint against transverse movement. The local or governing agency requirements need to be checked to determine the minimum bolt diameter and the minimum embedded length.

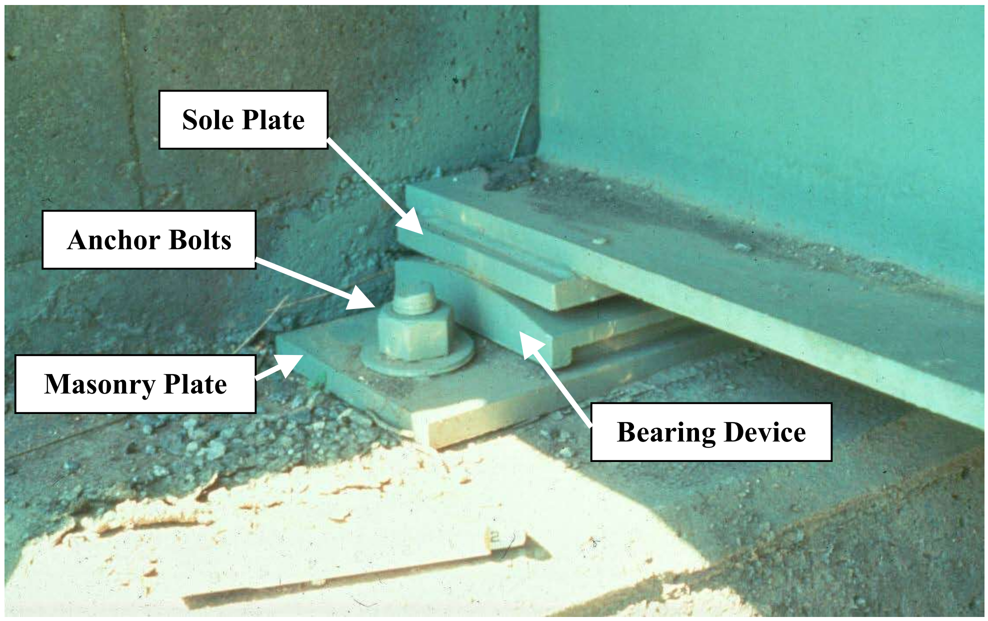

Figure 11.1.3 Elements of a Typical Bridge Bearing

Not every bearing has these specific components. Every bearing does, however, have features that fulfill the function of each of these components.

11.1.3 Bearing Types and Functionality

The American Association of State Highway and Transportation Officials (AASHTO) defines six bearing element types. Each type reflects the overall operation of the bearing.

These six types of bearings are:

- Moveable bearings
- Elastomeric bearings
- Pot bearings
- Disk bearings
- Fixed bearings
- Enclosed or concealed bearings

These bearing types along with "uncommon" bearings are presented in this section.

Moveable Bearings

Various moveable bearing types have evolved out of the need to accommodate superstructure movement, both reliably and efficiently. Seasonal changes impact the maximum and minimum ambient temperatures. Moveable bearings are responsible for allowing movement due to these fluctuations in temperature. Types of moveable bearings include:

- Sliding plate bearings
- Roller bearings
- Rocker bearings

Sliding Plate Bearings

Several types of sliding plate bearings have been used in bridges over the years. They are primarily used on structures with a span length less than 40 feet. Longitudinal movement is provided by one plate sliding upon another. The basic difference between types of sliding plate bearings is the method of lubrication. Among the various types of plates are those presented below.

Lubricated Steel Plates

The first generation of lubricated steel plates consisted of two steel plates with the bearing devices milled smooth (see Figure 11.1.4). Lubrication between the plates consisted of grease, graphite and tallow. Unfortunately, the lubricant typically held dirt, which absorbed moisture and eventually corroded and froze the bearing. “Freezing,” as used to describe bearings, indicates that the bearing movement or rotation is restricted due to corrosion, mechanical binding, dirt buildup, or other interference. The bearing cannot move or rotate as intended.

The next generation of lubricated steel plates consisted of a small plate sliding on a considerably larger one. The theory behind this was that if the contact area were smaller, the forces transmitted overcame the freezing forces. In application, the smaller plate actually wore a groove in the larger one, eventually freezing the bearing anyway.

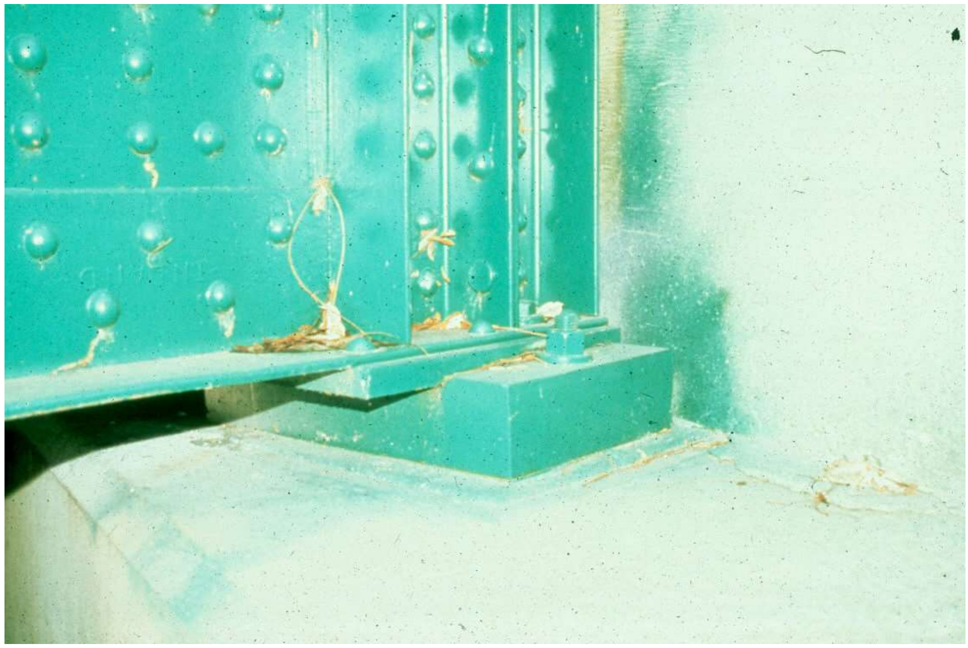

Figure 11.1.4 Lubricated Steel Plate Bearing

Lead Sheets Between Steel Plates

By placing a thin lead sheet between the steel plates, it is possible to keep the plates from freezing together when they corrode. Lead sheets are used to reduce corrosion between the plates, thereby providing more freedom of movement. However, in this type of bearing, the lead has a tendency to work its way out from between the plates.

Bronze Bearing Plates

A bronze bearing plate was introduced to avoid the corrosion problems of steel plates in contact with one another (see Figure 11.1.5). Since bronze does not corrode, it was used to maintain the freedom of movement. Although corrosion is reduced, the bronze, which is soft material, becomes worn due to trapped dirt and the action of expansion and contraction. Eventually, a freezing of the plates may take place.

Figure 11.1.5 Bronze Sliding Plate Bearing

Asbestos Sheet Packing Between Metal Plates

A graphite-impregnated asbestos sheet has been used between steel bearing plates to provide some movement in spans less than 40 feet.

Self-Lubricating Bronze Bearings

The self-lubricating bronze bearing was developed to ensure a graphite lubricant between bearing plates, regardless of their wear. Portions of the face of the bearing were removed and replaced with a graphite compound, which continuously lubricated the bearing surfaces. Some manufacturers claim that these bearings are corrosion resistant and never require any maintenance. The bearings may be maintenance free if they are kept free from dirt and abrasive dust.

These bearings are widely available in many different forms, including plates, plates with one side cut to a radius, and half cylinders. The flat (top) side provides translational movement, while rotational movement is provided by the radius side (bottom) (see Figure 11.1.6).

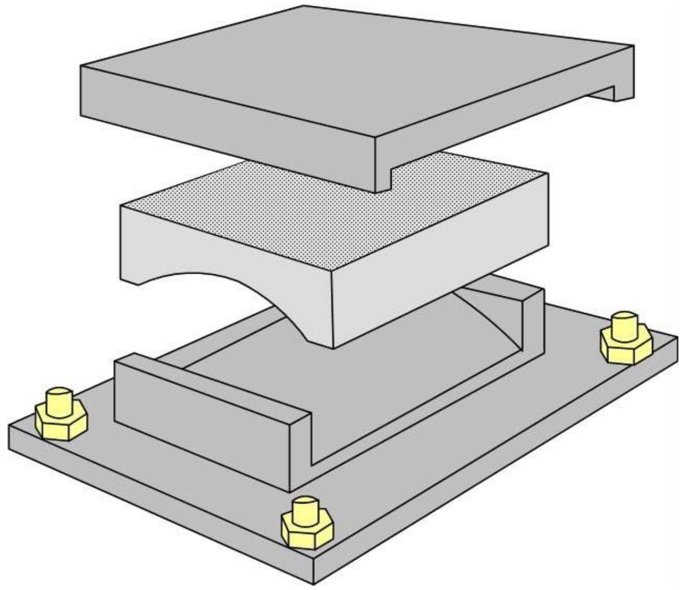

Figure 11.1.6 Self-lubricating Bronze Sliding Plate Bearing

Roofing Felt or Tar Paper

Another type of bearing consists of oil-soaked felt or tar paper that has been lightly coated with graphite. Several layers are placed on the bridge seat with the superstructure placed directly on it. This is a simple but effective bearing that is commonly used on short span concrete slabs and girders that sit on concrete abutments. These bearing types provide limited horizontal movement.

PTFE on Stainless Steel Plates

A compound known as "polytetrafluoroethylene" (PTFE) has the lowest coefficient of friction of any of the commonly available materials, making it quite desirable for use in bridge bearings.

Various types of bearings have been offered to take advantage of PTFE's characteristics. Today, bearings using PTFE have a sheet of stainless steel underneath the sole plate to slide across the PTFE. Pure PTFE has a low compressive strength and a high coefficient of thermal expansion. To make it suitable for use in bridge bearings, PTFE is combined with suitable fillers. These fillers are typically glass fiber and bronze. While giving strength to the PTFE, these fillers do not increase its low coefficient of friction.

Roller Bearings

A roller bearing consists of a cylinder that “rolls” between the sole plate and masonry plate as the superstructure expands and contracts (see Figure 11.1.7). Roller bearings are used in a wide variety of forms including single rollers and roller nests.

Single Roller Bearings

The single roller is one of the most widely used moveable bearings. Rollers can vary in size, with specified diameters ranging from 6 to 15 inches. While the larger rollers are less susceptible to corrosion problems, dirt may get trapped in the contact areas along the top and bottom of the bearing. This enables moisture absorption, eventually deteriorating the bearing surface. However, because only a small portion of the roller actually becomes corroded, the corroded roller can be rotated and another portion of the roller surface can be used. Many single roller bearings are made of corrosion resistant steel.

An unrestrained roller may gradually work itself out from underneath the bridge superstructure. For this reason, pintle pins are used to keep the roller in place. These pins fit tightly into the roller but loosely into the upper and lower plates. The loose fit allows for the necessary structure movement.

Figure 11.1.7 Single Roller Bearing

Roller Nest Bearings

First used in steel bridges in the early 1900's, roller nests consist of a group of rollers, each about 1.5 to 2 inches in diameter. When clean, roller nests work well. However, the small rollers offer many places for dirt and moisture to collect. This results in wear and corrosion of the rollers, and ultimately results in bearing failure. Attempts to seal this bearing require careful maintenance of protective covers and skirts, which are typically unsuccessful (see Figure 11.1.8).

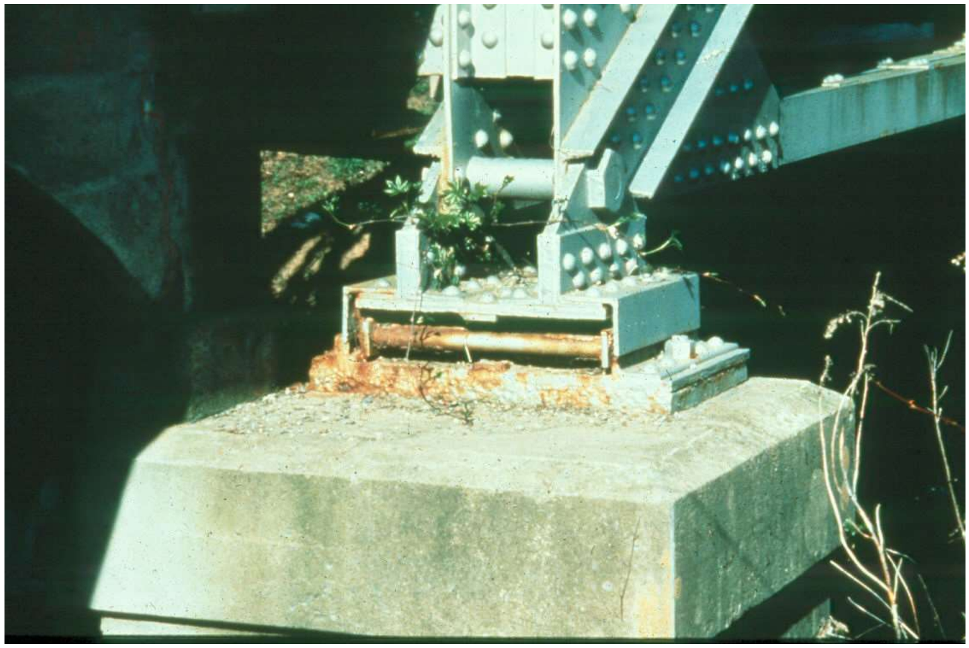

Figure 11.1.8 Roller Nest Bearing

Rocker Bearings

The rocker bearing functions in a similar manner to the roller bearing and is generally used where a substantial amount of longitudinal movement is required (see Figure 11.1.9). As with roller bearings, rocker bearings come in different forms, such as segmental rockers, rocker nests, and pinned rockers.

Figure 11.1.9 Rocker Bearing

Segmental Rocker Bearings

Segmental rocker bearings evolved out of the use of large rollers. When the rollers get up to 20 inches in diameter, they become very heavy and difficult to handle. Since only a small portion of the roller bearing is actually in contact between the sole plate and masonry plate, the unused portion may be cut away and a substantial weight savings obtained (see Figure 11.1.10).

Larger segmental rockers have also been fabricated from rectangular blocks, rounded at both ends, which allow the bearing to roll and the horizontal movement to take place.

Figure 11.1.10 Segmental Rocker Bearing

Rocker Nest Bearings

A group of several rockers forms a rocker nest bearing (see Figure 11.1.11). Similar to roller nests, rocker nests provide many small areas for dirt and moisture to collect. Moisture can lead to corrosion which may result in a bearing failure.

Figure 11.1.11 Segmental Rocker Nest Bearing

Pinned Rocker Bearings

The pinned rocker is the most popular rocker bearing in use today. The top is basically a large pin and helps to keep the bearing aligned correctly. Longitudinal movement is provided by the rotation allowed by the pin and the rolling provided by the rocker (see Figure 11.1.12). When exposed to adverse environmental conditions, however, the pin can corrode and freeze. Pinned rocker bearings can be quite large and are commonly used for relatively long spans and heavy loads. Holes in the radius portion of the bearing may be slotted to accommodate longitudinal movement.

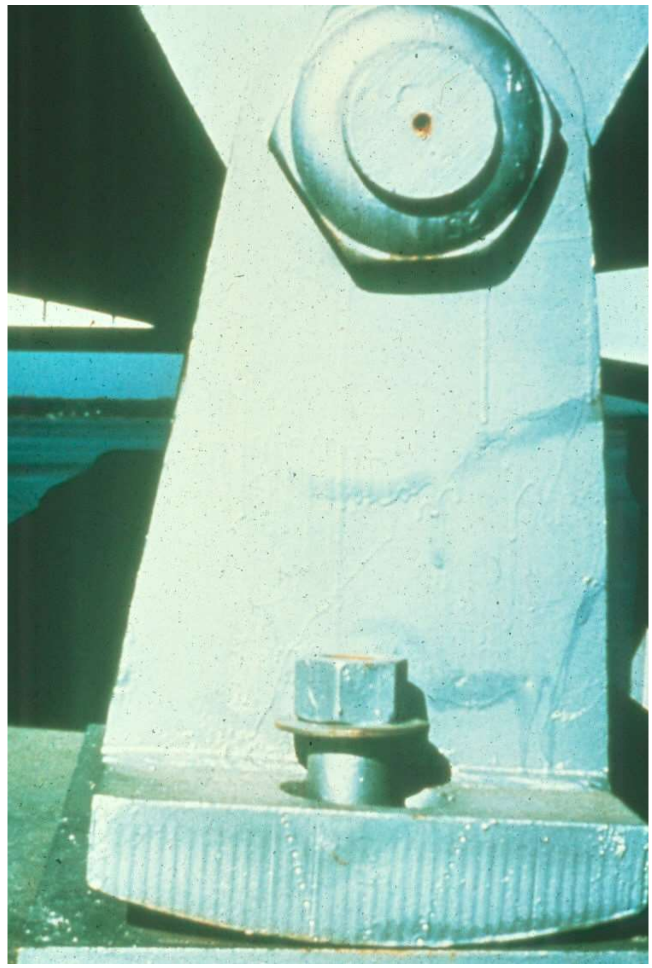

Figure 11.1.12 Pinned Rocker Bearing

Elastomeric Bearings

Elastomeric bearings include both plain and laminated neoprene pads. Neoprene is a heavy rubber-like material that deforms slightly under compression or shear.

Plain Neoprene Pads

A plain neoprene bearing consists of a rectangular or circular pad of pure neoprene and is used primarily on short span, prestressed concrete structures (see Figure 11.1.13). Neoprene bearings are popular for steel beam bridges as well. Expansion and contraction are achieved through a shearing deformation of the neoprene. These bearings are typically of uniform thickness.

Various means are used to prevent the neoprene bearing from "walking" out of position from under a beam. An epoxy compound has been used to bond the pad to the beam and the bridge seat, but it has not always been successful.

Figure 11.1.13 Plain Neoprene Bearing Pad

Laminated Neoprene Pads

A laminated neoprene bearing is simply a stack of neoprene pads with steel or fiberglass plates separating them (see Figure 11.1.14). The plates are not visible if the entire bearing is encased in neoprene. Laminated bearing pads are used on longer structures where the expansion and contraction requirements and the vertical superstructure loads are greater.

Although a single, thicker pad could conceivably do the job of the laminated bearing, excess bulging and wearing of the pad dramatically decreases its useful life. The laminated bearing eliminates this excess bulging and allows expansion and contraction without excessive wear.

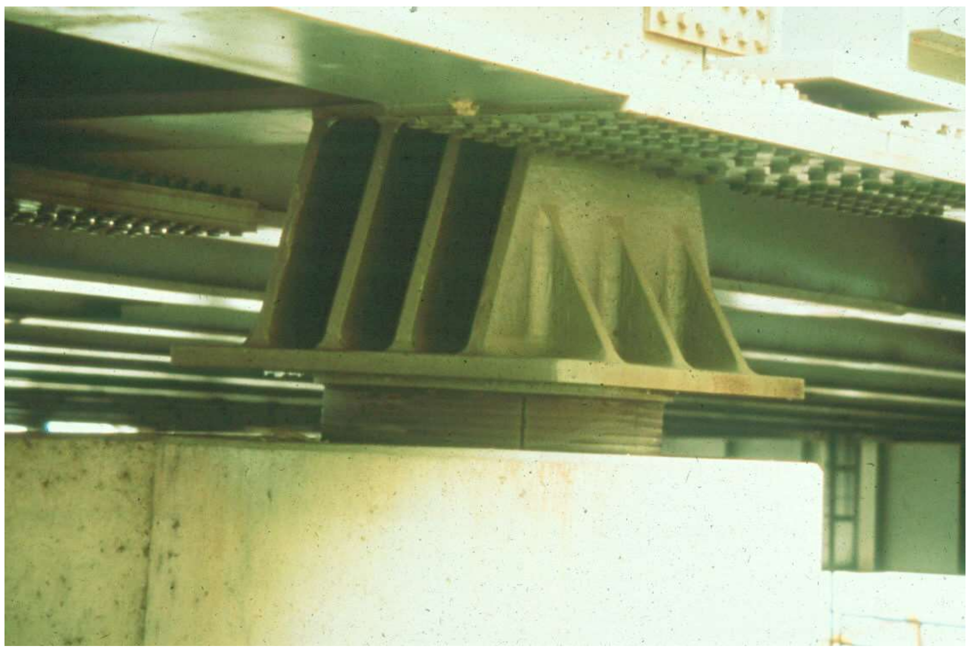

Figure 11.1.14 Laminated Neoprene Bearing Pad

Pot Bearings

Pot bearings allow for the multi-dimensional rotations of a structure.

Neoprene Pot Bearings

A neoprene pot bearing has a stainless steel plate that is attached to the sole plate. This stainless steel plate slides on a polytetrafluoroethylene (PTFE) disk. The PTFE disk is attached to a steel piston, which rests on a neoprene pad, allowing for the rotation of the structure. The pad rests in a shallow steel cylinder that is attached to the masonry plate. This cylinder is referred to as the pot. Guide bars in the pot bearing restrict transverse movement (see Figure 11.1.15).

A fixed bearing version of this configuration does not possess the stainless steel plate or the PTFE disk.

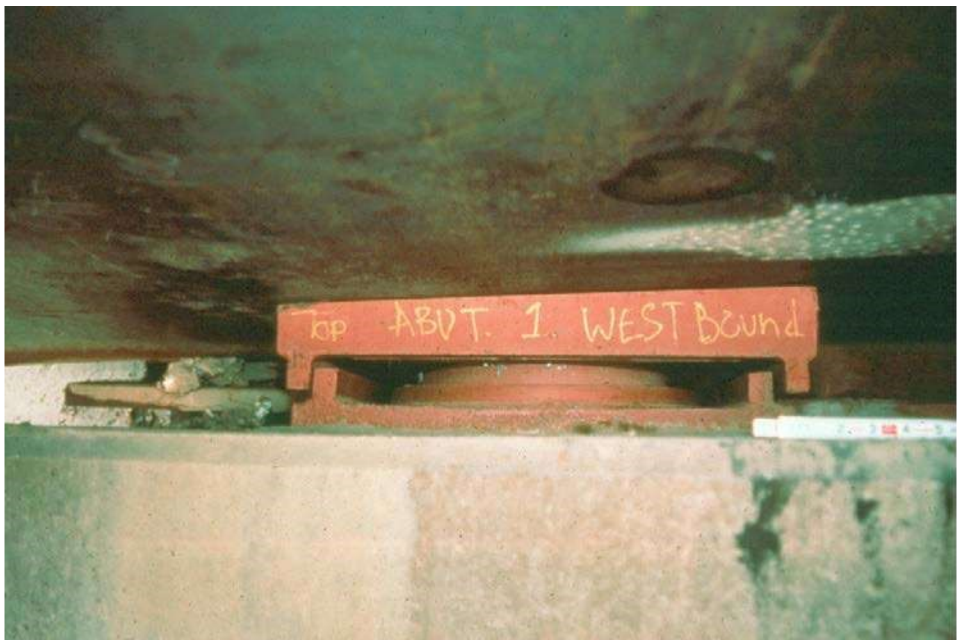

Figure 11.1.15 Neoprene Pot Bearing with Guide Bars

Disk Bearings

Disk bearings typically have a very low profile. As with pot bearings, disk bearings provide a high-capacity solution for bridges. The difference between a pot bearing and a disk bearing is the bearing device. Disk bearings accommodate rotations through the deformation of a hard plastic disk that is typically unconfined (see Figure 11.1.16).

Disk bearings may be configured to restrict translational movement or provide movement in one or more directions through a PTFE surface, stainless steel plates and guide bars (if applicable).

Figure 11.1.16 Disk Bearing

Fixed Bearings

Fixed bearings are classified as only allowing rotational movement. They rely on the rotation around the pins to accommodate end rotation. Fixed bearings also prevent translational (or horizontal) movement.

Figure 11.1.17 shows a fixed bearing. As with the moveable bearing, the vertical superstructure loads are transmitted down to the fixed bearing and then passed down to the substructure. In addition to transmitting vertical loads, a fixed bearing also transmits horizontal loads from the superstructure to the substructure. The fixed bearing also accommodates any rotation resulting from the transient (live load) deflection, but does not provide for any longitudinal movement.

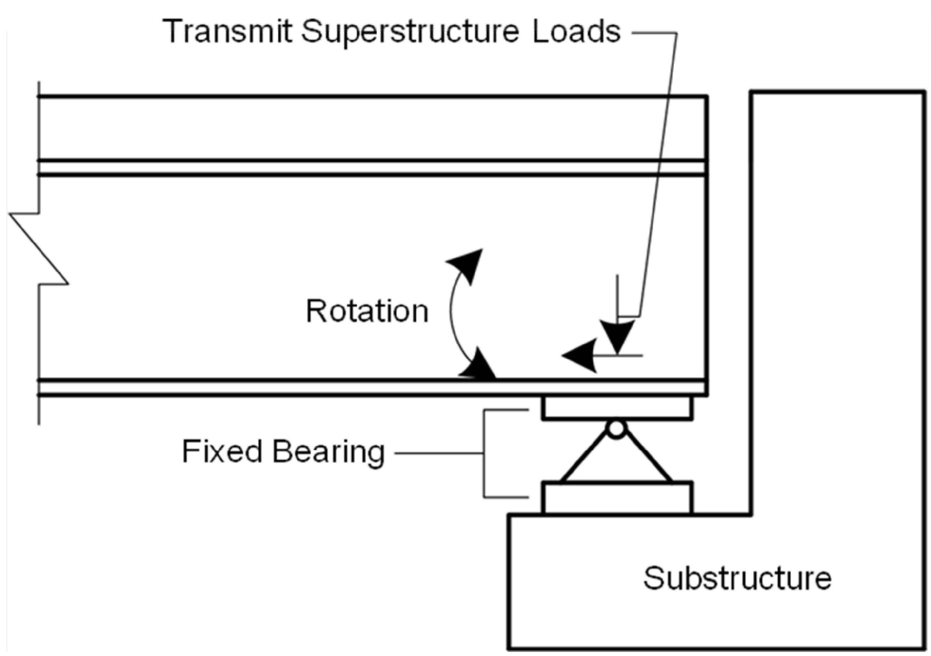

Figure 11.1.17 Fixed Bearing

Enclosed or Concealed Bearings

For some bearings, the line-of-sight between the inspector and the bearing may be compromised. These bearings are said to be enclosed or concealed bearings and cannot be adequately evaluated through a visual inspection (see Figure 11.1.18).

Examples of bearings that may be considered enclosed or concealed include bridges with integral end diaphragms.

It is important to note the difference between a bridge with concealed bearings and a bridge with integral abutments, which has no bearings.

Figure 11.1.18 Enclosed or Concealed Bearing

Uncommon Bearings

Uncommon bearings are not a specific bearing element type defined by AASHTO. Instead, uncommon bearings represent specific types of bearings that are still in service, but are no longer utilized in modern bridge construction.

The AASHTO-defined bearing element types can still be used to inventory these uncommon bearings for element level inspection (moveable, expansion, pot, disk, or fixed bearings, or enclosed/concealed if they cannot be visually inspected), which have been included in parenthesis after the bearing name.

Pin and Link Bearings (Moveable or Enclosed/Concealed Bearings)

The pin and link bearing is typically used on continuous cantilever structures to support the ends of a suspended span. It can also be used as a type of restraining device, which is discussed later in this topic. This bearing type consists of two vertically oriented steel plates pinned at the top and bottom to allow longitudinal movement (see Figure 11.1.19). A disadvantage of this type of bearing is that, as the superstructure expands and contracts, the deck rises and falls (but only slightly). Another disadvantage is that pins can fracture when frozen by corrosion.

Figure 11.1.19 Pin and Link Bearing

Restraining Bearings (Moveable, Pot, Disk, Fixed or Enclosed/Concealed Bearings)

Restraining bearings serve to hold a bridge down in the case of uplift. Uplift usually occurs on cantilever anchor spans. The devices used to resist uplift can be as simple as long bolts running through the bearings on short span bridges or as complex as chains of eyebars on larger structures (see Figure 11.1.20). Lock nuts are used with bolted restraining devices to resist uplift. Pin and link members are also used as restraining devices. The type of restraining device used depends on the magnitude of the uplift force.

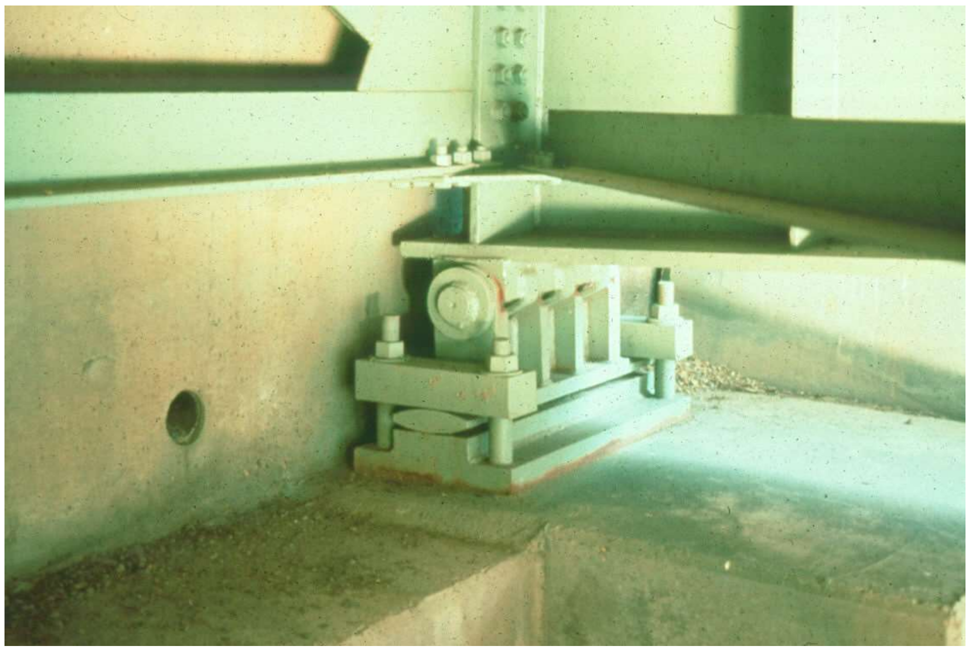

Figure 11.1.20 Restraining Bearing

Isolation Bearings (Moveable, Elastomeric or Pot Bearings)

Isolation bearings were developed to protect structures against extreme horizontal loadings due to earthquakes. Isolation bearings may also be used to accommodate horizontal movements due to large truck loadings. These bearings operate by allowing larger than normal relative movement, which reduces lateral loads applied to the structure.

Types of isolation bearings include lead-core isolation, friction pendulum and high-damping rubber.

Lead Core Bearings (Elastomeric Bearings)

Lead core bearings are a type of isolation bearing. These bearings are similar to laminated neoprene bearings in that they are a sandwich of neoprene and steel plates (see Figures 11.1.21 and 11.1.22). These bearings contain a lead core that stiffens the bearing to help resist the effects of high horizontal bridge loading. During seismic loads, the lead core is designed to yield, thereby making the bearing more flexible and allowing it to isolate the bridge from the effects of earthquake motion. The downside to lead core bearings is the possibility of requiring replacement after a seismic event, since the lead core may have yielded. However, the cost to replace these bearings is favorable considering the damage an earthquake may cause to the bridge structure.

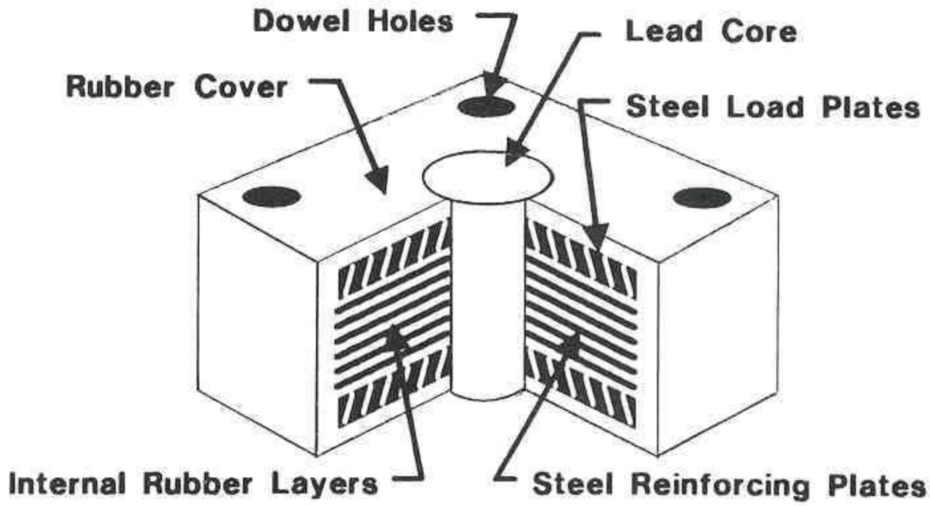

Figure 11.1.21 Sketch of a Lead Core Isolation Bearing

Figure 11.1.22 Lead Core Isolation Bearing

Friction Pendulum Bearings (Moveable or Pot Bearings)

Another bearing type designed to protect against earthquake damage is a friction pendulum bearing. These bearings are designed to reduce lateral loads and shaking movements transmitted to the structure (see Figure 11.1.23). They can protect structures and their contents during strong, high magnitude earthquakes and can operate near fault pulses and deep soil sites.

Friction pendulum bearings incorporate the characteristics of a pendulum to lengthen the natural period of the isolated structure so as to avoid the strongest earthquake forces (see Figure 11.1.24). The period of the bearing is selected by choosing the radius of curvature of the concave surface. It is independent of the loads of the superstructure. Torsion motions of the substructure are minimized because the center of stiffness of the bearings automatically coincides with the center of mass of the superstructure.

Figure 11.1.23 Friction Pendulum Bearing

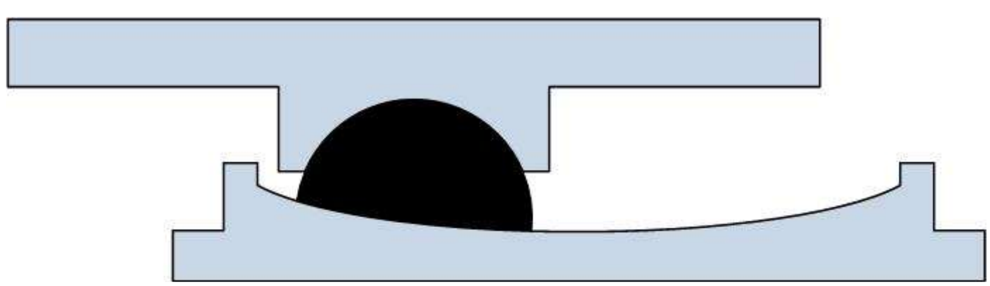

Figure 11.1.24 Sketch of a Friction Pendulum Bearing

High-Damping Rubber Bearings (Elastomeric Bearings)

High-damping rubber bearings were also developed to protect structures from the damage of earthquakes. Under service load conditions, the bearing provides support in a similar fashion to elastomeric bearings. Its rigidity is provided by a high rubber modulus at small shear strains. During an earthquake, a special hysteretic rubber compound in the bearing dissipates the energy of the earthquake. As a result, the structure is isolated from the shaking forces of the earthquake and is less likely to collapse.

Spherical Pot Bearings (Pot Bearings)

Spherical bearings allow for multi-directional rotation. They are similar to neoprene pot bearings, except that the polytetraflouroethylene (or PTFE) disk is bonded to a spherical aluminum casting that rotates within a PTFE-coated pot. The pot is attached to the masonry plate.

Anchorage bolt holes are incorporated on the sliding plate. Directly beneath the sliding plate, a PTFE disk is bonded to a spherical aluminum casting (that serves as the bearing device). This disk allows for multi-directional translation between the sliding plate and bearing device. Rotational movement is then provided by the curved surface of the bearing device and PTFE-coated pot. The pot may be cylindrical (as shown in Figure 11.1.25) or rectangular in shape. Beneath the pot is the masonry plate, which allows for the bearing to be anchored to the substructure unit.

Spherical pot bearings may also incorporate exterior guide bars. These guide bars function similar to those found on pot bearings and disk bearings, limiting or preventing horizontal translation.

A fixed bearing version of this configuration has the upper aluminum casting attached to the sole plate and incorporates edge-guide bars. Fixed spherical bearings also do not utilize stainless steel plates on a PTFE disk.

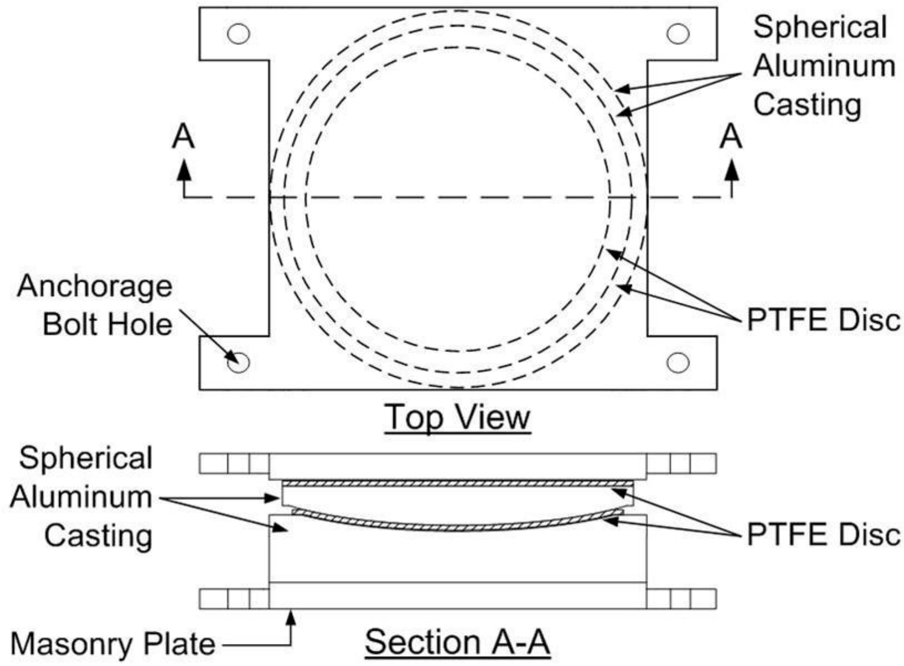

Figure 11.1.25 Spherical Pot Bearing

11.1.4 Inspection Methods and Locations

The inspection of bearings is broken down into three different categories:

- General
- Steel bearings
- Elastomeric bearings

For each of these different categories, the inspection of bearings may utilize one or more of the three inspection methods: visual, physical and advanced.

Most deficiencies are first detected by visual inspection. This inspection method is a hands-on inspection or inspection where the inspector is close enough to touch the area being inspected. More exact visual observations can also be employed using a magnifying unit after cleaning the suspect area.

Physical inspection methods may also be employed into the categories of inspection. These methods include the use of a wire brush, grinder or sand blaster. Degreasing spray may also be used to help remove paint and reveal the deficiency. Measurement of the deficiencies to check to irregular dimensions or section loss is also a physical inspection method.

Several advanced inspection methods are available for steel bearings. Applicable advanced methods for steel bearings are listed below within the Steel Bearings subtopic.

General

When inspecting a bearing, the inspector first determines if the bearing was initially intended to be fixed or moveable. If the bearing was designed to allow for translation or movement of the superstructure, then it is a moveable bearing; if not, then it is a fixed bearing. The inspector refers to the design plans if available. It is critical that the inspector assess whether moveable bearings still allow for translation or horizontal movement.

Check that bearings are properly aligned horizontally and vertically, with the bearing surfaces clean and in full contact with each other. If only partial contact is made, damage can occur to the bearing device, superstructure, or substructure. This damage can occur when a girder has moved horizontally so that the bearing rests on only a portion of the masonry plate. In this situation, the full load of the superstructure is applied to a smaller area on the masonry plate and results in a higher stress that could crush the bridge seat. Also, such redistribution of the load may cause buckling to occur in the girder web of the superstructure above the bearing. Distress in the form of cracking or spalling under the bearings may be an indication that the bearings are not handling the anticipated horizontal movement of the superstructure.

Bearings need to have a suitable support. A distance of several inches needs to exist between the edge of the masonry plate and the edge of the supporting member, abutment, or pier. Note any loss to the supporting member near the bearing (e.g., spalling of a concrete bridge seat) (see Figure 11.1.26).

Bearings and the concrete substructure lateral shear keys on skewed bridges are inspected for binding and damage due to the creep effect of the bridge (i.e., the tendency of the bridge to move laterally along the skew).

Record the temperature during the inspection. Special thermometers with magnets are available to measure the actual temperature of the superstructure and bearing. Measure the movement of the bearings and compare it to the recorded temperature. The bearings need to be in the expanded position for temperatures greater than the design (or average) temperature and in the contracted position for temperatures less than the design (or average) temperature. The design temperature is 68 degrees Fahrenheit unless otherwise noted.

Small maintenance problems with bearings can grow progressively worse if ignored, eventually causing major problems for the bridge. Inoperable bearings can transfer significant overstress to the superstructure or substructure.

Figure 11.1.26 Spalling of Concrete Bridge Seat Due to High Edge Stress

Inspection of Steel Bearings

Various metallic materials have been used in bearings, including steel, bronze, aluminum, lead, and cast iron. However, steel is by far the most prominent and also the most susceptible to deterioration, while most other materials are either non-corrosive or corrosion-resistant. Consequently, the following discussions concentrate the inspection of steel bearings.

Most defects in steel bearings are first detected by a visual assessment. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being evaluated, is required. Use a wire brush, grinder or hammer to remove any loose or flaked steel. Use appropriate personal protective equipment when disturbing potentially hazardous coatings and materials.

Several advanced methods may be required to evaluate the steel bearing.

Nondestructive methods for steel bearings, described in Topic 15.3.2, include:

- Acoustic emissions testing
- Dye penetrant
- Magnetic particle
- Radiography testing
- Ultrasonic testing (see Figure 11.1.27)
- Eddy current

Figure 11.1.27 Ultrasonic Testing Inspection of a Pin in a Bearing

Corrosive Forces

Check all bearing elements for any pitting, section loss, deterioration and debris build-up, which can cause the bearing to bind up or freeze (see Figure 11.1.28). Evidence of a frozen bearing includes bending, buckling, improper alignment of members, or cracks in the bearing seat. Check for bent, broken or missing anchor bolts.

Figure 11.1.28 Heavy Corrosion on a Steel Rocker Bearing

Looseness

Loose bearings can be identified by noise at the bearing or observing bearing movement when loaded. Loosening may be caused by any of the following (see Figures 11.1.29 through 11.1.31):

- Settlement or movement of the bearing support away from the portion of the bridge being supported
- Excessive rust or corrosion, which results in a loss of material in the bearing itself
- Excessive deflection or vibration in the bridge
- Loose, missing or broken fasteners that are used to attach the bearing to either the superstructure or the substructure
- Worn bearing elements
- Uplift in curved bridge superstructures
- Pavement pressure, which drives the backwall into the beams

Specific inspection items for the various types of steel bearings are detailed following this paragraph.

Figure 11.1.29 Rocker Bearing with Excessive Horizontal Movement

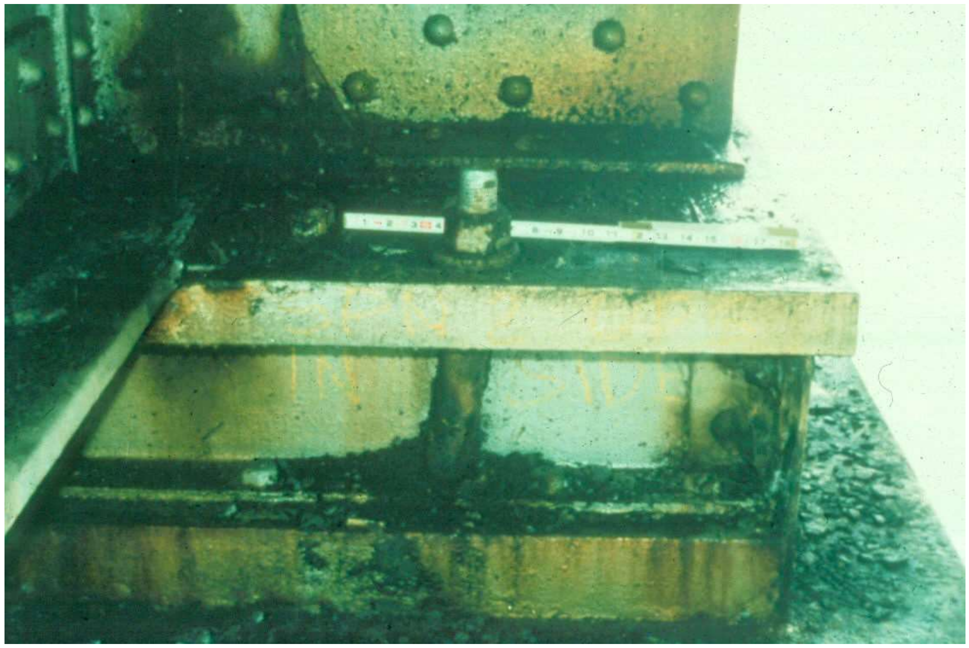

Figure 11.1.30 Bent Anchor Bolt due to Excessive Horizontal Movement

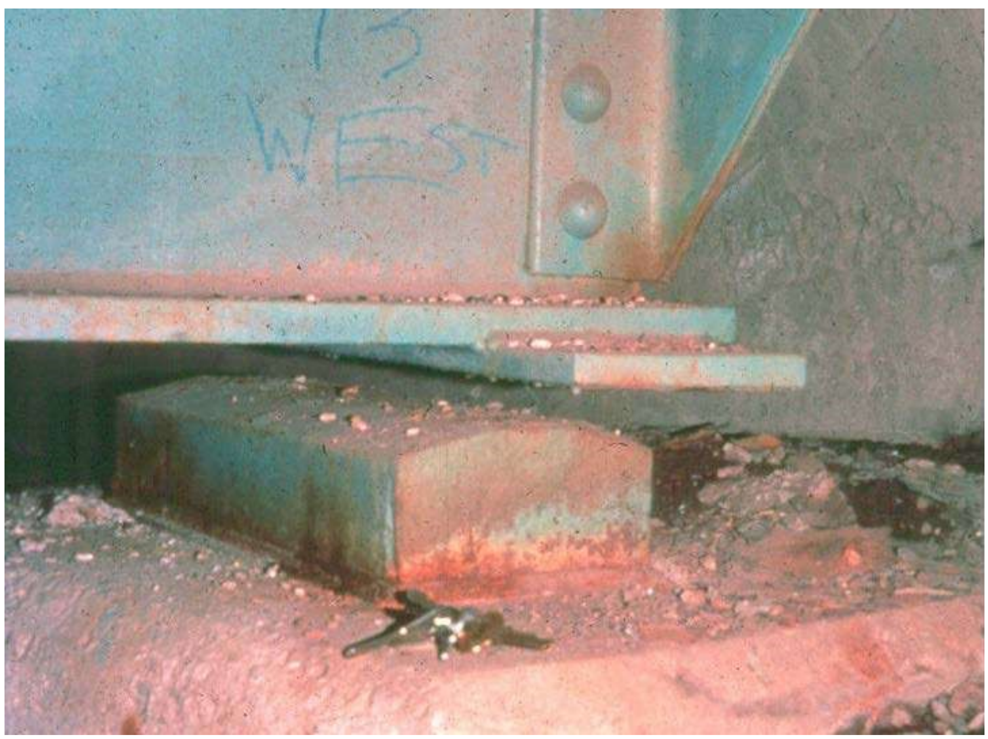

Figure 11.1.31 Uplift at Bridge Bearing

Sliding Plate Bearings

When a bridge is constructed, the upper and lower plates of the sliding plate bearing are placed such that they are centered with respect to each other at a certain temperature, usually 68 degrees Fahrenheit. Any movement of the bearing can be measured based on this initial alignment.

For plates of equal size, the amount of expansion or longitudinal movement that has occurred is the distance from the front or back of the top plate to the front or back of the bottom plate or, alternatively, the distance between the centers of the top and bottom plates (see Figure 11.1.32). For plates of unequal size, the amount of expansion is one half of the difference between the front and back distances between the top and bottom plates. Alternatively, and perhaps easier to measure, the expansion is the distance between the centers of the top and bottom plates. These dimensions need to be measured to the nearest one-eighth inch, in additional to the bridge element temperature at the time of inspection.

Bearings employing bronze sliding plates with steel masonry plates on bridges exposed to a salt air environment need to be examined for signs of electrolytic corrosion between the bronze and steel plates. Galvanic corrosion can also occur between aluminum and steel plates.

See Figure 11.1.33 for a checklist of sliding plate bearing inspection items.

Figure 11.1.32 Longitudinal Misalignment in Bronze Sliding Plate Bearing

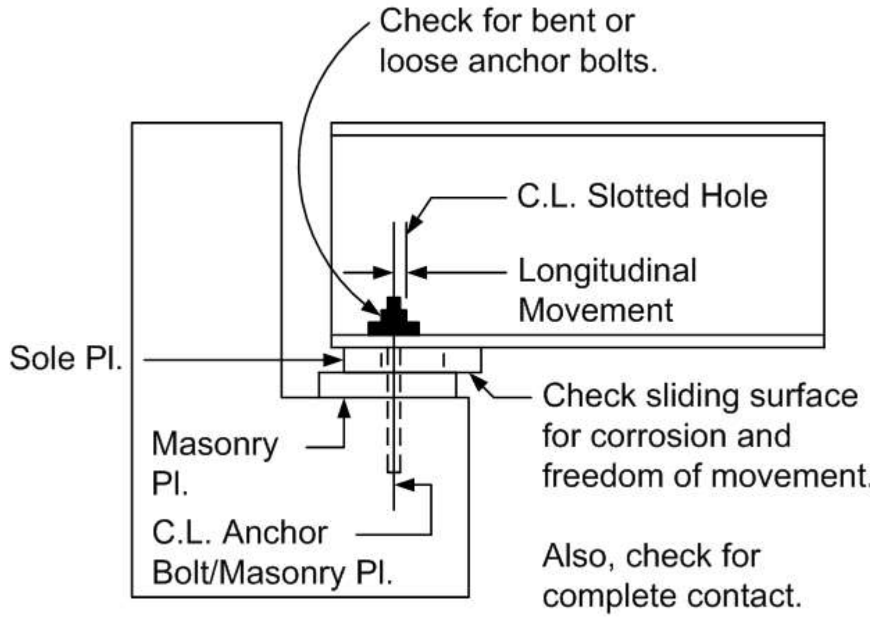

Figure 11.1.33 Sliding Plate Bearing Inspection Checklist Items

Roller Bearings

Roller bearings are similar to sliding plate bearings in that the roller unit needs to be centered on the masonry plate at its design erection temperature. Therefore, the expansion (or contraction) is one half of the difference between the front of plate-to-roller distance and the back of plate-to-roller distance. Alternatively, and perhaps easier to measure, the expansion (or contraction) is also the distance between the center of the roller (where it contacts the masonry plate) and the center of the masonry plate. Again, the temperature at the time of inspection needs to be recorded.

Rollers and masonry plates need to be clean and free of corrosion in order to remain operable. They need to be inspected for signs of wear.

The position of the roller also needs to be examined to see if the pintles are exposed or missing. Such conditions may indicate excessive superstructure expansion or contraction movement or undesirable substructure movement. See Figure 11.1.34 for an example of a damaged roller nest bearing.

Figure 11.1.34 Damaged Roller Nest Bearing

Rocker Bearings

See Figure 11.1.35 for a checklist of rocker bearing inspection items.

Some rocker bearings have markings on the rocker and masonry plates. With no expansion or contraction, these marks need to line up perfectly vertically. The amount of longitudinal movement can be determined by measuring the distance along the masonry plate between the two marks.

If the bearing has no markings, the expansion can be determined by measuring the distance between the current point of contact between the rocker and the masonry plate and the original point of contact, which is assumed to be the midpoint along the rocker's curved surface (see Figure 11.1.36).

Measurements need to be to the nearest one eighth inch, and the inspection temperature needs to be recorded.

Rockers need to be inspected for proper tilt. In warmer temperatures (above 68ºF), the rockers need to be tilted towards the backwall in the expanded direction; in colder temperatures, the rockers need to be tilted backward in the contracted position away from the backwall (see Figure 11.1.35). Also check for the condition of the pintles if they are visible.

Rocker bearings and pins (if present) need to be examined for corrosion, wear and freedom of movement (see Figure 11.1.37).

Check the condition of the anchor bolts and nuts for corrosion and freedom of movement on expansion bearings.

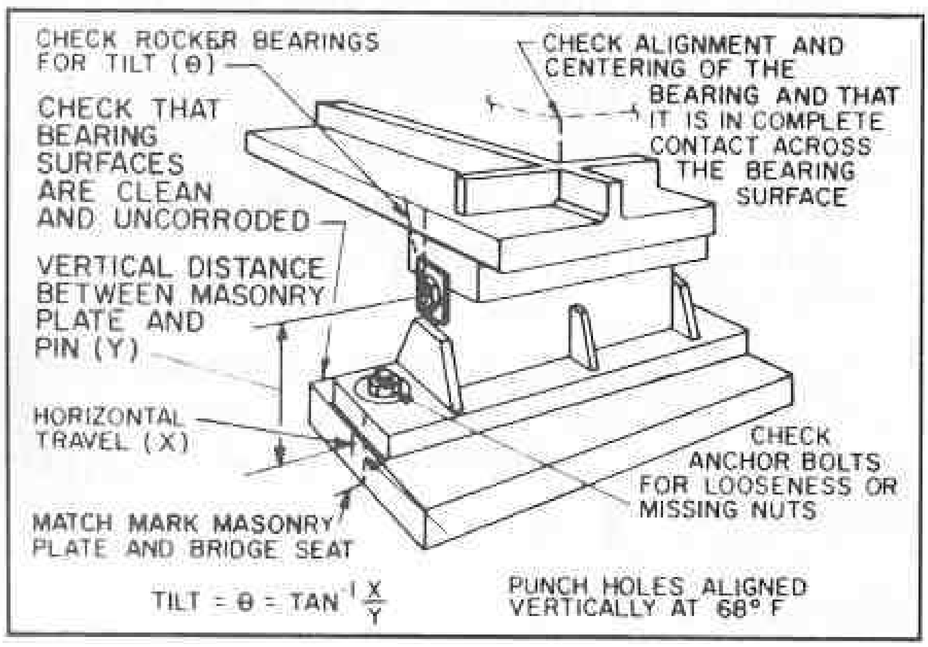

Figure 11.1.35 Rocker Bearing Inspection Checklist Items

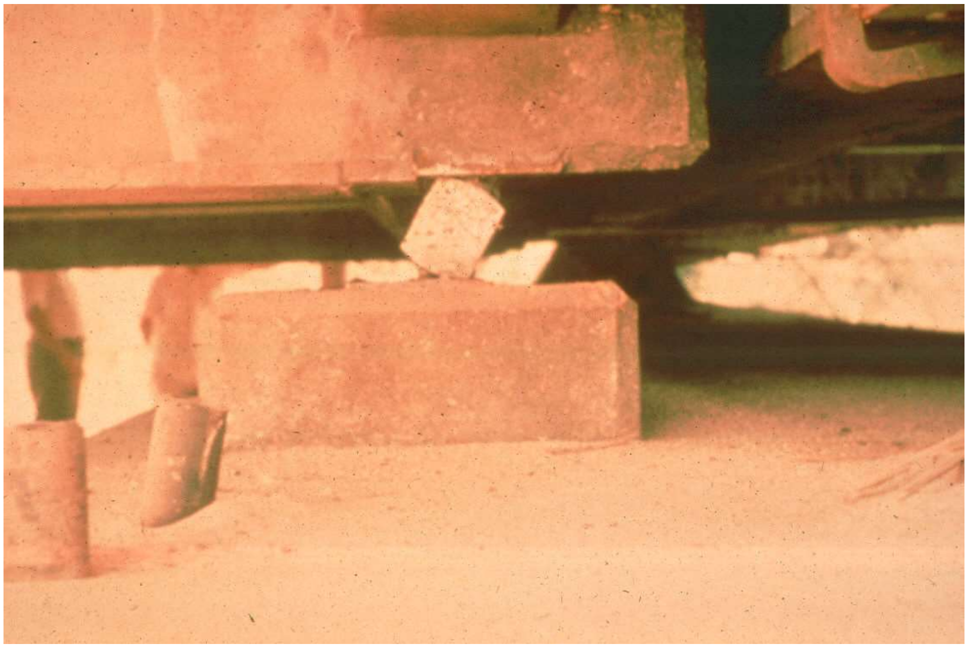

Figure 11.1.36 Excessive Tilt in a Segmental Rocker

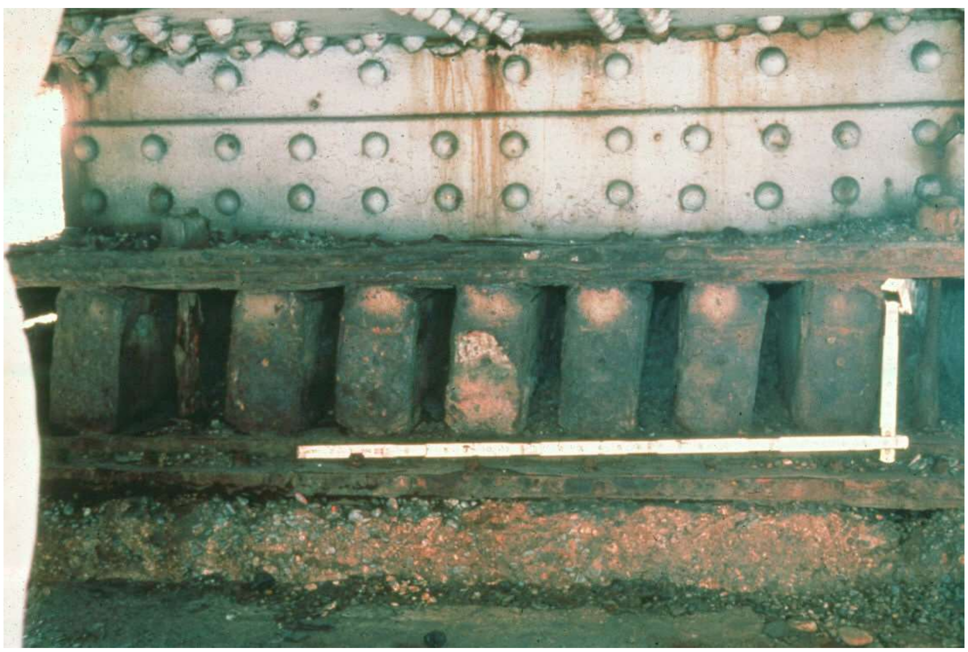

Figure 11.1.37 Frozen Rocker Nest

Pot Bearings

Pot bearing longitudinal movement can be measured in the same way as for a sliding plate bearing. The movement is one half of the difference between the front and back distances of the top and bottom plates. If the pot bearing allows movement in two directions, the inspector needs to investigate transverse movement as well. The inspection temperature at which measurements are taken also needs to be recorded.

Although not normally required, pot bearing rotation also needs to be measured if it appears to be excessive. The top and bottom plates of a pot bearing are usually designed to be parallel if no rotation has taken place. Rotation can therefore be determined by measuring the length of the bottom plate and the distance between the two plates at the front and back of the bearing. The angle of rotation, measured from the horizontal, can be calculated using the following equation and Figure 11.1.38:

$$\text{Rotation (Degrees)} = \tan^{-1}\left[(\text{Height}_1 - \text{Height}_2)/\text{Plate Length}\right]$$

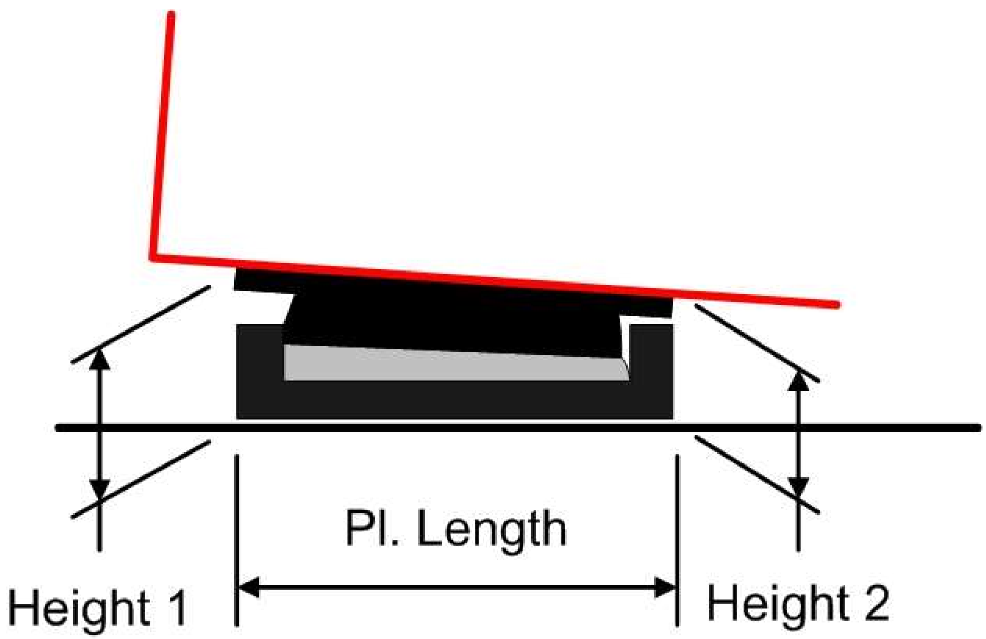

Figure 11.1.38 Frozen Rocker Nest

Since the pot bearing allows multidirectional rotation, the inspector needs to check rotation along both sides of the bearing.

Examine pot bearings for proper seating of the various elements with respect to one another. That is, check to see that the neoprene pad is properly seated within the pot and that the top plate is located properly over the elements below. Determine if the neoprene element is being extruded from the pot. Inspect guide bars for wear, binding, cracking and deterioration.

Investigate welds for cracks, and examine for any separation between the PTFE and the steel surface to which it is bonded. Although they are usually hidden from view, check any exposed portions of the neoprene elements for splitting or tearing. Look for any buildup of dirt and debris in and around the bearing that could affect the smooth operation of the bearing.

Pin and Link Bearings (Uncommon Bearing)

Inspection of pin and link bearings are essentially the same as that described for pins and hangers in Topic 10.9. The amount of corrosion and ability of the connection to move freely is of critical concern, especially for suspended span bridges.

The amount of corrosion on the pin and the interior portion of the link adjacent to it are impossible to detect visually. Ultrasonic testing or disassembly of the connection is required to determine the actual extent of deterioration. For a discussion of ultrasonic testing, refer to Topic 15.3. Since disassembly is impractical during normal periodic bridge inspections, the inspector needs to closely examine exposed portions of the pin and link for signs of corrosion, wear, stress, cracks, bending, and misalignment. If warranted, the inspector needs to recommend further action (i.e., special testing or disassembly of the pin and link).

Also examine the hanger/link for proper amount of tilt using a plumb line or level, record the opening between the ends of the girders, and record the inspection temperature.

Restraining Bearings (Uncommon Bearing)

Inspection of restraining bearings is very similar to that for pin and link bearings in that the condition of the main tension elements (i.e., hanger plates, eyebars, and anchor rods or bolts) and pins is the main concern. Where these elements encompass a normal bridge bearing, the inspection of the bearing assembly itself follows the methods normally used for that particular type of bearing.

The elements that make up the restraining portion of the bearing need to be investigated for deterioration, misalignment, or other defects that could affect the normal operation of the bearing. Anchor bolts may need nondestructive testing to determine their condition.

Inspection of Elastomeric Bearings

Inspection of elastomeric bearings is somewhat simpler than the steel bearings since there are usually fewer elements to inspect. However, certain defects in elastomeric bearings are rather difficult to detect. Elements that are common to both steel bearings and elastomeric bearings are sole plates, masonry plates, and anchor bolts. Only the elastomeric elements or elements specific to elastomeric bearings are discussed here. See Figure 11.1.39 for a checklist of elastomeric bearing inspection items.

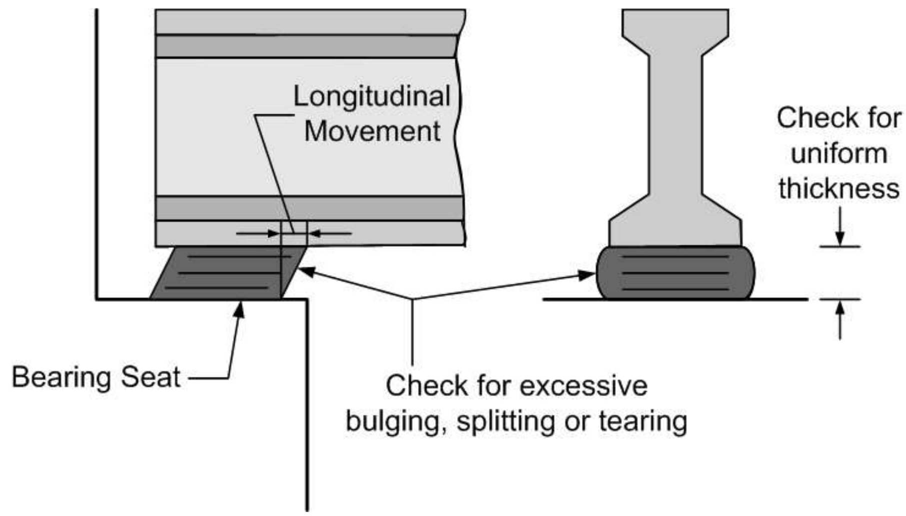

Figure 11.1.39 Elastomeric Bearing Inspection Checklist Items

Neoprene Bearings

Neoprene bearing pads need to be inspected for excessive bulging (approximately greater than 15 percent of thickness) (see Figure 11.1.40). This indicates that the bearing might be too tall for the application and therefore improperly designed. Slight bulging in the sides of the pad can be expected. Whether or not it is excessive may be difficult to determine, but if it appears excessive for the height or thickness of the pad, then it needs to be noted. As expansion and contraction of the structure takes place, the bulge tends to roll on the beam or bridge seat.

The bearing pad needs to be inspected for any splitting or tearing. Close attention needs to be paid to laminated neoprene bearings. Improper manufacturing can sometimes cause a failure in the area where the neoprene and interior steel shims are bonded together.

The pad also needs to be inspected for variable thickness other than that attributable to normal rotation of the bearing.

A plain (unlaminated) pad needs to be examined for any apparent growth in the length of the pad at the masonry plate. This growth indicates excessive strain in the pad. This is not a normal condition and usually indicates a problem with the design or manufacturing of the bearing. If this condition persists, the pad eventually experiences a shearing failure. Pad growth is not usually a problem with laminated bearings.

Close attention needs to be given to the area where the pad is bonded to the sole and masonry plates. This is where a neoprene bearing frequently fails. Therefore, some agencies prohibit bonding of the bearing. Sometimes the pad tends to "walk" out from under the beam or girder. Some agencies prohibit painting of the contact surface between the neoprene and the sole plate for this reason.

The longitudinal movement of a neoprene bearing pad is measured in nearly the same manner as for a sliding plate bearing. The longitudinal movement is the horizontal offset (in the longitudinal direction) between the top edge of the pad and the bottom edge of the pad. Record the temperature at the time of inspection.

The rotation on a neoprene bearing is measured the same way as for a pot bearing. The top and bottom of the pad are normally parallel if no rotation has taken place. The inspector needs to measure the length of the pad and the height of the pad at the front and rear of the bearing. The equation presented in the pot bearing section can then be used to calculate the rotation. If a beveled pad is used to accommodate a bridge on grade, then the original dimensions of the pad needs to be known in order to determine the bearing rotation.

Figure 11.1.40 Neoprene Bearing Pad Excessive Bulging

Isolation Bearings (Uncommon Bearing)

The inspection items for isolation bearings (lead core and high-damping rubber) are essentially the same as those for plain or laminated neoprene bearings. The only elements unique to isolation bearings (lead core) are the lead core and steel dowels, both of which are hidden from view and cannot be inspected (see Figure 11.1.41). The lead core may yield during an earthquake. After a seismic event, the bearing shape and horizontal alignment in both the longitudinal and transverse direction needs to be closely inspected. It may be necessary to replace lead core bearings after an earthquake.

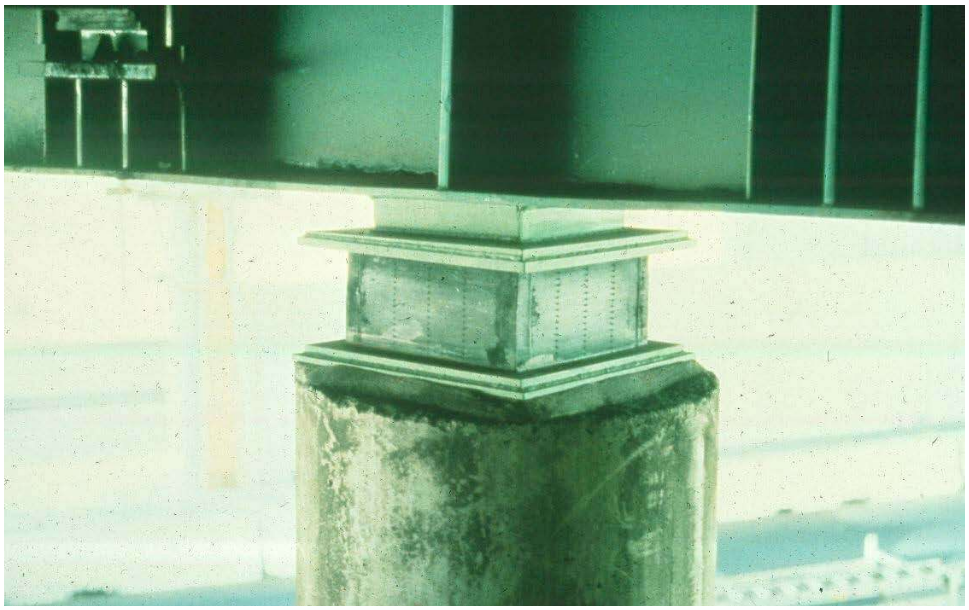

Figure 11.1.41 Lead Core Isolation Bearing

11.1.5 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of bearings. 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 condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, bearings can impact the superstructure component condition rating shown on the Federal Structure Inventory and Appraisal (SI&A) in extreme situations. There is no item for bearings under superstructure in the SI&A.

The bearing type and the condition of the bearing are noted on the inspection form, but no rating is given. Some bridge owners do ask inspectors to provide a condition rating for bearings.

Element Level Condition State Assessment

In an element level condition state assessment of a bridge bearing, the National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
310	Elastomeric Bearing
311	Moveable Bearing (roller, sliding, etc.)
312	Enclosed / Concealed Bearing
313	Fixed Bearing
314	Pot Bearing
315	Disk Bearing

BME No.	Description
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating

The unit quantity for the bearing elements is each. Each bearing element is placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coating is square feet, and the total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating for bearings. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

Note that "uncommon bearings" are classified according to one of the six AASHTO bearing designations.

The following Defect Flags are applicable in the evaluation of bearings:

Smart Flag No.	Description
356	Steel Cracking/Fatigue
357	Pack Rust
363	Steel Section Loss

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

Serious Bearing Conditions

The superstructure condition rating is affected when serious bearing conditions exist that may cause local failures for the supported primary load-carrying members.

If such a serious condition exists with the bearings, then the bearings have an impact on the superstructure condition rating (see Figures 11.1.42 and 11.1.43). Otherwise, the bearings have no effect in the superstructure rating, though the bearing condition and deficiencies are still noted by the inspector.

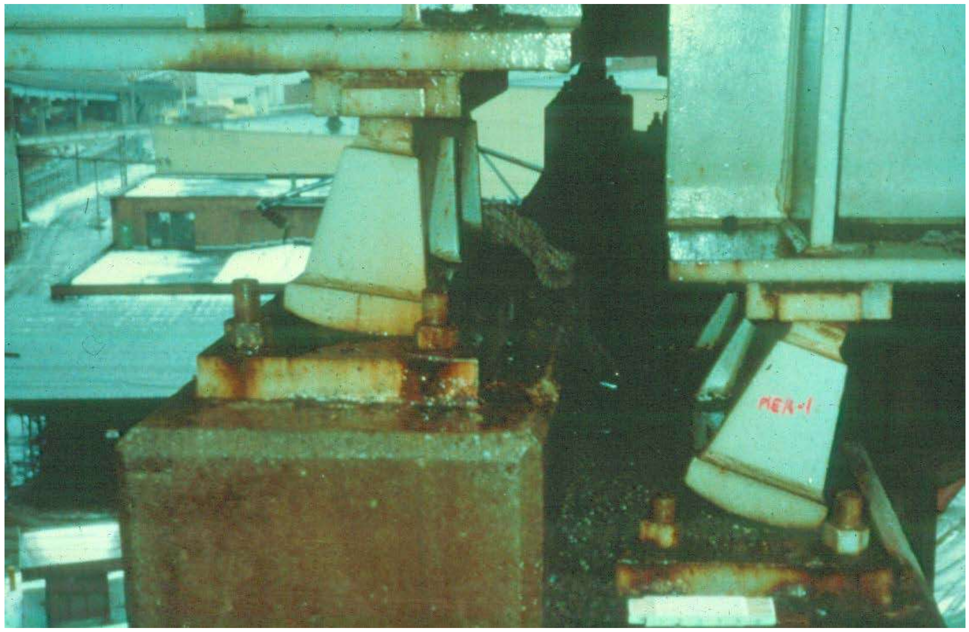

Figure 11.1.42 Serious Bearing Condition

Figure 11.1.43 Broken Pintle on a Bearing

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Chapter 12

Inspection and Evaluation of Substructures

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Chapter 12
Inspection and Evaluation of Substructures

Topic 12.1 Abutments and Wingwalls

12.1.1 Introduction

The substructure is the component of a bridge that includes all elements supporting the superstructure. Its purpose is to transfer the loads from the superstructure to the foundation soil or rock.

An abutment is a substructure unit located at the end of a bridge. Its function is to provide end support for the bridge superstructure and to retain the approach roadway embankment. Wingwalls are also located at the ends of a bridge. Their function is only to retain the approach roadway embankment and not to provide end support for the bridge.

Wingwalls are considered part of the substructure component only if they are integral with the abutment. When there is an expansion joint or construction joint between the abutment and the wingwall, that wingwall is defined as an independent wingwall, i.e., a retaining wall, and not considered in the condition evaluation of the abutment-substructure component.

12.1.2 Design Characteristics of Abutments

Common Abutment Types

Abutments are classified according to their locations with respect to the approach roadway embankment. The most common abutment types are presented in Figure 12.1.1 and include:

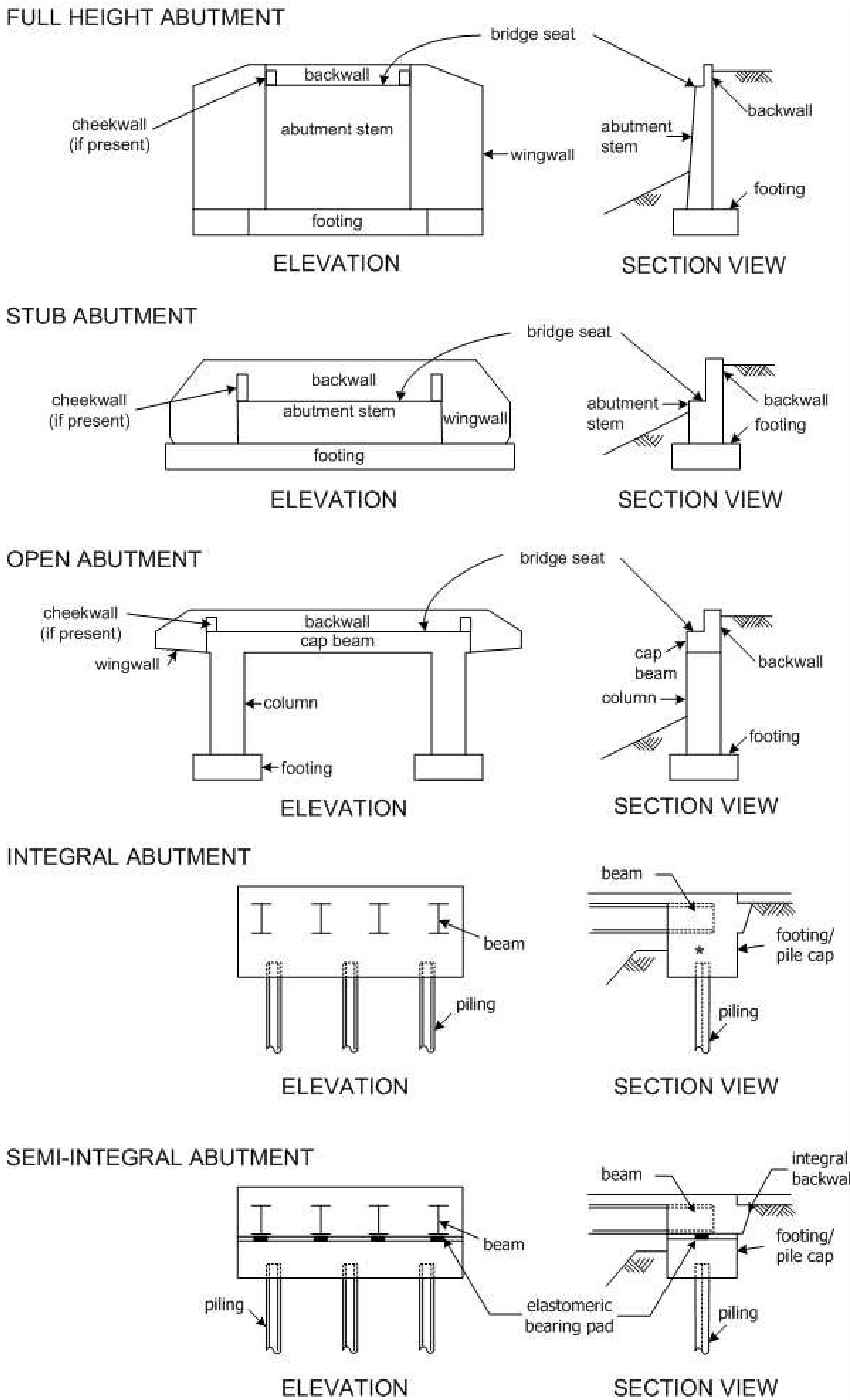
- Full height or closed type
- Stub, semi-stub, or shelf type
- Open or spill-through type
- Integral
- Semi-integral

Foundations consist of either spread footings or deep foundations. See page 12.1.16 for a detailed description of abutment foundation types.

Less common abutments used to support highway bridges are shown in Figures 12.1.2 and 12.1.3, and include:

- Mechanically Stabilized Earth (MSE)
- Geosynthetic Reinforced Soil (GRS)

Detailed descriptions of abutment elements are provided on page 12.1.15.



* Some agencies weld beam and piles together prior to concrete placement

Figure 12.1.1 Schematic of Common Abutment Types

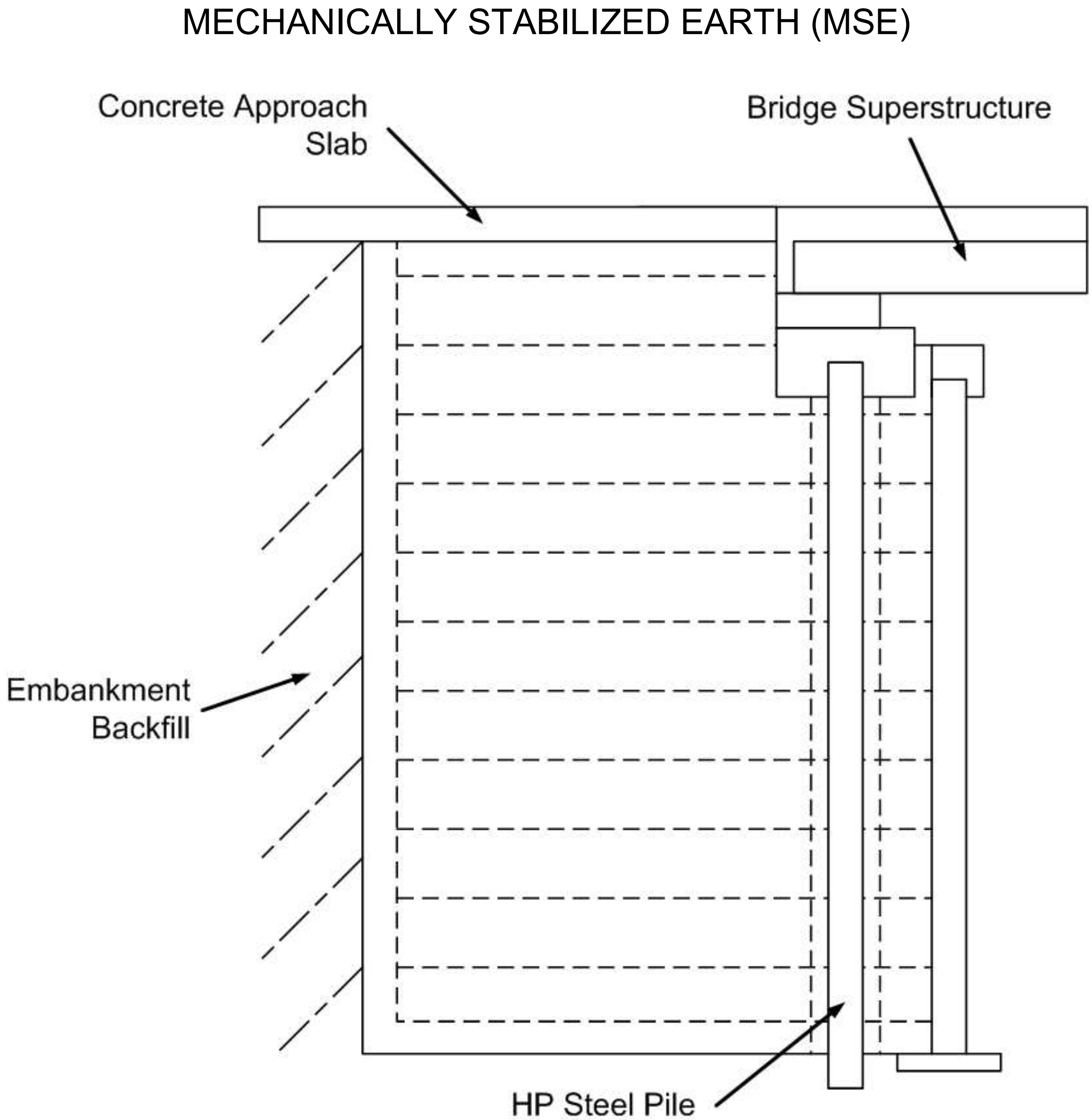


Figure 12.1.2 Section View of Less Common Abutment Types (Mechanically Stabilized Earth)

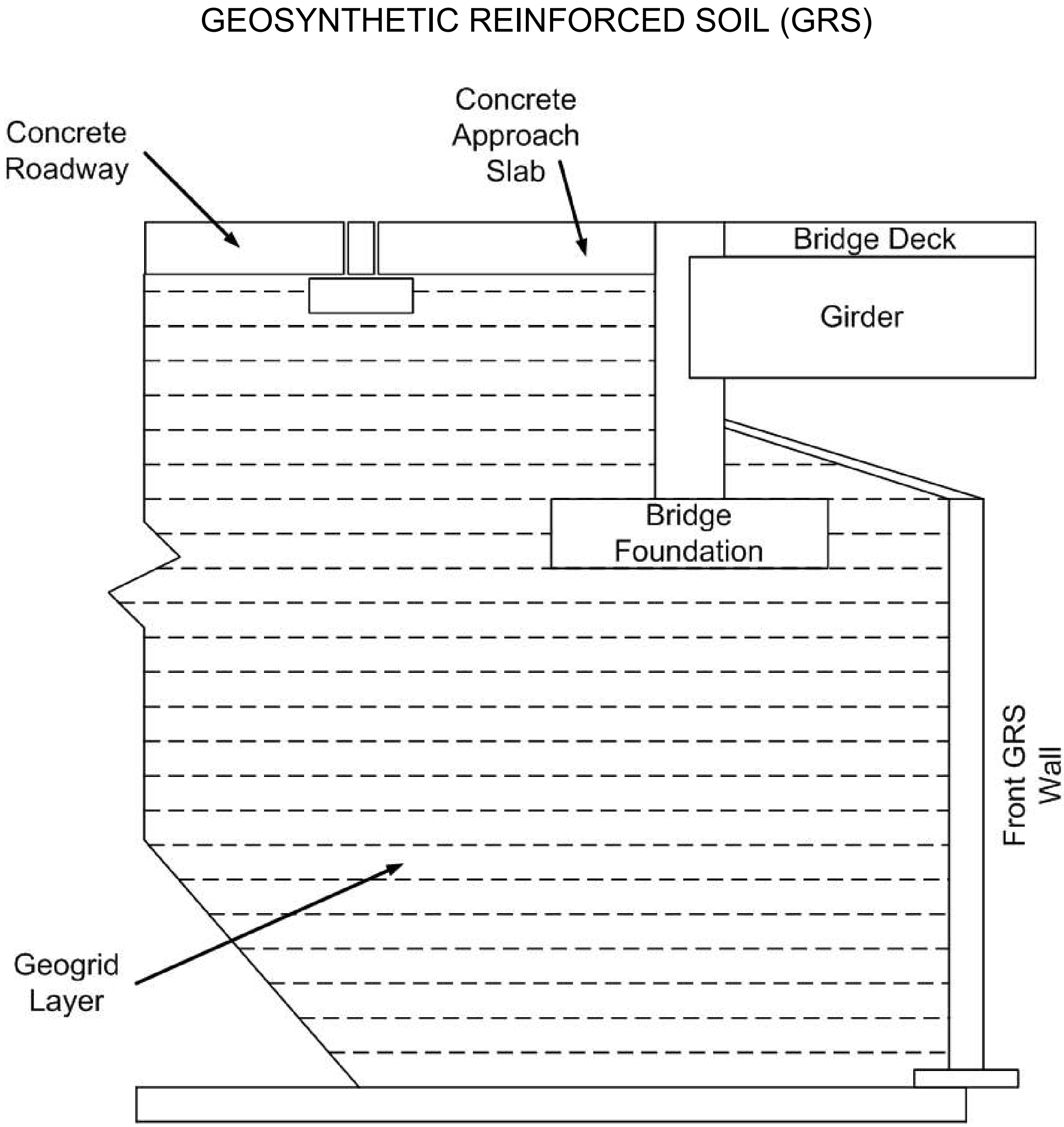


Figure 12.1.3 Section View of Less Common Abutment Types (Geosynthetic Reinforced Soil)

Full Height Abutments and Stub Abutments

Full height abutments are used when shorter spans are desired or if there are Right-of-Way or terrain issues (see Figure 12.1.4). This reduces the initial superstructure costs. Stub abutments may be used when it is desirable to keep the abutments away from the underlying roadway or waterway (see Figure 12.1.5). Longer spans are required when stub abutments are used. Using stub abutments reduces the cost of the substructure but increases the cost of the superstructure.

Figure 12.1.4 Full Height Abutment

Figure 12.1.5 Stub Abutment

Open Abutments

Open, or spill-through, abutments are similar in construction to multi-column piers. Instead of being retained by a solid wall, the approach roadway embankment extends on a slope below the bridge seat and between ("through") the supporting columns. Only the topmost few feet of the embankment are actually retained by the abutment cap (see Figure 12.1.6).

The advantages of the open abutment are lower construction cost since most of the horizontal load is eliminated, so the massive construction and heavy reinforcement usually associated with the abutment stem is not needed. This substructure type has the ability to convert the abutment to a pier if additional spans are added in the future.

Open abutment disadvantages include a tendency for the fill to settle around the columns since good compaction is difficult to achieve in the confined spaces. Excessive erosion or scour may also occur in the fore slope. Rock fill is sometimes used to counter these problems. This abutment type is not suitable adjacent to streams due to susceptibility to scour.

Figure 12.1.6 Open Abutment

Integral Abutments and Semi-Integral Abutments

Most bridges have superstructures that are independent of the substructure to accommodate bridge length changes due to thermal effects. Expansion devices such as deck joints and expansion bearings allow for thermal movements but deteriorate quickly and create a wide range of maintenance needs for the bridge. In extreme cases, lack of movement due to failed expansion devices can lead to undesirable stresses in the bridge. Integral abutments supported by a single row of piles are becoming more popular and provide a solution to these problems.

In this design, the superstructure and substructure are integral and act as one unit without an expansion joint (see Figure 12.1.7). Relative movement of the abutment with respect to the backfill allows the structure to adjust to thermal expansions and contractions. Pavement joints at the ends of approach slabs are provided to accommodate the relative movement between the bridge and the approach roadway pavement.

The advantage of the integral abutment is that it lacks bearing devices and joints to repair, or replace, or maintain (see Figure 12.1.8). There are two disadvantages of integral abutments: settlement of the roadway approach due to undercompaction of backfill; and cracking of the abutment concrete due to movement restriction caused by overcompaction of backfill or superstructure rotations due to heavy skews.

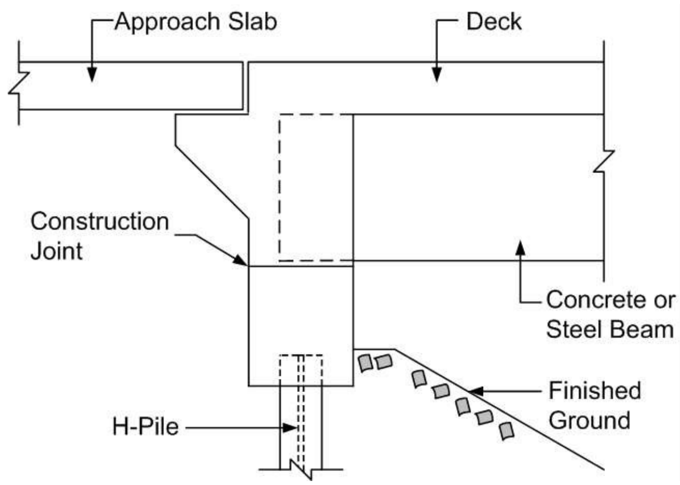


Figure 12.1.7 Integral Abutment

Figure 12.1.8 Integral Abutment

Semi-integral abutments are similar to integral abutments, however, the superstructure and the top of the abutment act as one unit, but the bottom portion act independently of the superstructure. This is achieved by a joint between the top and bottom portions of the abutment which will allow for un-restrained rotation and thermal movement.

Less Common Abutment Types

Some Agencies utilize additional, less common abutment types.

Mechanically Stabilized Earth Abutments

Mechanically Stabilized Earth (MSE) abutment typically consists of precast concrete panels, metallic soil reinforcing strips (flat strips or welded bar grids), and backfill to support the superstructure and support the roadway approach roadway embankment (see Figure 12.1.9). Two MSE abutment design concepts have been used. The first utilizes an MSE wall supporting a slab, or coping, on which the bridge bearings rest. Vertical loads are transmitted through the reinforced fill. The second concept utilizes piles or columns to support a stub abutment at the top of the reinforced fill. The piles provide vertical support for the bridge. The MSE provides lateral support for the approach roadway embankment. Problems have occurred when the MSE wall supports the bearings, since the MSE walls bulge out when they support vertical superstructure loads, which are transmitted through the bearings. Current construction practices call for stub abutments behind the MSE wall.

Precast vertical concrete panels are erected first, followed by the placement and compaction of a layer of backfill. The layers of backfill are sometimes referred to as “lifts.” Horizontal soil reinforcement is then placed and bolted to the panels and covered with more backfill (see Figure 12.1.10). This process, which allows the wall to remain stable during construction, is repeated until the designed height is attained.

Advantages of this substructure are its internal stability and its ability to counteract shear forces, especially during earthquakes. It is generally lower in cost and has favorable esthetics when compared to a reinforced concrete full height abutment. Disadvantages include difficulty in repairing failed soil reinforcement and limited site applications. Another disadvantage is the possible settlement of an MSE wall that directly supports the superstructure (i.e. no stub abutment with piles).

Figure 12.1.9 Mechanically Stabilized Earth Abutment (Note Precast Concrete Panels)

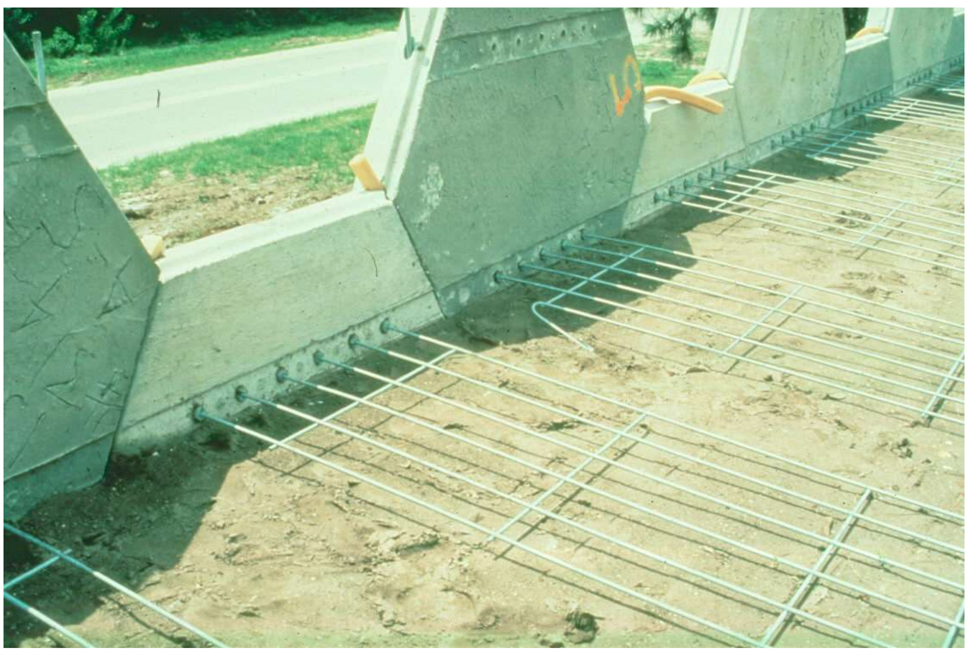


Figure 12.1.10 Mechanically Stabilized Earth Wall Under Construction

Geosynthetic Reinforced Soil Abutments

Another less common, fairly new type of abutment is the geosynthetic reinforced soil (GRS) abutment. GRS abutments are basically constructed on a level surface starting with a base structure of common, but high quality, cinder blocks. Fill is then placed and compacted with a sheet of geosynthetic reinforcement, which can be a series of polymer sheets or grids. These materials are layered until the designed height is attained. GRS abutments, which are internally supported, use friction to hold the blocks together and obtain their strength through proper spacing of the layers of reinforcement. Advantages of GRS abutments are their simplicity to construct and their aesthetic appearance. GRS technology works well with simple overpasses; however, they are not ideal where severe flooding or scour could occur (see Figures 12.1.11 and 12.1.12).

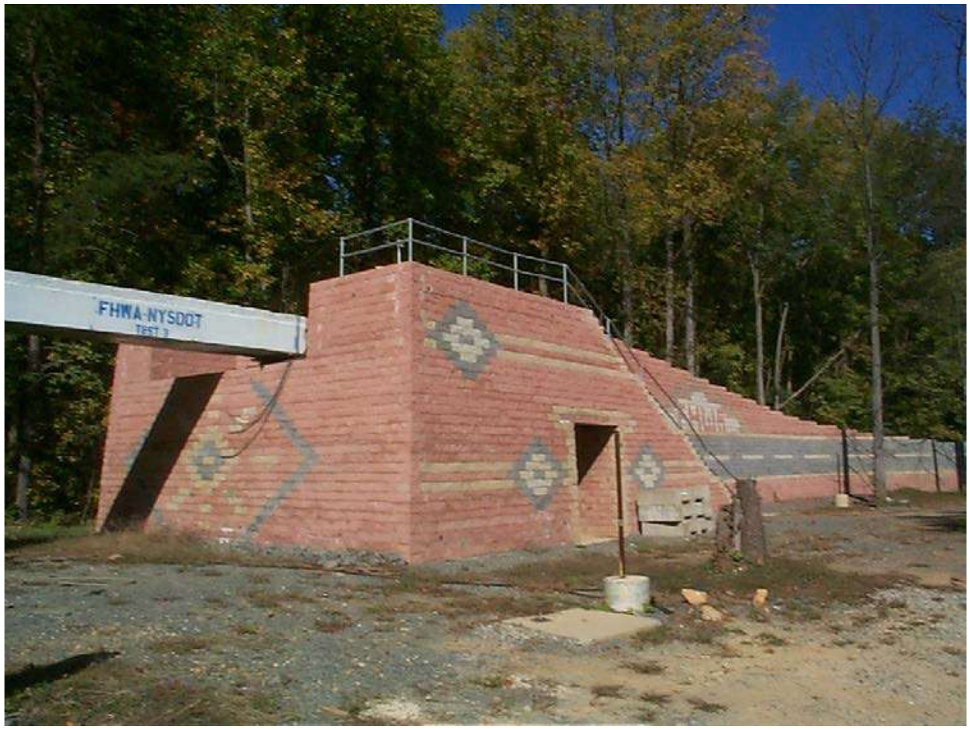


Figure 12.1.11 GRS Bridge Abutment at the FHWA Turner-Fairbank Highway Research Center

The stabilized earth concepts, using metallic or geosynthetic reinforcement, are more commonly used as retaining walls or wing walls than as abutments. See Report No. FHWA-SA-96-071 (Demo 82 Manual) for a detailed description of these systems.

Figure 12.1.12 View of the Founders/Meadows Bridge Supported by GRS Abutments

Primary Materials

The primary materials used in abutment construction are unreinforced concrete, reinforced concrete, stone masonry, steel (although not very common), timber, reinforcing strips (either metallic or geosynthetic), or a combination of these materials (see Figures 12.1.13 thru 12.1.17).

Figure 12.1.13 Plain Unreinforced Concrete Gravity Abutment

Figure 12.1.14 Reinforced Concrete Cantilever Abutment

Figure 12.1.15 Stone Masonry Gravity Abutment

Figure 12.1.16 Combination: Timber Pile Bent Abutment with Reinforced Concrete Cap

Figure 12.1.17 Steel Abutment

Primary and Secondary Reinforcement

The pattern of primary steel reinforcement used in concrete abutments depends on the abutment type (see Figure 12.1.18). In a cantilever abutment, primary tension reinforcement include: vertical bars in the rear face of the stem and backwall, horizontal bars in the bottom of the footing (toe steel), and horizontal bars in the top of the footing (heel steel). In a concrete open or spill-through abutment, the primary reinforcement consists of both tension and shear steel reinforcement. Tension steel reinforcement generally consists of vertical bars in the rear face of the backwall and cap beam, horizontal bars in the bottom face of the cap beam, vertical bars in the columns and horizontal bars in the bottom of the footing.

Stirrups are used to resist shear in the cap beam. The column spirals or ties are generally considered to be secondary reinforcement to reduce the un-braced length of the vertical bars in the column (see Figure 12.1.19). The spirals or ties may be considered primary reinforcement in seismic zones. Bars used for temperature and shrinkage reinforcement are considered secondary reinforcement.

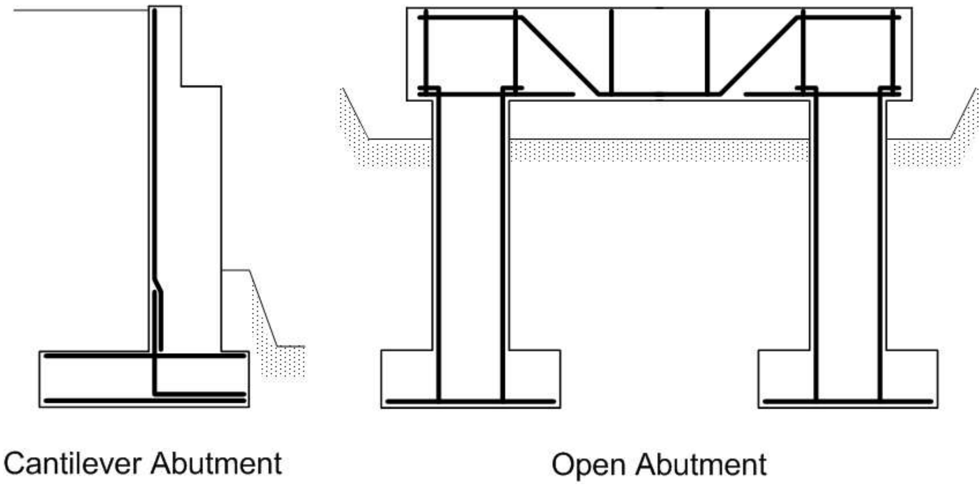


Figure 12.1.18 Primary Reinforcement in Concrete Abutments

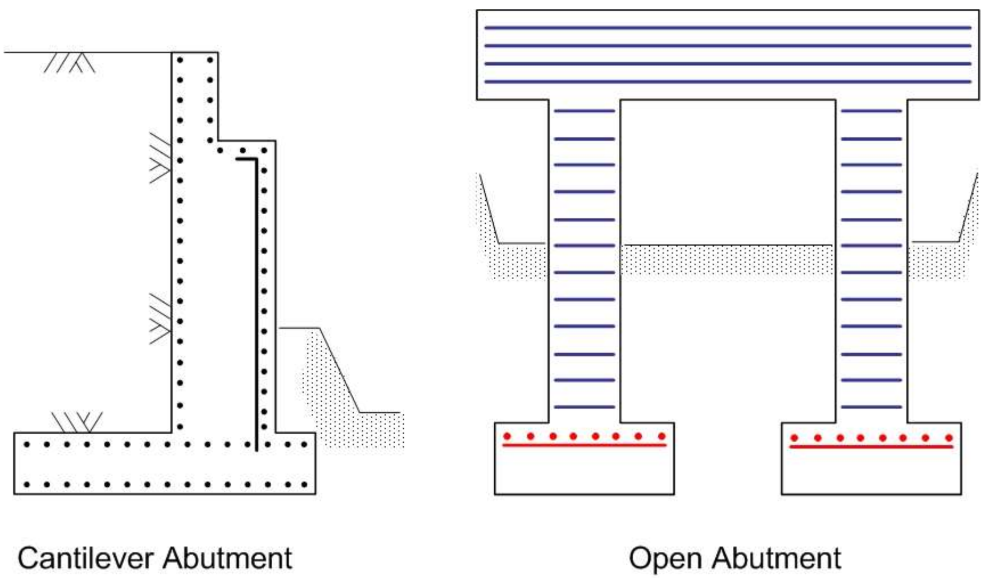


Figure 12.1.19 Secondary Reinforcement in Concrete Abutments

Abutment Members

Common abutment members include:

- Bridge seat
- Backwall
- Footing and pile cap
- Cheek wall
- Abutment stem (breast wall)
- Tie backs
- Soil reinforcing strips

- Precast panels
- Spread footings
- Deep foundations
- Geotextiles

The basic abutment elements are shown in Figure 12.1.1 through Figure 12.1.3 and described below.

The bridge seat provides a bearing area that supports the bridge superstructure. The backwall retains the approach roadway sub-base and keeps it from sliding onto the bridge seat. It also provides support for the approach slab and for the expansion joint, if one is present. The cheek wall is mostly cosmetic but also protects the end bearings from the elements, (see Figure 12.1.20). A cheek wall is not always present.

The abutment stem or breast wall supports the bridge seat and retains the soil behind the abutment. The foundation, either spread footing or deep foundation (piles, drilled shafts, etc.), transmits the weight of the abutment, the soil backfill loads, and the bridge reactions to the supporting soil or rock (see Figure 12.1.21). It also provides stability against overturning and sliding forces. The portion of the footing in front of the wall is called the toe, and the portion behind the wall, under the approach embankment, is called the heel.

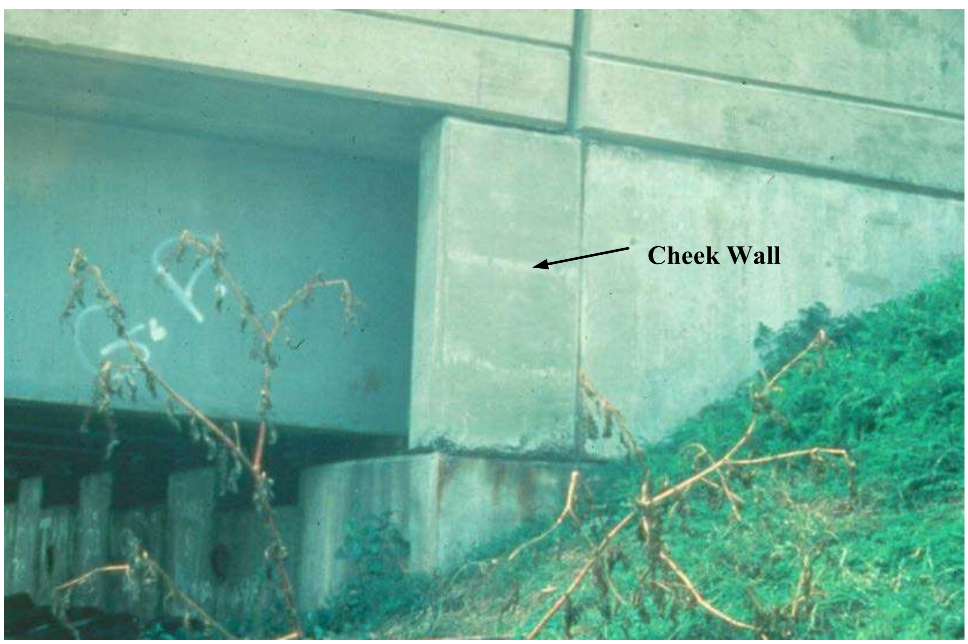


Figure 12.1.20 Cheek Wall

Mechanically stabilized earth (MSE) walls consist of a reinforced soil mass and a concrete facing which is vertical or near vertical. The facing is often precast panels which are used to hold the soil in position at the face of the wall. The reinforced soil mass consists of select granular backfill. The tensile reinforcements and their connections may be proprietary, and may employ either metallic (i.e., strip- or grid-type) or polymeric (i.e., sheet-, strip-, or grid-type) reinforcement. The soil reinforcing strips hold the wall facing panels in position and provide reinforcement for the soil. Geotextiles are used to cover the joint between the panels. Geotextiles are placed behind the precast panels to keep the soil from being eroded through the joints and allow excess water to flow out. Tie backs are steel bars or strands grouted into the soil or rock behind the abutment stem. Tie backs, if present, are used when lateral earth forces cannot be resisted by the footing alone.

Foundation Types

Foundations are critical to the stability of the bridge since the foundation ultimately supports the entire structure. The two basic types of bridge foundations shown in Figure 12.1.21 are:

- Spread footings
- Deep foundations

A spread footing is used when bedrock is close to the ground surface or when the soil is capable of supporting the bridge. A spread footing is typically a rectangular reinforced concrete slab. This type of foundation "spreads out" or distributes the loads from the bridge to the underlying rock or soil. While a spread footing is usually buried, it is generally covered with a minimal amount of soil. In cold regions, the bottom of a spread footing is placed below the recognized maximum frost line depth for that area.

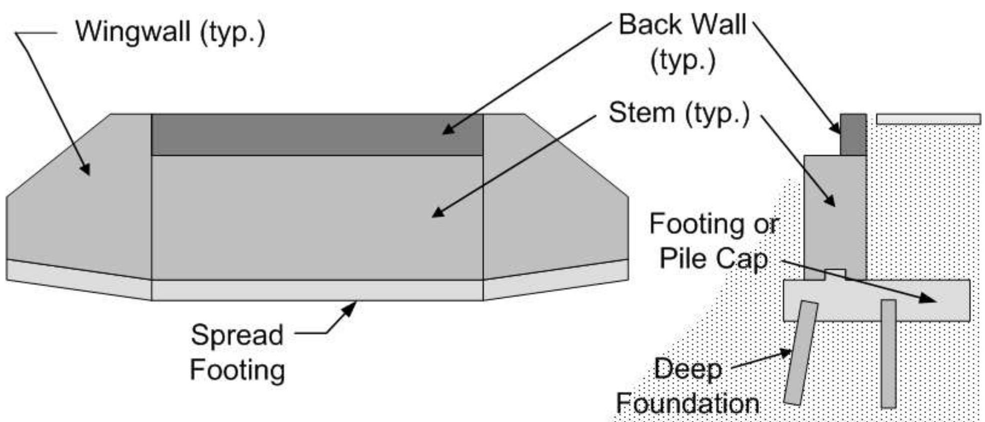


Figure 12.1.21 Spread Footing and Deep Foundations

A deep foundation is used when the soil is not suited for supporting the bridge.

A pile is a long, slender support which is typically driven into the ground but can be placed in predrilled holes. Piles can be partially exposed and are made of steel, concrete (cast-in-place or precast), or timber (see Figure 12.1.22). Various numbers and configurations of piles can be used to support a bridge foundation. This type of foundation transfers load to sound material well below the surface or, in the case of friction piles, to the surrounding soil.

"Caisson", "drilled shaft", or "bored pile" is another type of deep foundation used when the soil is not competent to support a spread footing. Holes are drilled through the soil and filled with reinforced concrete. Temporary or permanent steel casing is utilized during the construction process to support and retain the sides of a borehole. Temporary steel casing is removed after the concrete is placed and is capable of withstanding the surrounding pressures. The minimum caisson diameter used for bridge substructure construction is normally 30 inches. Caissons, drilled shafts or bored piles may be extended through voids such as caverns or mines to reach bedrock under the bridge.

Figure 12.1.22 Stub Abutment on Piles with Piles Exposed

12.1.3 Design Characteristics of Wingwalls

General

Wingwalls are located on the sides of an abutment and enclose the approach fill. Wingwalls are generally considered to be retaining walls since they are designed to maintain a difference in ground surface elevations on the two sides of the wall (see Figure 12.1.23).

A wingwall is similar to an abutment except that it is not required to carry any loads from the superstructure. The absence of the vertical superstructure load usually necessitates a wider footing to resist the overturning moment or horizontal sliding due to lateral earth pressure.

Figure 12.1.23 Typical Wingwall

Geometrical Classifications

There are several geometrical classifications of wingwalls, and their use is dependent on the design requirements of the structure:

- Straight - extensions of the abutment wall (see Figure 12.1.24)
- Flared - form an acute angle with the bridge roadway (see Figure 12.1.25)
- U-wings - parallel to the bridge roadway (see Figure 12.1.26)

Figure 12.1.24 Straight Wingwall

Figure 12.1.25 Flared Wingwall

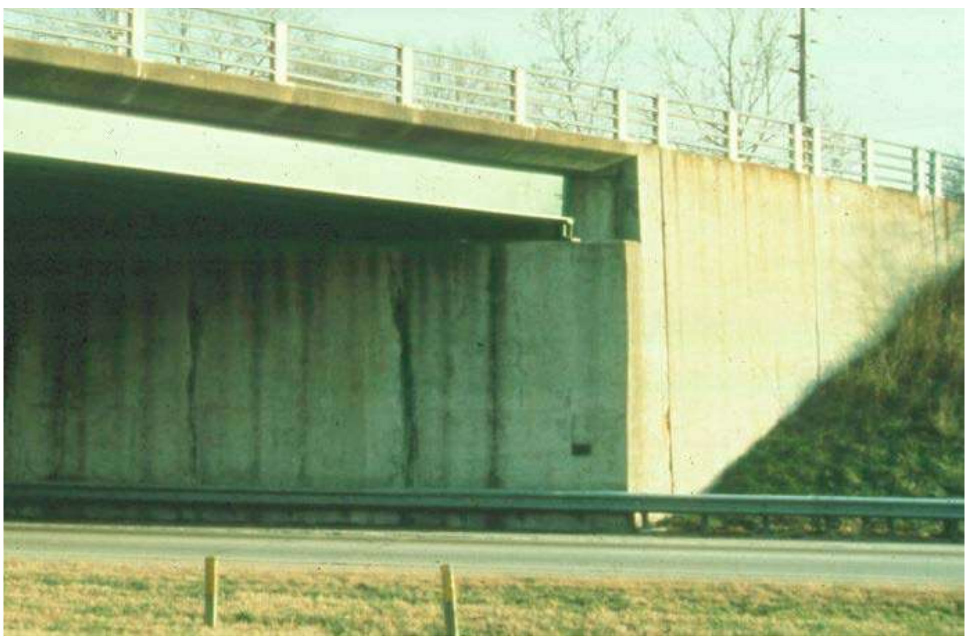


Figure 12.1.26 U-wingwall

Construction Classifications

There are several construction classifications of wingwalls:

- Integral - constructed monolithically with the abutment (see Figure 12.1.27) normally cast-in-place concrete with no expansion or construction joint between the abutment and wingwall
- Independent - constructed separately from the abutment; usually an expansion or construction joint separates the wingwall from the abutment (see Figure 12.1.28)

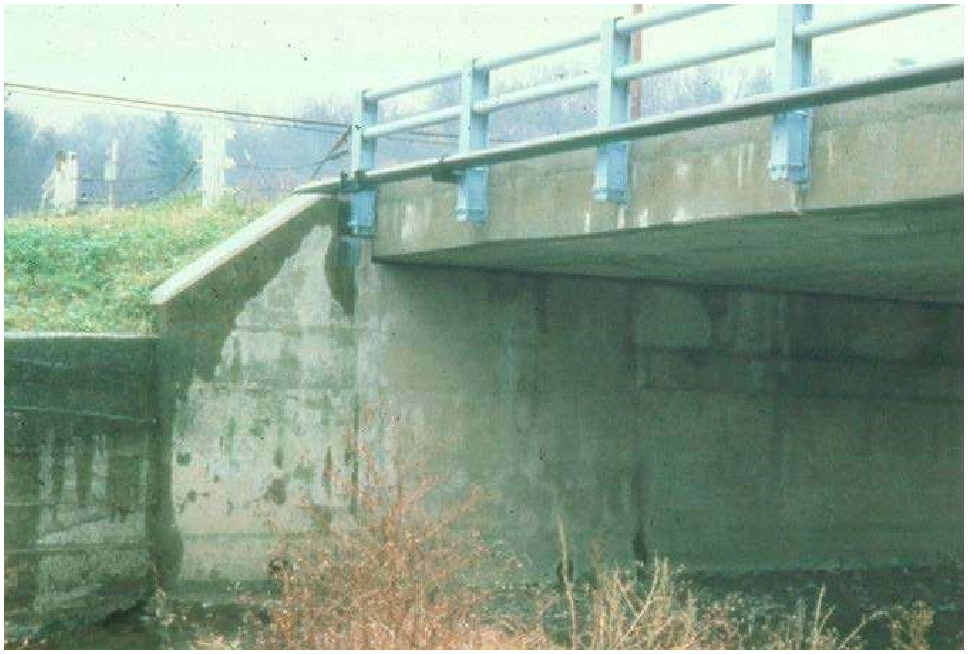


Figure 12.1.27 Integral Wingwall

Figure 12.1.28 Independent MSE Wingwall

Primary Materials

Wingwalls may be constructed of concrete, stone masonry, steel, or timber or a combination of these materials (see Figure 12.1.29).

Figure 12.1.29 Masonry Wingwall

Primary and Secondary Reinforcement

In a concrete cantilever wingwall, the primary reinforcing steel consists of vertical bars in the rear face of the stem, horizontal bars in the bottom of the footing (toe steel), and horizontal bars in the top of the footing (heel steel) (see Figure 12.1.30). Secondary reinforcement is used to resist temperature and shrinkage.

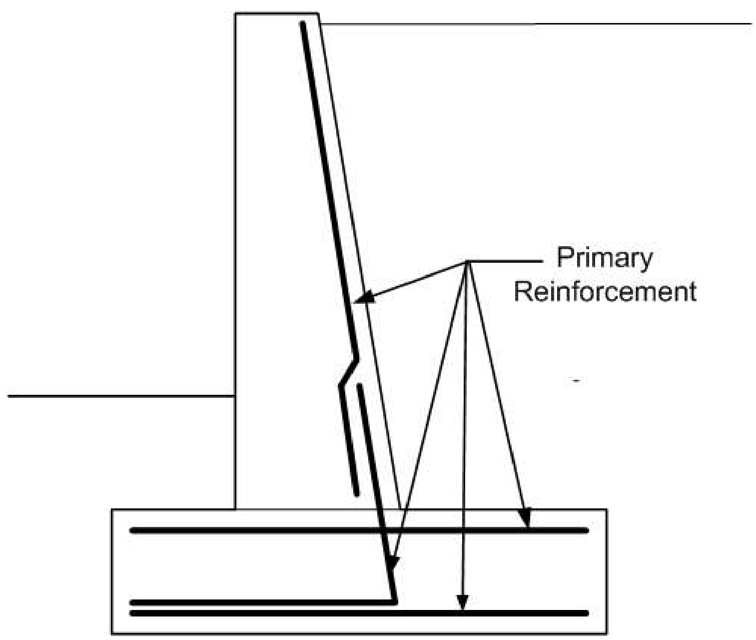


Figure 12.1.30 Primary Reinforcement in Concrete Cantilever Wingwall

12.1.4 Inspection Methods and Locations

The inspection methods and locations for most wingwalls are similar to those for abutments. Many of the problems that occur in abutments are also common in wingwalls.

Methods

The specific visual, physical and advanced inspection methods are dependent upon the type of material used in the abutment and wingwalls. The inspection method used is based on the type of material the abutment or wingwall is made of and the methods are similar to the inspection of superstructures. See Topics 6.1 and 15.1 (Timber), Topics 6.2 and 15.2 (Concrete), 6.3 and 15.3 (Steel), or Topic 6.4 (Stone Masonry) for specific material defects and inspection methods.

Visual

There are two types of visual inspections that may be required of an inspector. The first, called a routine inspection, involves reviewing the previous inspection report and visually examining the members of the bridge. A routine inspection involves a visual assessment to identify obvious defects.

The second type of visual inspection is called an in-depth inspection. An in-depth inspection is an inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine methods. Hands-on inspection may be necessary at some locations. This type of visual inspection requires the inspector to visually assess every defective surface at a distance no further than an arm's length. Surfaces are given close visual attention to quantify and qualify any defects.

As presented in Topic 6.2.6, visually inspect for the following concrete deficiencies:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation) (See Figure 12.1.31)
- Scaling (See Figure 12.1.32)
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence (See Figure 12.1.33)
- Alkali-Silica Reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion

- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation
- Other causes (temperature changes, chemical attack, moisture absorption, differential foundation movement, design and construction deficiencies, unintended objects in concrete, fire damage)

As presented in Topic 6.5.4, visually inspect for the following masonry deficiencies:

- Weathering – hard surfaces degenerate in to small granules, giving stones a smooth, rounded look; mortar disintegrates (See Figure 12.1.34)
- Spalling – small pieces of rock break out
- Splitting – seams or cracks open up in rocks, eventually breaking them into smaller pieces
- Fire – masonry is not flammable but can be damaged by high temperatures

Figure 12.1.31 Cracking in Bearing Seat of Concrete and Stone Abutment

Figure 12.1.32 Spalled Concrete Wingwall

Figure 12.1.33 Cracking and Efflorescence in Abutment Backwall

Figure 12.1.34 Stone Masonry Abutment with Deteriorated Joints

As presented in Topic 6.3.5, visually inspect for the following steel deficiencies (see Figure 12.1.35):

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

Figure 12.1.35 Steel Abutment

As presented in Topic 6.1.5, visually inspect for the following timber deficiencies:

- Inherent defects: checks, splits, shakes, knots
- Fungi
- Insects (see Figure 12.1.36)
- Marine borers
- Chemical attack
- Delaminations
- Loose connection (see Figure 12.1.37)
- Surface depressions
- Fire
- Collision damage
- Wear
- Abrasion (see Figure 12.1.38)
- Overstress (see Figure 12.1.37)
- Protective coating failure

Figure 12.1.36 Decay caused by insects in Timber Abutment

Figure 12.1.37 Local Failure in Timber Pile due to Lateral Movement of Abutment

Figure 12.1.38 Decayed Lagging and Abrasion Caused by Scour at a Timber Pile Bent Abutment

Physical

Once the defects are identified visually, physical methods are used to verify the extent of the defect. Carefully measure and record deficiencies found during physical inspection methods.

Areas of concrete or rebar deterioration identified visually need to be examined physically using an inspection hammer. This hands-on effort verifies the extent of the deficiency and its severity. A delaminated area has a distinctive hollow “clacking” sound when tapped with a hammer. The location, length and width of cracks found during the visual inspection need to be measured and recorded.

For steel members, the main physical inspection methods involve the use of an inspection hammer or wire brush. Excessive hammering, brushing or grinding may close surface cracks and make the cracks difficult to find. Corrosion results in loss of member material. This partial loss of cross section due to corrosion is known as section loss. Section loss may be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or an ultrasonic thickness gauge (D-meter), are used to measure the remaining section of steel. The inspector removes all corrosion products (rust scale) prior to taking measurements.

For timber members, an inspection hammer is used to tap on areas and determine the extent of internal decay. This is done by listening to the sound the hammer makes. If it sounds hollow, internal decay may be present.

Advanced Inspection Methods

If the extent of the deficiency cannot be determined by the visual and physical inspection methods described above, advanced inspection methods are used.

For concrete inspections, non-destructive methods, described in Topic 15.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Electrical methods
- Delamination detection machinery
- Ground-penetrating radar
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
- Impact-echo testing
- Infrared thermography
- Laser ultrasonic testing
- Magnetic field disturbance
- Neutron probe for detection of chlorides
- Nuclear methods
- Pachometer
- Rebound and penetration methods
- Ultrasonic testing
- Smart concrete
- Carbonation

Other advanced methods for concrete members, described in Topic 15.2.3, include:

- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Petrographic examination
- Reinforcing steel strength
- Chloride test
- Matrix analysis
- ASR evaluation

For steel inspections, non-destructive methods, described in Topic 15.3.2, include:

- Acoustic emissions testing
- Corrosion sensors
- Smart coatings
- Dye penetrant
- Magnetic particle
- Radiography testing
- Computed tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Electrochemical fatigue sensor (EFS)
- Magnetic flux leakage (external PT tendons and stay cables)
- Laser vibrometer (for stay cable vibration measurement and cable force determination)

Other advanced methods for steel members, described in Topic 15.3.3, include:

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

For timber inspections, non-destructive methods, described in Topic 15.1.1, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other advanced methods for timber members, described in Topic 15.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Field Ohmmeter

Locations

Stability is a paramount concern; therefore checking for various forms of movement is required during the inspection of abutments and wingwalls.

The locations for inspection can be related to common abutment and wingwall problems.

The most common problems observed during the inspection of abutments and wingwalls are associated with:

- Areas subjected to movement
- High stress areas
- Areas exposed to drainage
- Areas exposed to traffic
- Areas previously repaired
- Scour and undermining
- Problematic details and fracture critical members

Areas Subjected to Movement

The most common types of movement observed during the inspection of abutments and wingwalls are:

- Vertical movement
- Lateral movement
- Rotational movement

Vertical movement can occur in the form of uniform settlement or differential settlement. A uniform settlement of the bridge substructure units, including abutments, and piers and bents, has little effect on the structure. Uniform settlements of one foot have been detected on small bridges with no signs of distress.

Differential settlement can produce severe distress in a bridge. Differential settlement may occur between different substructure units, causing damage of varying magnitude depending on span length and bridge type (see Figure 12.1.39). It may also occur under a single substructure unit (see Figure 12.1.40). This may cause an opening of the expansion joint between the abutment and wingwall, or it may cause cracking or tipping of the abutment, pier, or wall.

The most common causes of vertical movement are soil bearing failure, consolidation of soil, scour, undermining and subsidence from mining or solution cavities.

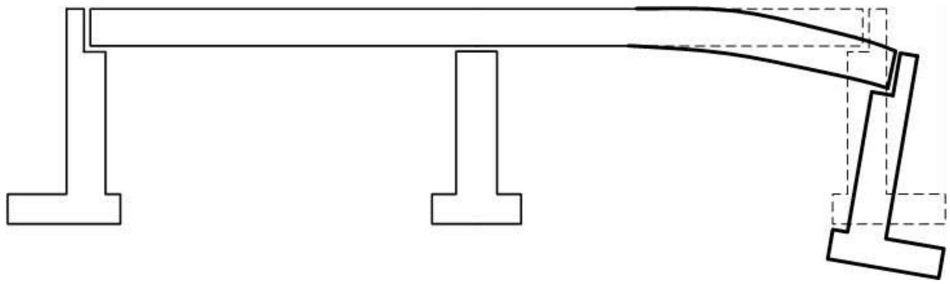


Figure 12.1.39 Differential Settlement Between Different Substructure Units

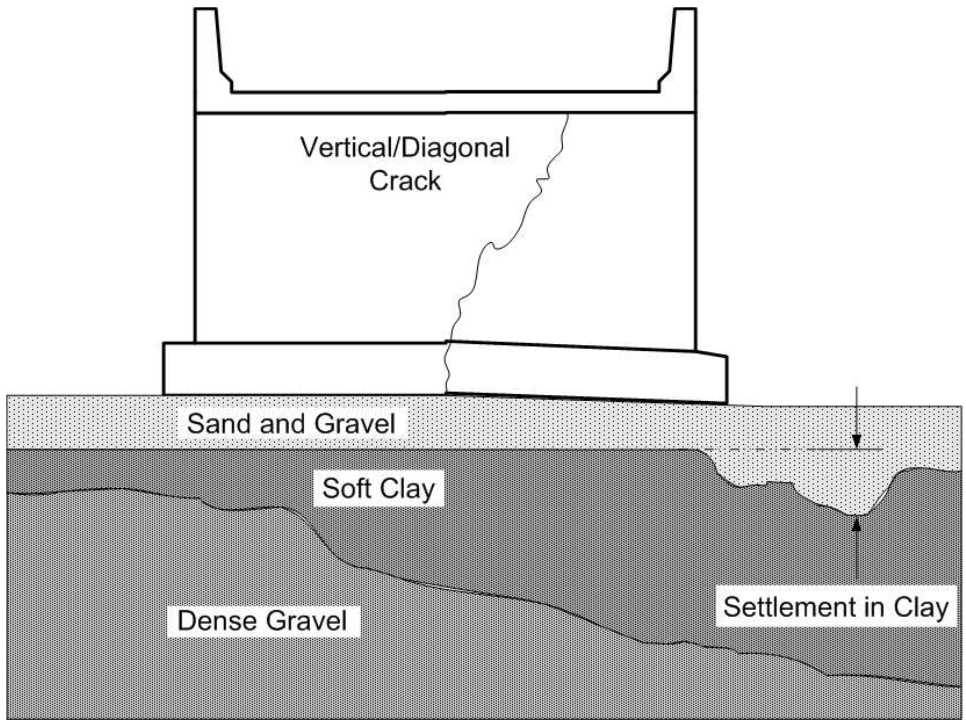


Figure 12.1.40 Differential Settlement Under an Abutment

Inspection for vertical movement, or settlement, includes:

- Inspect the joint opening between the end of the approach slab and the bridge deck. In some cases, pavement expansion or approach fill expansion could conceivably cause vertical movement in the approach slab.
- Investigate existing and new cracks for signs of settlement (see Figure 12.1.41).
- Examine the superstructure alignment for evidence of settlement (particularly the bridge railing and deck joints).
- Check for scour and undermining around the abutment footing or foundation.
- Inspect the joint that separates the wingwall and abutment for proper alignment.

Figure 12.1.41 Crack in Abutment due to Settlement

Earth retaining structures, such as abutments and retaining walls, are susceptible to lateral movements, or sliding (see Figure 12.1.42). Lateral movement occurs when the horizontal earth pressure acting on the wall exceeds the friction forces that hold the structure in place.

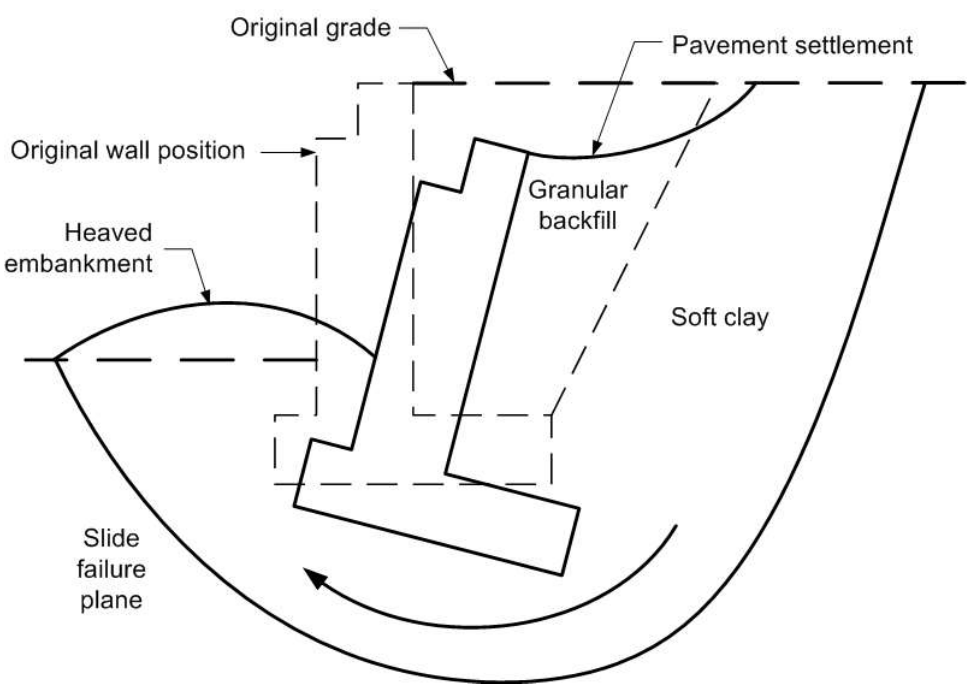


Figure 12.1.42 Lateral Movement of an Abutment due to Slope Failure

The most common causes of lateral movement are slope failure, seepage, changes in soil characteristics (e.g., frost action and ice), and time consolidation of the original soil.

Inspection for lateral movement, or sliding, includes:

- Inspect the general alignment of the abutment.
- Check the bearings for evidence of lateral displacement (see Figure 12.1.43).
- Examine the opening in the construction joint between the wingwall and the abutment.
- Investigate the joint opening between the deck and the approach slab (see Figure 12.1.44).
- Check the approach roadway for settlement.
- Check the distance between the end of the superstructure and the backwall.
- Examine for clogged drains (approach roadway, weep holes, and substructure drainage).
- Inspect for erosion, scour or undermining of the embankment material in front of the abutment or wingwall (see Figure 12.1.45).

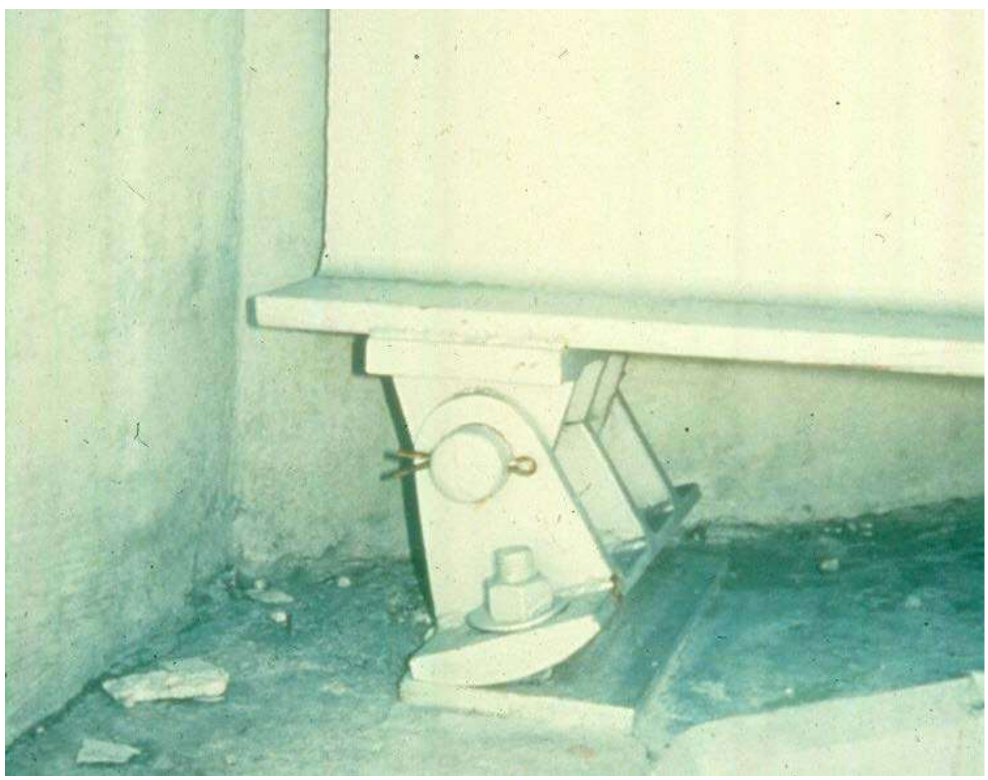


Figure 12.1.43 Excessive Rocker Bearing Displacement Indicating Possible Lateral Displacement of Abutment

Figure 12.1.44 Vertical Misalignment Between Approach Slab (left) and Bridge Deck (right)

Figure 12.1.45 Erosion at Abutment Exposing Footing

Rotational movement, or tipping, of substructure units is generally the result of differential settlements, lateral movements, or a combination of both due to horizontal earth pressure (see Figure 12.1.46). Abutments and walls are subject to this type of movement.

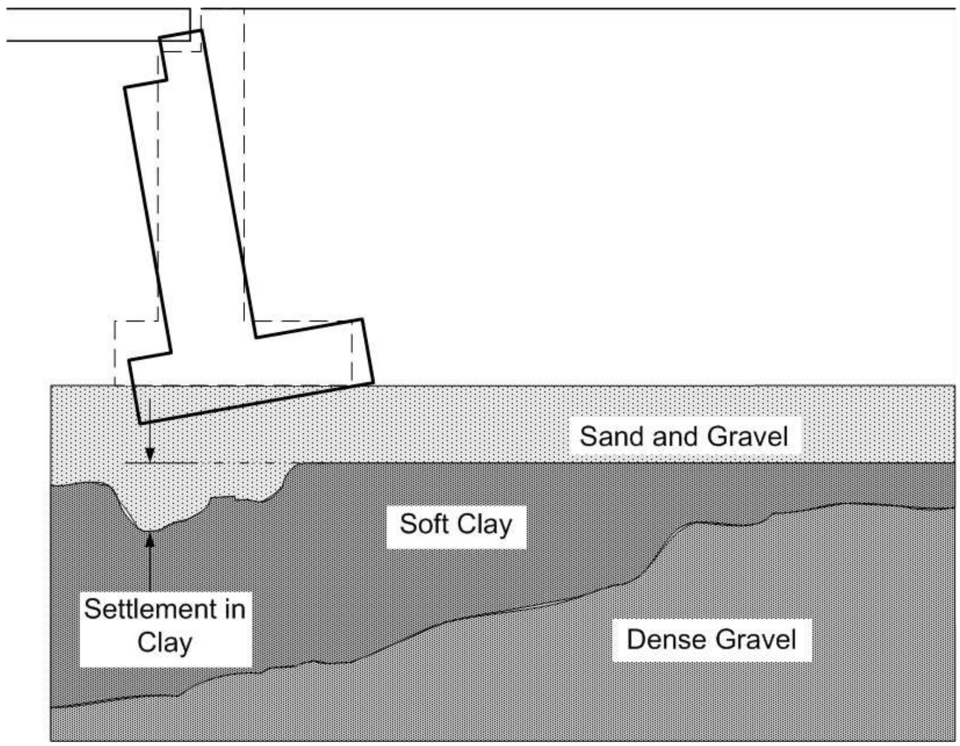


Figure 12.1.46 Rotational Movement of an Abutment

The most common causes of rotational movement are differential settlement, undermining, scour, saturation of backfill, soil bearing failure, erosion of backfill along the sides of the abutment, and improper design.

Inspection for rotational movement, or tipping, includes:

- Check the vertical alignment of the abutment using a plumb bob or level; keep in mind that some abutments are constructed with a battered or sloped front face (see Figures 12.1.47, 12.1.48 and 12.1.49).
- Examine the clearance between the beams and the backwall.
- Inspect for clogged drains or weep holes.
- Investigate for unusual cracks or spalls.
- Check for scour or undermining around the abutment footing. See Topic 13.2 for a detailed description of scour and undermining. See Topic 13.3 for a detailed description of underwater inspection.

Figure 12.1.47 Rotational Movement at Abutment

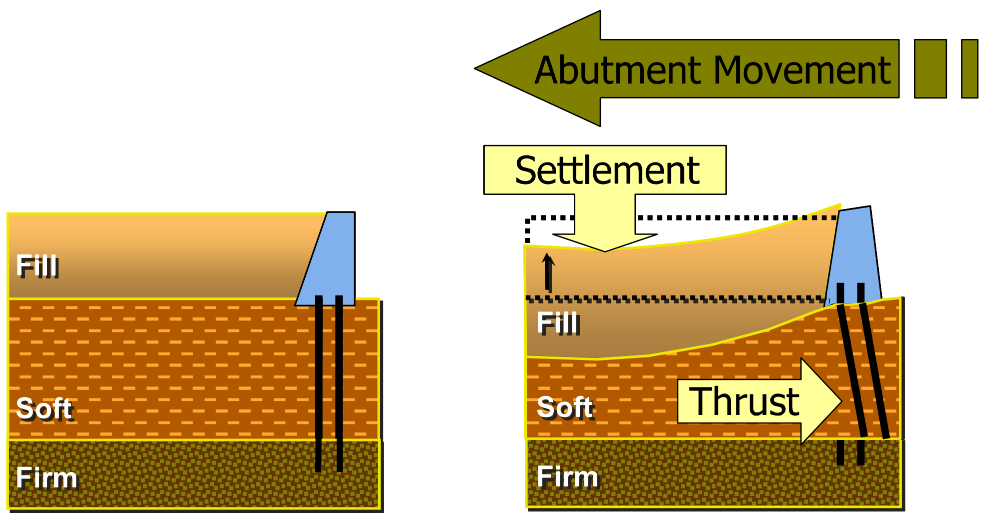


Figure 12.1.48 Rotational Movement due to "Lateral Squeeze" of Embankment Material

Figure 12.1.49 Rotational Movement at Concrete Wingwall

Bearing Areas

High bearing zones include the bridge seats, the abutment stem and footing connection, and the area where the footing is supported by earth or deep foundations. In timber abutments, look for crushing. Look for cracking or spalling in concrete and masonry members. Examine steel members for buckling or distortion.

Shear Zones

Horizontal forces cause high shear zones at the bottom of the backwall, and bottom of abutment stem. In timber abutments, look for splitting. Look for diagonal cracks in concrete and masonry. Examine steel members for buckling or distortion.

Flexural Zones

High flexural moments caused by horizontal forces occur at the bottom of the backwall and abutment stem connection. High flexural moments may be occurring at the footing toe and abutment stem. Moments cause compression and tension depending on the load type and location of the member neutral axis. Look for deterioration caused by overstress due to compression or tension caused by flexural moments. Check compression areas for timber splitting, concrete crushing or steel buckling. Examine tension members for cracking or distortion.

Areas Exposed to Drainage

Water can leak through the deck joints. Examine areas such as backwalls and bridge seats for signs of water leakage, and dirt and debris build-up. Look for material deficiencies caused by exposure to moisture, such as corrosion and section loss on steel, spalls and delaminations on concrete and decay on timber. Examine the abutment stem at the ground level or water level for similar deteriorations.

Water can build up horizontal pressure behind an abutment. Allowing the water to exit from behind the abutment relieves this pressure. Weep holes, normally four inches in diameter, allow water to pass through the abutment. Sometimes abutments have subsurface drainage pipes that are parallel to the rear face of the abutment stem. These pipes are sloped to drain the water out at the end of the abutment.

Check weep holes and subsurface drainage pipes to see that they are clear and functioning. Be careful of any animal or insect nests that may be in the weep holes. Look for signs of discoloration under the weep holes, which may indicate that the weep holes or substructure drainage pipes are functioning properly. Check the condition of any drainage system that is placed adjacent to the abutment that may result in deterioration of the abutment.

Areas Exposed to Traffic

Check for collision damage from vehicles or vessels passing adjacent to structural members.

Damage to concrete abutments may include spalls and exposed reinforcement and possibly steel reinforcement section loss. Steel abutments may experience cracks, section loss, or distortion which needs to be documented. Timber abutments may experience cracks, section loss, distortion or loose connections which need to be documented.

Areas Previously Repaired

Examine thoroughly any repairs that have been previously made. Determine if repaired areas are sound and functioning properly.

For concrete members, effective repairs and patching are usually limited to protection of exposed reinforcement. For steel members, document the location and condition of any repair plates and their connections. For timber members, document the location and condition of repaired areas and their connections.

Scour and Undermining

Scour is the removal of material from a streambed as a result of the erosive action of running water (see Figure 12.1.50). Scour can cause undermining or the removal of supporting foundation material from beneath the abutments when streams or rivers flow adjacent to them. Refer to Topic 13.2 for a more detailed description of scour and undermining.

Figure 12.1.50 Abutment with Undermining due to Scour

Inspection for scour includes probing around the abutment and wingwall footings for signs of undermining (see Figure 12.1.51 and 12.1.52). Sometimes silt loosely fills in a scour hole and offers no protection or bearing capacity for the abutment footing.

Figure 12.1.51 Inspector Checking for Scour

Figure 12.1.52 Scour and Possible Undermining of Concrete Wingwall

Problematic Details and Fracture Critical Members

Steel abutments may contain problematic or fatigue prone details. Closely examine these details for section loss due to corrosion and cracking. The members of a steel abutment may be fracture critical. See Topic 6.4 for a detailed description of problematic details and fracture critical members.

12.1.5 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of substructures. 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 condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment method.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the entire substructure including abutments and piers. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 60) for additional details about NBI component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating. Recognize that abutments may be affected by scour or other conditions that may only be able to be accessed and evaluated by a separate underwater inspection. Therefore, the results of both the routine and underwater inspection, if applicable, are integrated and evaluated together to arrive at the correct component condition rating for the substructure. Note the findings of the underwater inspection in the narrative portion of the routine inspection report as documentation and justification for the determined substructure component condition rating code.

Element Level Condition State Assessment

In an element level condition state assessment of an abutment, possible AASHTO Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Substructure	
219	Steel Abutment
215	Reinforced Concrete Abutment
216	Timber Abutment
217	Masonry Abutment
218	Other Abutment
202	Steel Column/Pile Extension
225	Steel Submerged Pile
231	Steel Pier Cap
204	Prestressed Concrete Column/Pile Extension
226	Prestressed Concrete Submerged Pile
233	Prestressed Concrete Pier Cap
205	Reinforced Concrete Column/Pile Extension
220	Reinforced Concrete Pile Cap/Footing
227	Reinforced Concrete Submerged Pile
234	Reinforced Concrete Pier Cap
206	Timber Column/Pile Extension
228	Timber Submerged Pile
235	Timber Pier Cap

BME No.	Description
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating
521	Concrete Protective Coating

The unit quantity for the substructure elements is feet, measured horizontally across the abutment. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for columns and piles is each, and the total quantity is placed in one of the available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coatings is square feet, with the total area distributed among the four available condition states depending on the extent and severity of the deficiency. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of abutments:

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
361	Scour
363	Steel Section Loss
364	Steel Out-of-Plane (Compression Members)
367	Substructure Traffic Impact (load capacity)

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

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Chapter 12
Inspection and Evaluation of Substructures

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Topic 12.2 Piers and Bents

12.2.1 Introduction

A pier or bent is an intermediate substructure unit located between the ends of a bridge. Its function is to support the bridge at intermediate intervals with minimal obstruction to the flow of traffic or water below the bridge (see Figure 12.2.1). There is no functional difference between piers and bents. A pier generally has only one column or shaft supported by one footing. Bents have two or more columns and each column is supported by an individual footing.

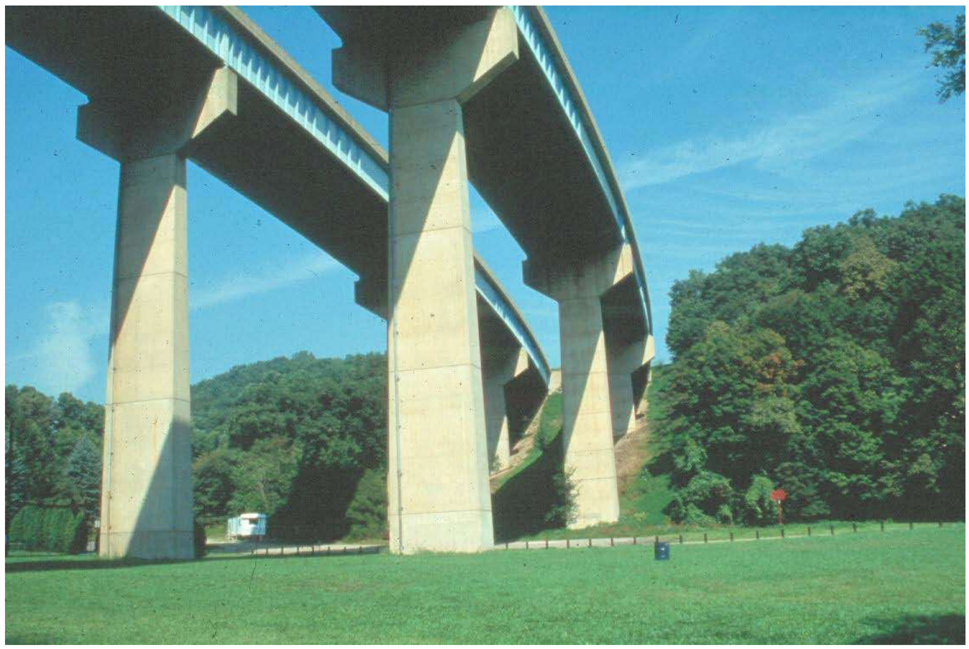

Figure 12.2.1 Example of Piers as Intermediate Supports for a Bridge

12.2.2 Design Characteristics

Pier and Bent Types

The most common pier and bent types are:

- Solid shaft pier (see Figure 12.2.2)
- Column pier (see Figure 12.2.3)
- Column pier with web wall (see Figures 12.2.4 and 12.2.5)
- Cantilever pier or hammerhead pier (see Figures 12.2.6 and 12.2.7)
- Column bent or open bent (see Figure 12.2.8)
- Pile bent (see Figure 12.2.9)

Detailed descriptions of pier and bent members are provided on page 12.2.13.

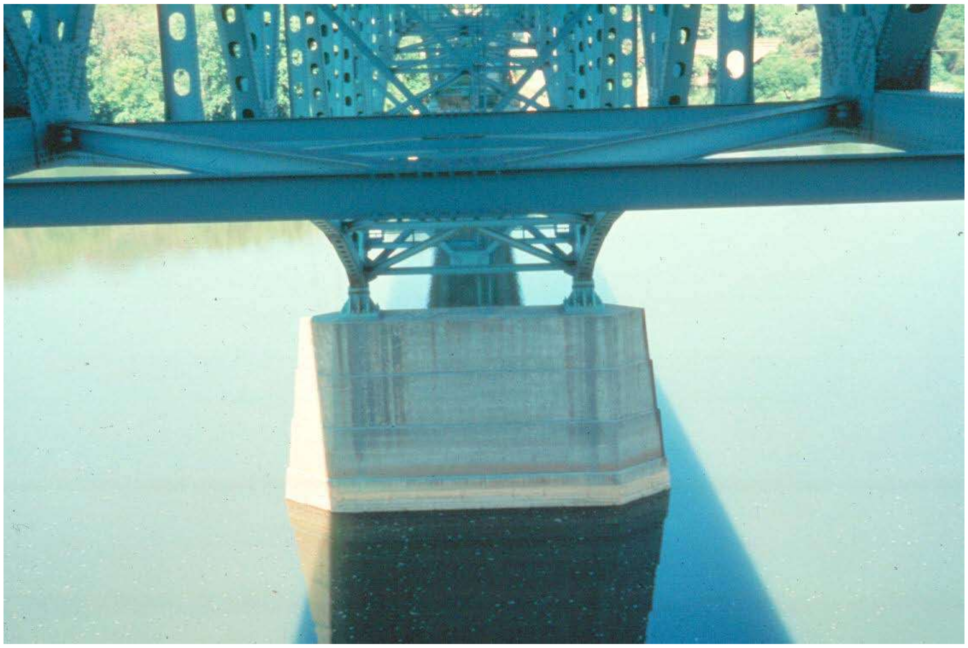

Figure 12.2.2 Solid Shaft Pier

Solid shaft piers are used when a large mass is advantageous or when a limited number of load points are required for the superstructure.

Figure 12.2.3 Column Pier

Column piers are used when limited clearance is available under the structure or when narrow superstructure widths are required.

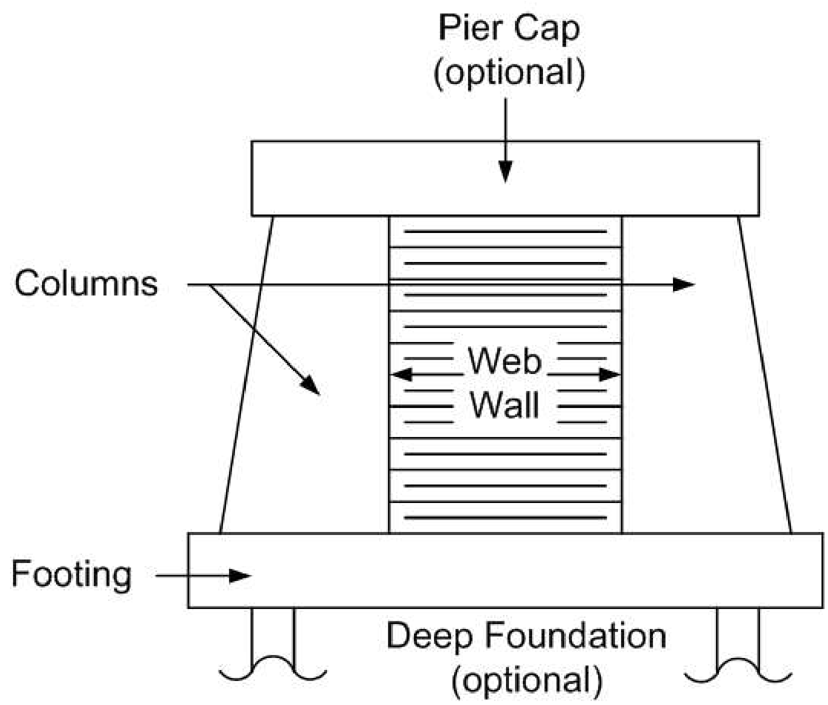

Figure 12.2.4 Column Pier with Web Wall

A web wall can be connected to columns to add stability to the pier. The web wall is non-structural relative to superstructure loads. Web walls also serve to strengthen the columns in the event of a vehicular collision.

Figure 12.2.5 Column Pier with Web Wall

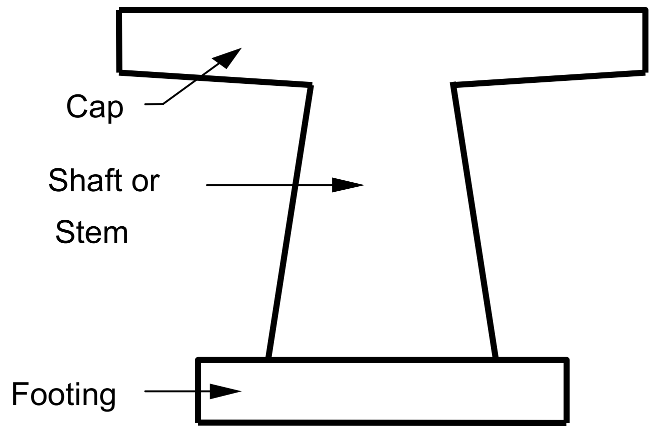

Figure 12.2.6 Single Stem Pier (Cantilever or Hammerhead)

The cantilever or hammerhead pier is a modified column pier for use with wide superstructures.

Figure 12.2.7 Cantilever Pier

Figure 12.2.8 Column Bent or Open Bent

The column bent is a common pier type for highway grade crossings.

Figure 12.2.9 Concrete Pile Bent

Pile bents may be constructed of concrete, steel or timber. Typically, piles are driven in place and support a continuous cap or timber cap for timber piles.

Two other specialized types of piers include the hollow pier and the integral pier. Hollow piers are usually tall shaft type piers built for bridges crossing deep valleys Being hollow greatly reduces the dead load of the pier and increases its ductility. Whether precast or cast-in-place, hollow piers are constructed in segments. If precast, the segments are post-tensioned together and the joints are epoxy-sealed.

The decrease in the dead load, or self-weight, of the piers provides eases in transporting segments to the site, and the high ductility provides for better performance against seismic forces.

Integral piers incorporate the pier cap into the depth of the superstructure. Integral piers provide for a more rigid structure, and they are typically used in situations where vertical clearance beneath the structure is limited. Integral piers may consist of steel or cast-in-place concrete caps within a girder superstructure. The concrete cap is likely to be post-tensioned rather than conventionally reinforced (see Figures 12.2.10 thru 12.2.12).

Figure 12.2.10 Concrete Pier with Integral Steel Pier Cap

Figure 12.2.11 Integral Concrete Pier and Pier Cap

Figure 12.2.12 Integral Concrete Pier and Pier Cap

Primary Materials

The primary materials used in pier and bent construction are unreinforced concrete, reinforced concrete, stone masonry, steel, timber, or a combination of these materials (see Figures 12.2.13 thru 12.2.17).

Figure 12.2.13 Reinforced Concrete Piers under Construction

Figure 12.2.14 Stone Masonry Pier

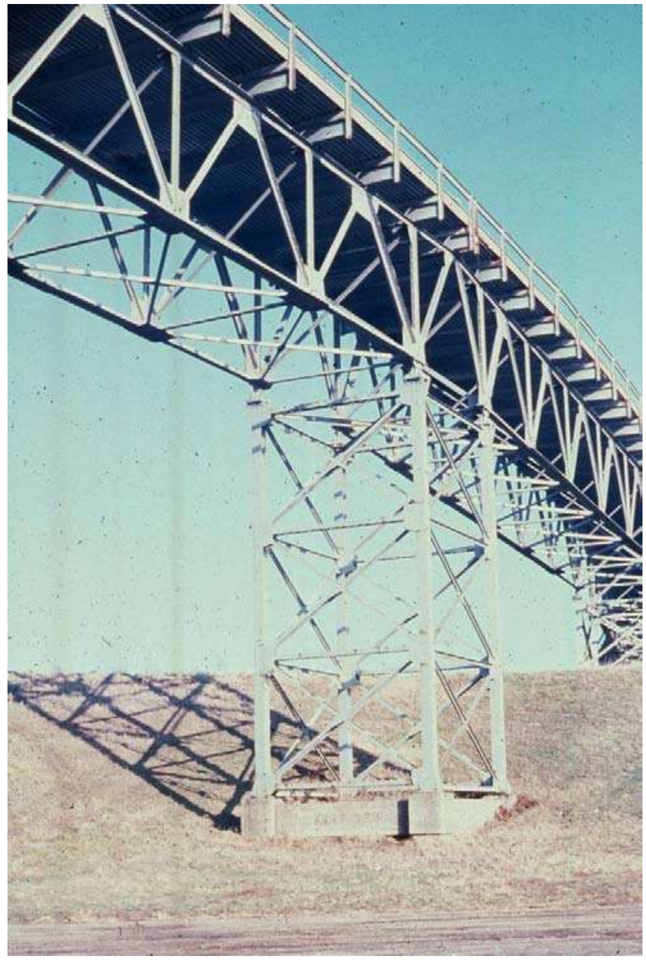

Figure 12.2.15 Steel Bent

Figure 12.2.16 Timber Pile Bent

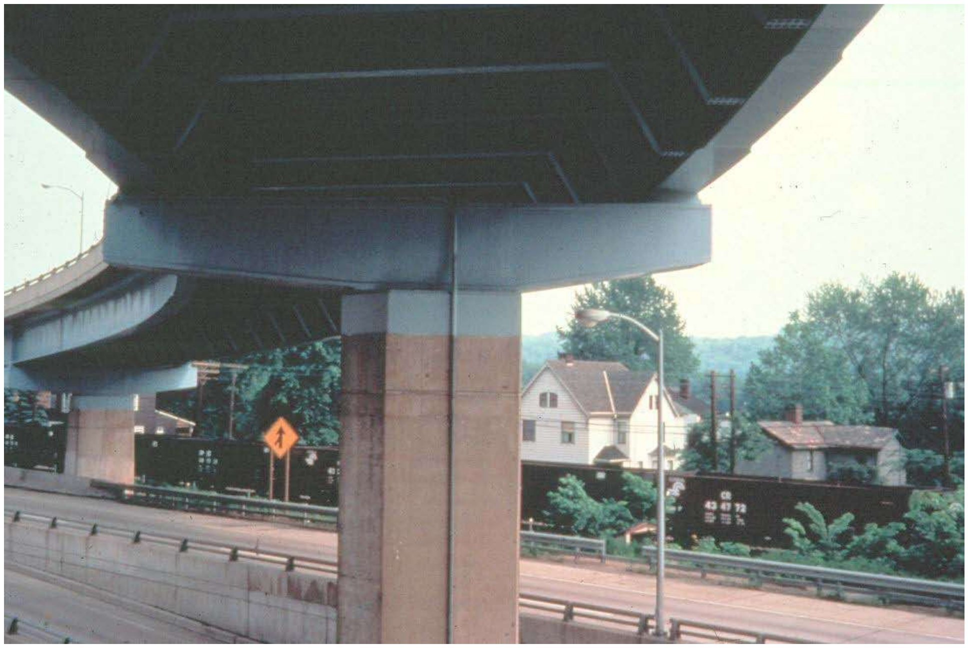

Figure 12.2.17 Combination: Reinforced Concrete Column with Steel Pier Cap

Primary and Secondary Reinforcement

The pattern of primary reinforcement for concrete piers depends upon the pier configuration. Piers with relatively small columns, whether of the single shaft, multi-column, or column and web wall design, have heavy vertical reinforcement confined within closely spaced ties or spirals in the columns. Pier caps are reinforced according to their beam function. Cantilevered caps have primary tension steel near the top surface. Caps spanning between columns have primary tension steel near the bottom surface. Primary shear steel consists of vertical stirrups, usually more closely spaced near support columns or piles.

Wall type piers are more lightly reinforced, but still have significant vertical reinforcement to resist horizontal loads.

If primary steel is not required at a given location, then secondary reforcement for temperature and shrinkage is provided. Each concrete face is reinforced in both the vertical and horizontal directions.

Pier foundations are likewise reinforced to match their function in resisting applied loads. Shear stirrups are generally not required for footings as they are designed thick enough to permit only the concrete to resist the shear. Modern designs, however, do incorporate seismic ties (vertical bars with hooks at each end) to tie the top and bottom mats of rebar together.

Figures 12.2.18 thru 12.2.21 illustrate typical reinforcement patterns.

New design specifications may call for epoxy coated reinforcement if the substructure is subjected to de-icing chemicals or salt water.

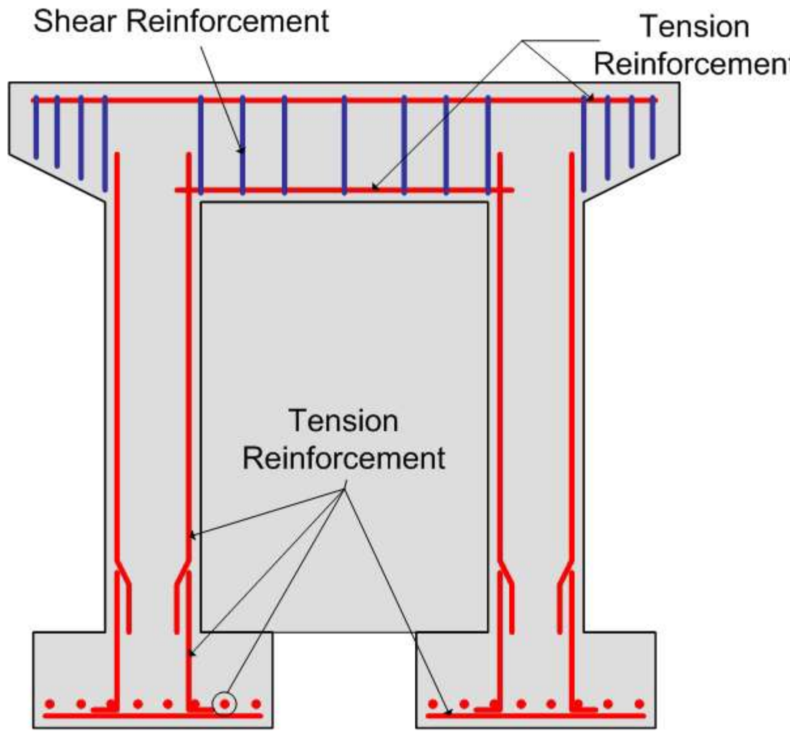

Figure 12.2.18 Primary Reinforcement in Column Bent with Web Wall

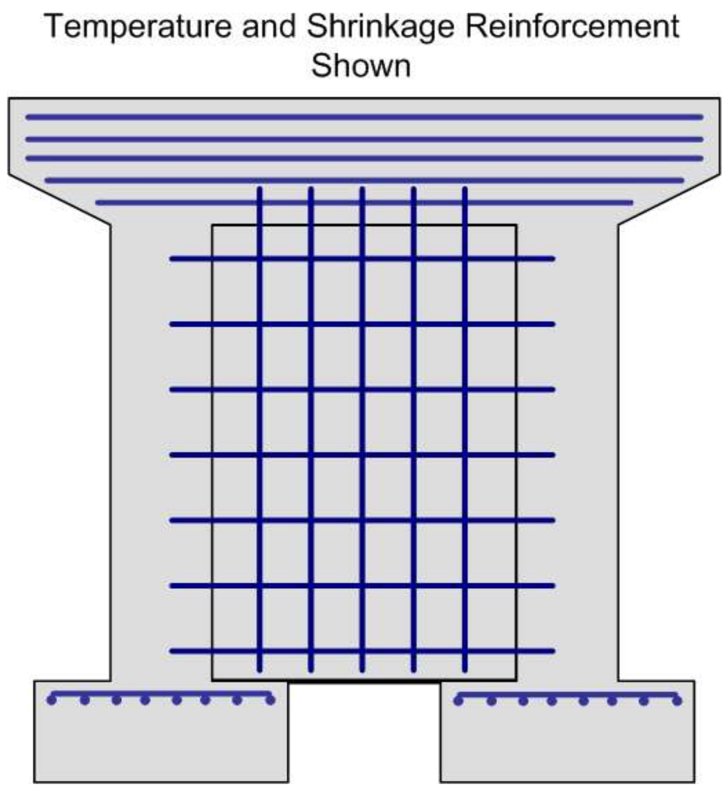

Figure 12.2.19 Secondary Reinforcement in Column Bent with Web Wall

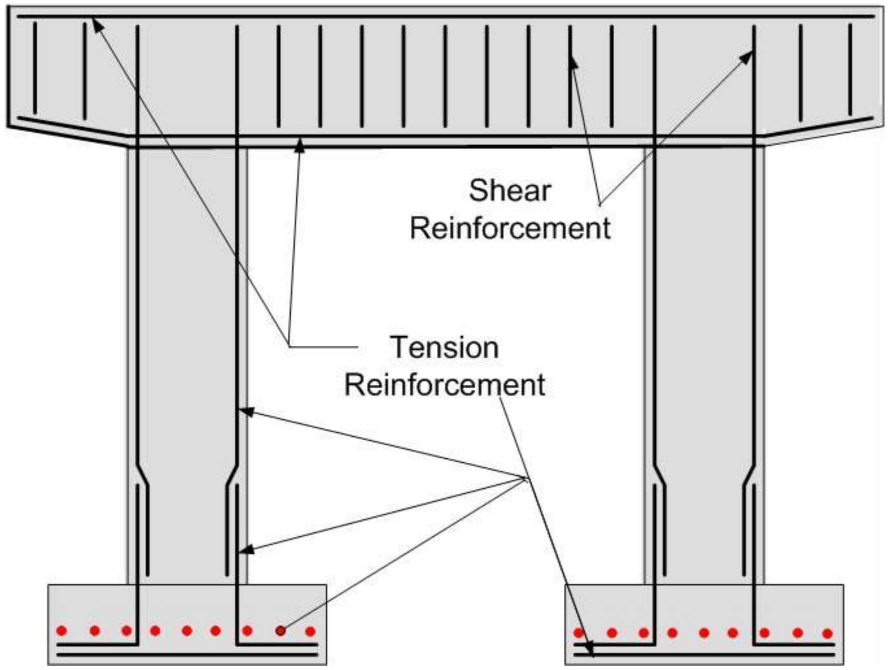

Figure 12.2.20 Primary Reinforcement in Column Bents

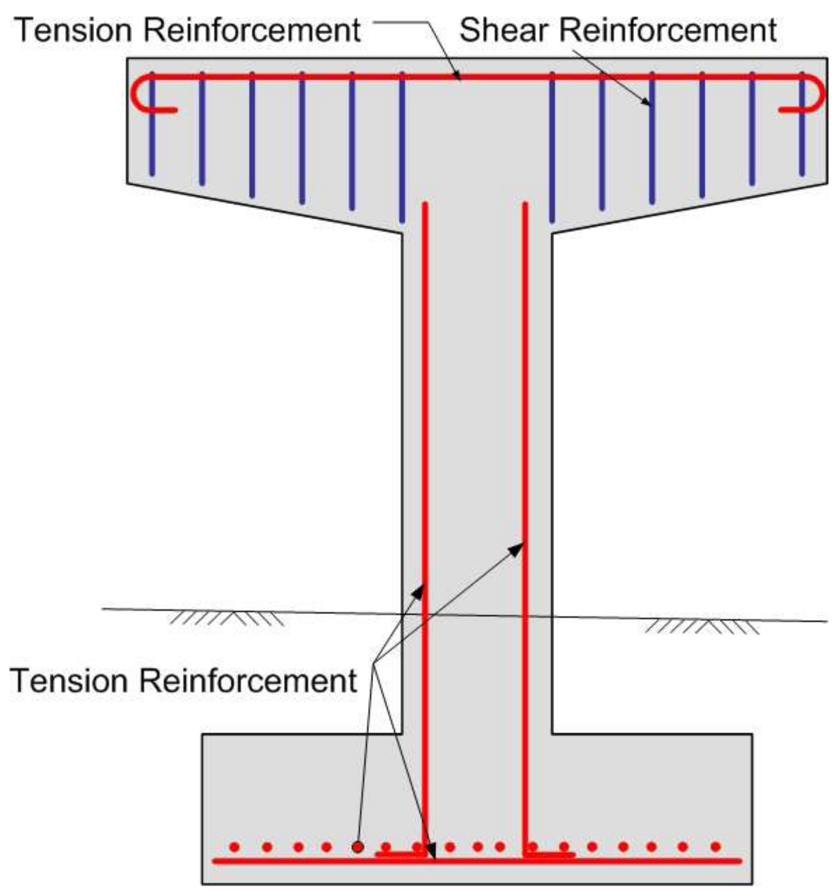

Figure 12.2.21 Primary Reinforcement for a Cantilevered Pier

Pier and Bent Members

The primary pier and bent members are:

- Pier cap or bent cap
- Pier wall / stem / or shaft
- Column
- Footing
- Piles or Drilled Shafts

The pier cap or bent cap provides support for the bearings and the superstructure (see Figures 12.2.22 and 12.2.23).

The pier wall or stem transmits loads from the pier cap to the footing.

Columns transmit loads from the pier or bent cap to the footing.

The footing transmits the weight of piers or bents, and the bridge reactions to the supporting soil or rock. The footing also provides stability to the pier or bent against overturning and sliding forces.

Figure 12.2.22 Cantilevered Piers Joined by a Web Wall

Figure 12.2.23 Pile Bent

Foundation Types

Foundations are critical to the stability of the bridge since the foundation ultimately supports the entire structure. There are two basic types of bridge foundations:

- Spread footings
- Deep foundations

Spread footing and deep foundations are described on page 12.1.16 of Topic 12.1, Abutments and Wingwalls.

Pier Protection

Piers are vulnerable to collision damage from trucks, trains, ships, ice flows and waterborne debris. Wall type piers are resistant to this type of collision damage and for this reason are often used in navigable waterways and waterways subject to freezing. Web walls also serve to protect columns (see Figures 12.2.24 and 12.2.25). External barriers are often provided for single- or multi-column piers. Dolphins are single, large diameter, sand-filled, sheet pile cylinders; clusters of timber piles or steel tubes; or large concrete blocks placed in front of a pier to protect it from collision (see Figures 12.2.26 and 12.2.27). Fenders are protective fences surrounding a pier to protect it from marine traffic. They may consist of timber bent arrangements, steel or concrete frames, or cofferdam sheets (see Figures 12.2.28 and 12.2.29).

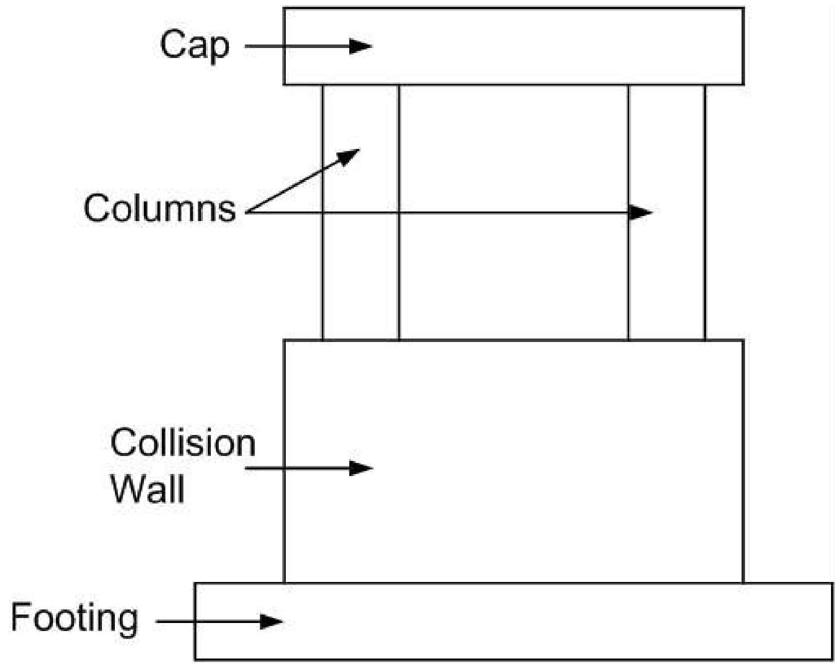

Figure 12.2.24 Collision Wall

Figure 12.2.25 Collision Wall

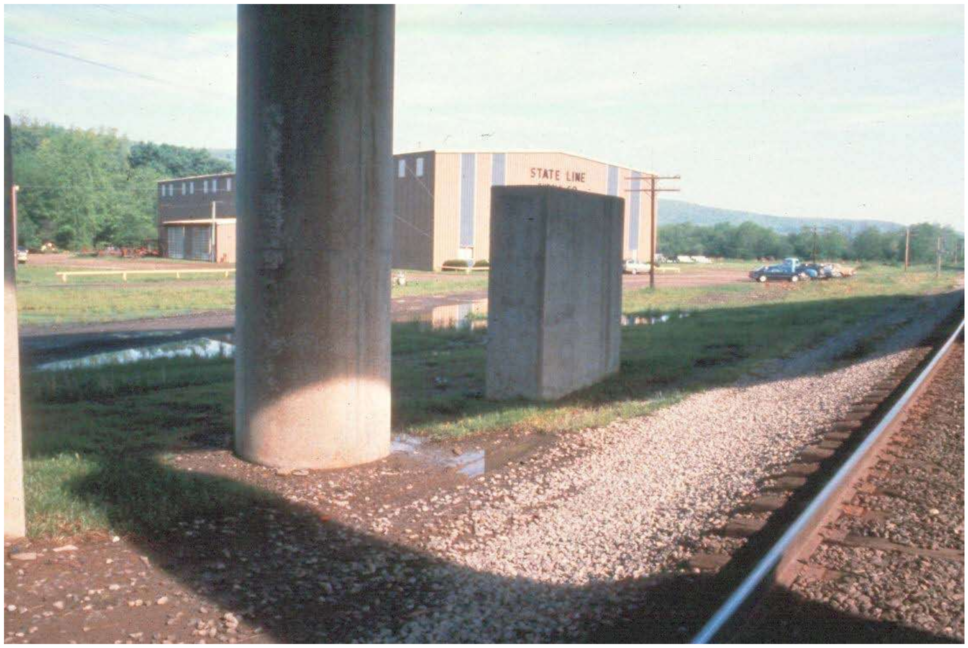

Figure 12.2.26 Concrete Block Dolphin

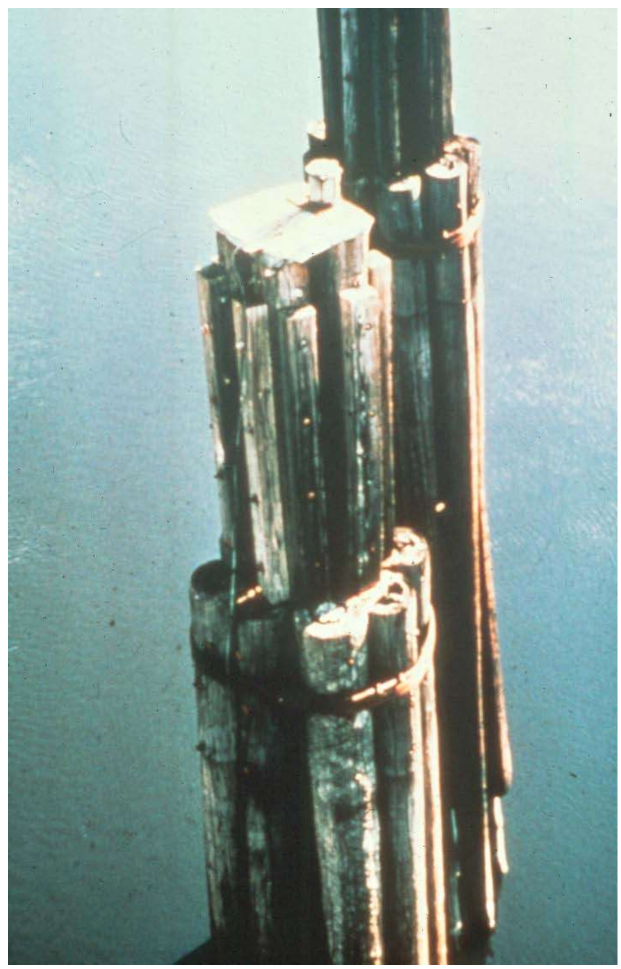

Figure 12.2.27 Timber Dolphin

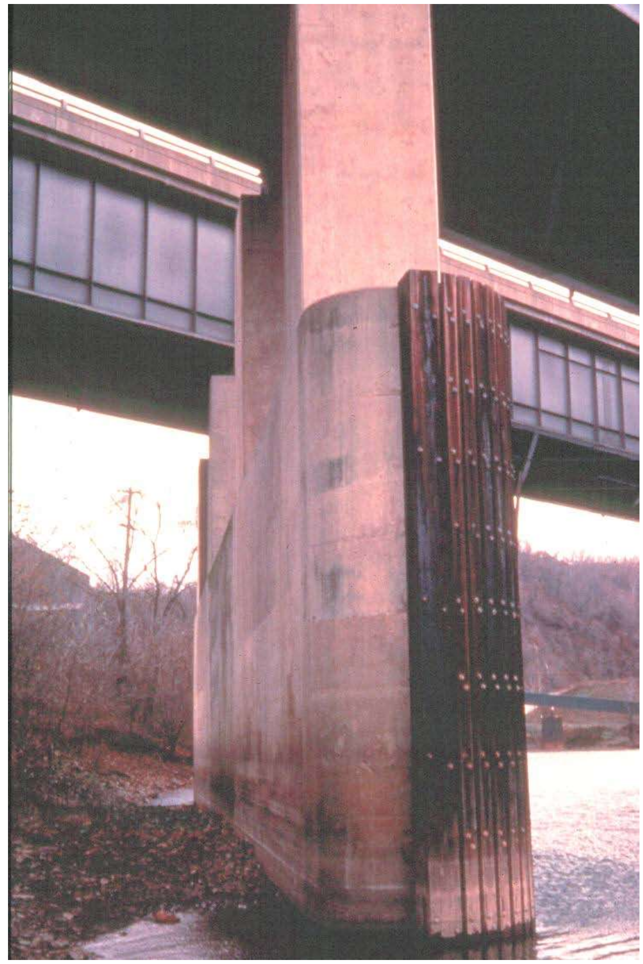

Figure 12.2.28 Pier Fender

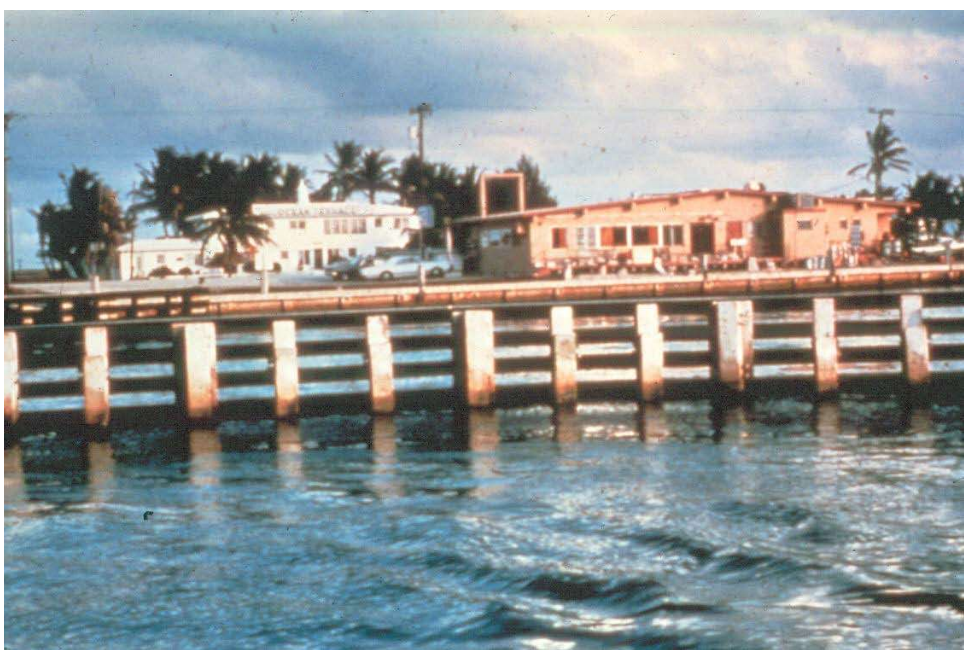

Figure 12.2.29 Fender System

12.2.3

Inspection Methods and Locations

Inspection methods for piers and bents are similar to superstructures, particularly when it involves material deterioration.

Methods

There are three basic methods used to inspect a member. Depending on the type of inspection, the inspector may be required to use only one individual method or all methods. They include:

- Visual
- Physical
- Advanced inspection methods

The inspection method used is based on the type of material the pier or bent is made of and the methods are similar to the inspection of superstructures. See Topics 6.1 and 15.1 (Timber), Topics 6.2 and 15.2 (Concrete), 6.3 and 15.3 (Steel), or Topic 6.4 (Stone Masonry) for specific material defects and inspection methods.

Visual

There are two types of visual inspections that may be required of an inspector. The first, called a routine inspection, involves reviewing the previous inspection report and visually examining the members of the bridge. A routine inspection involves a visual assessment to identify obvious deficiencies.

The second type of visual inspection is called an in-depth inspection. An in-depth inspection is an inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine methods. Hands-on inspection may be necessary at some locations. This type of visual inspection requires the inspector to visually assess all deficient surfaces at a distance no further than an arm's length. Surfaces are given close visual attention to quantify and qualify any deficiencies.

Concrete

As presented in Topic 6.2.6, visually inspect for the following concrete deficiencies:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation) (see Figure 12.2.31)
- Scaling
- Delamination
- Spalling (see Figures 12.2.30 and 12.2.32)
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali-Silica Reactivity (ASR)

- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage (see Figure 12.2.33)
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation
- Other causes (temperature changes, chemical attack, moisture absorption, differential foundation movement, design and construction deficiencies, unintended objects in concrete, fire damage)

Figure 12.2.30 Concrete Spalling due to Contaminated Drainage

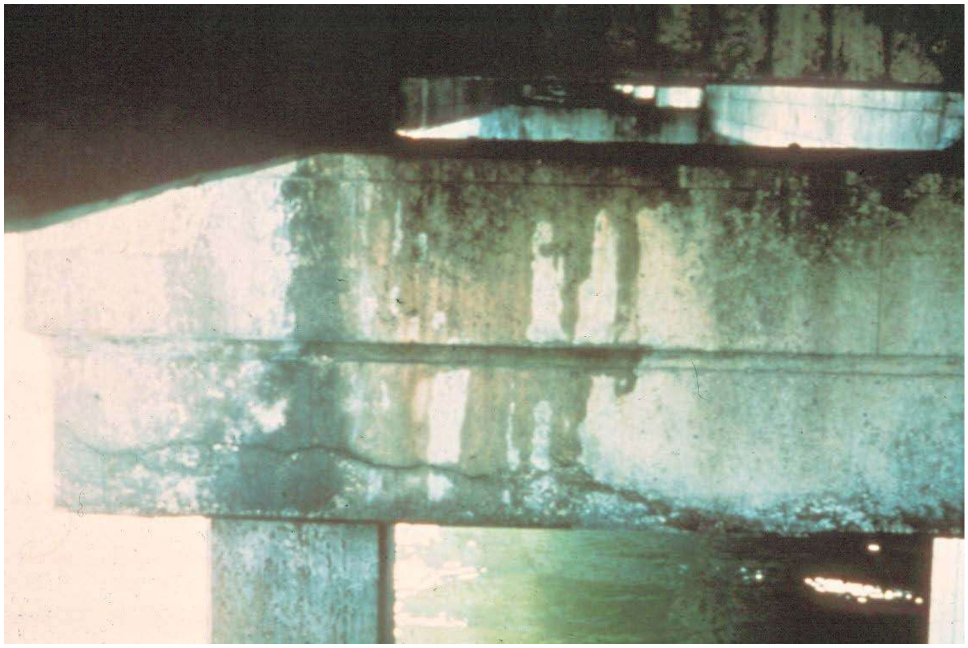

Figure 12.2.31 Crack in Concrete Bent Cap

Figure 12.2.32 Concrete Spalling on Bent Cap

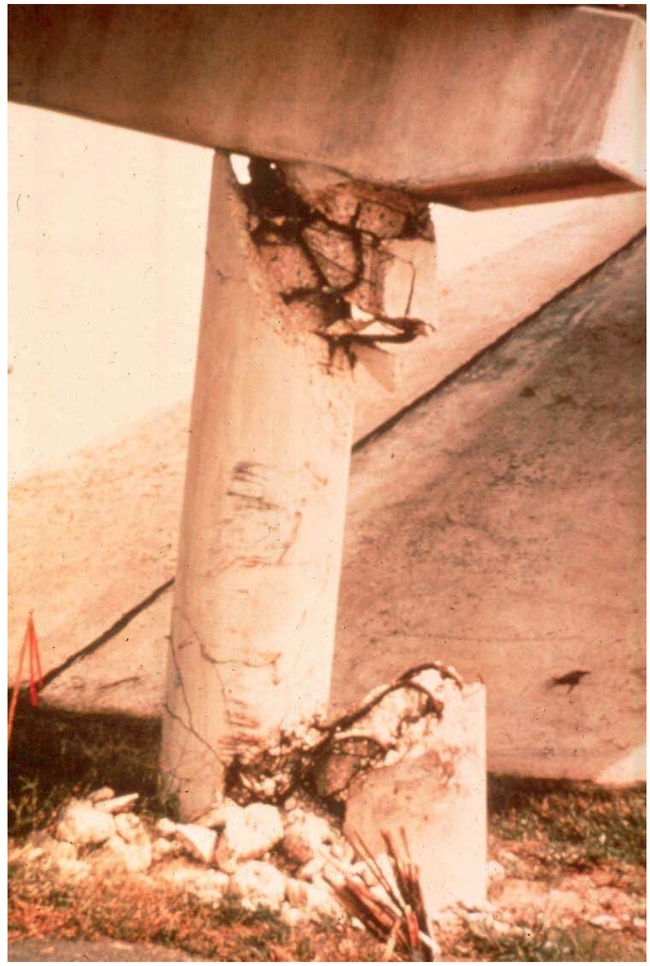

Figure 12.2.33 Collision Damage to Concrete Pier Column

Masonry

As presented in Topic 6.5.4, visually inspect for the following masonry deficiencies:

- Weathering – hard surfaces degenerate into small granules, giving stones a smooth, rounded look; mortar disintegrates
- Spalling – small pieces of rock break out (see Figure 12.2.34)
- Splitting – seams or cracks open up in rocks, eventually breaking them into smaller pieces (see Figure 12.2.34)
- Fire – masonry is not flammable but can be damaged by high temperatures

Figure 12.2.34 Deteriorated and Missing Stone at Masonry Pier

Steel

As presented in Topic 6.3.5, visually inspect for the following steel deficiencies:

- Corrosion (see Figures 12.2.35 and 12.2.36)
- Fatigue cracking
- Overloads (see Figure 12.2.37)
- Collision damage
- Heat damage
- Coating failures

Figure 12.2.35 Deterioration of Steel Bent Leg

Figure 12.2.36 Corrosion of Steel Pile Bent at Water Surface

Figure 12.2.37 Steel Column Pile Bent with Cantilever - High Stress Areas for Moment, Shear and Bearing

Timber

As presented in Topic 6.1.5, visually inspect for the following timber deficiencies:

- Inherent defects: checks, splits, shakes, knots
- Fungi (see Figures 12.2.38 and 12.2.40)
- Insects
- Marine borers (see Figures 12.2.42 and 12.2.43)
- Chemical attack
- Delaminations
- Loose connection (see Figure 12.2.40)
- Surface depressions
- Fire
- Collision damage
- Wear
- Abrasion (see Figure 12.2.39)
- Overstress (see Figure 12.2.41)
- Protective coating failure

Several advanced methods are available for timber inspection. Non-destructive and other methods are described in Topics 13.1.2 and 13.1.3.

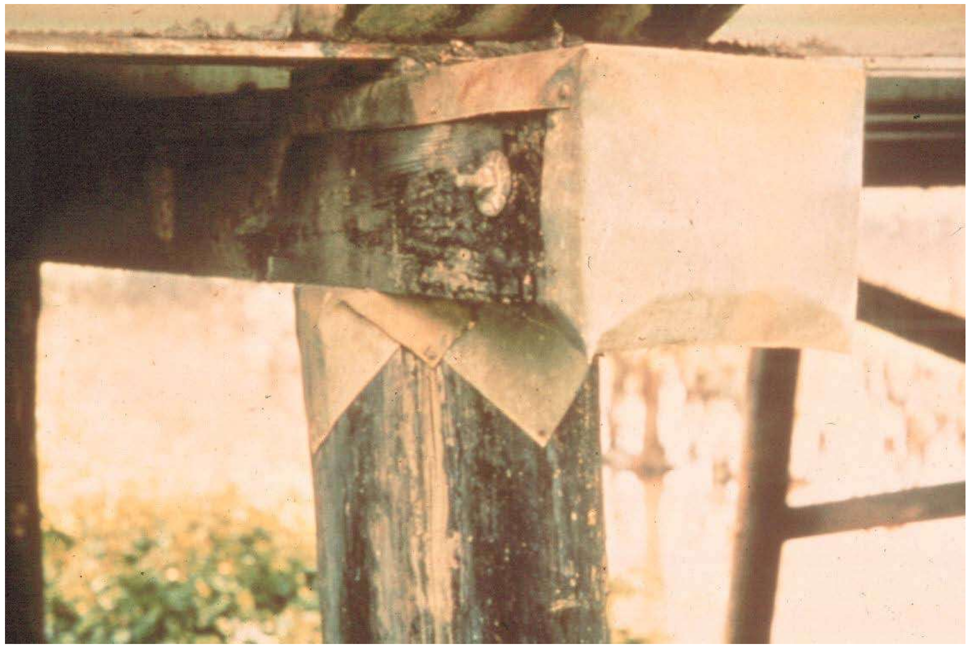

Figure 12.2.38 Decay in Timber Bent Cap (Note "Protective" Cover / Flashing)

Figure 12.2.39 Timber Bent Columns in Water

Figure 12.2.40 Decay of Timber Bent Column at Ground Line/Loose Connection

Figure 12.2.41 Timber Pile Bent with Overstress-Partial "Brooming" Failure at First Pile

Figure 12.2.42 Timber Pile Damage due to Limnoria Marine Borers

Figure 12.2.43 Timber Bent Damage due to Shipworm Marine Borers

Physical

Once the deficiencies are identified visually, physical methods are used to verify the extent of the deficiencies. Carefully measure and record deficiencies found during physical inspection methods.

Areas of concrete or rebar deterioration identified visually need to be examined physically using an inspection hammer. This hands-on effort verifies the extent of the deficiency and its severity. A delaminated area has a distinctive hollow "clacking" sound when tapped with a hammer. The location, length and width of cracks found during the visual inspection need to be measured and recorded.

For steel members, the main physical inspection methods involve the use of an inspection hammer or wire brush. Excessive hammering, brushing or grinding may close surface cracks and make the cracks difficult to find. Corrosion results in loss of member material. This partial loss of cross section due to corrosion is known as section loss. Section loss may be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or an ultrasonic thickness gauge (D-meter), are used to measure the remaining section of steel. The inspector removes all corrosion products (rust scale) prior to taking measurements.

For timber members, an inspection hammer is used to tap on areas and determine the presence and extent of internal decay. This is done by listening to the sound the hammer makes. If it sounds hollow, internal decay may be present.

Advanced Inspection Methods

If the extent of the deficiency cannot be determined by the visual and physical inspection methods described above, advanced inspection methods are used.

For concrete inspections, non-destructive methods, described in Topic 15.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Electrical methods
- Delamination detection machinery
- Ground-penetrating radar
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
- Impact-echo testing
- Infrared thermography
- Laser ultrasonic testing
- Magnetic field disturbance
- Neutron probe for detection of chlorides
- Nuclear methods

- Pachometer
- Rebound and penetration methods
- Ultrasonic testing
- Smart concrete
- Carbonation

Other advanced methods for concrete members, described in Topic 15.2.3, include:

- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Petrographic examination
- Reinforcing steel strength
- Chloride test
- Matrix analysis
- ASR evaluation

For steel inspections, non-destructive methods, described in Topic 15.3.2, include:

- Acoustic emissions testing
- Corrosion sensors
- Smart coatings
- Dye penetrant
- Magnetic particle
- Radiography testing
- Computed tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Electrochemical fatigue sensor (EFS)
- Magnetic flux leakage (external PT tendons and stay cables)
- Laser vibrometer (for stay cable vibration measurement and cable force determination)

Other advanced methods for steel members, described in Topic 15.3.3, include:

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

For timber inspections, non-destructive methods, described in Topic 15.1.1, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other advanced methods for timber members, described in Topic 15.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Field Ohmmeter

Locations

Stability is a paramount concern; therefore checking for various forms of movement is required during the inspection of piers or bents.

The locations for inspection can be related to common pier and bent problems.

The most common problems observed during the inspection of piers and bents are associated with:

- Areas subjected to movement
- High stress areas
- Areas exposed to drainage
- Areas exposed to traffic
- Areas previously repaired
- Scour and undermining
- Problematic details and fracture critical members
- Dolphins and fenders

Areas Subjected to Movement

The most common types of movement observed during the inspection of piers and bents are:

- Vertical movement
- Lateral movement
- Rotational movement

Vertical movement can occur in the form of differential settlement. Differential settlement at piers can cause severe problems in a bridge (see Figures 12.2.44 and 12.2.45). Deck joints can open excessively or close up completely. Local deterioration, such as spalling, cracking, and buckling, can also occur.

The most common causes of vertical movement are soil bearing failure, soil consolidation, scour, undermining, and subsidence from mining or solution cavities.

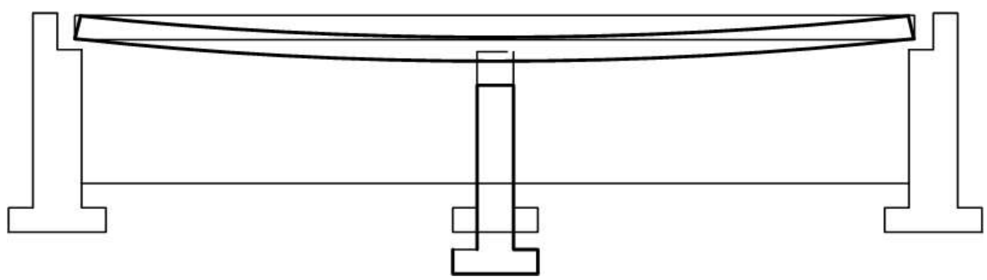

Figure 12.2.44 Differential Settlement Between Different Substructure Units

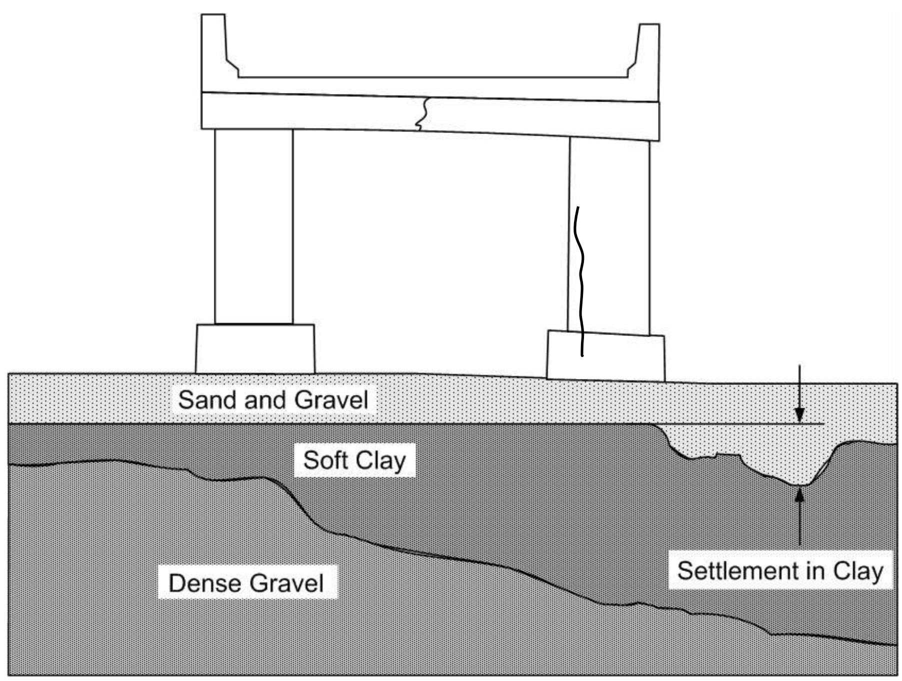

Figure 12.2.45 Differential Settlement Under a Pier

Inspection for vertical movement, or settlement, includes:

- For bridges with multiple simple spans, examine the joint in the deck above the pier as well as at adjacent piers and at the abutments.
- Check for any new or unusual cracking in the pier or bent.
- Investigate for buckling in steel columns of the pier or bent.
- Check the superstructure for evidence of settlement. Sight along parapets, bridge rails, etc. (see Figure 12.2.46).
- Investigate for scour and undermining around the pier footing.
- In some cases, a check of bearing seat or top of pier elevations using surveying equipment may be necessary.

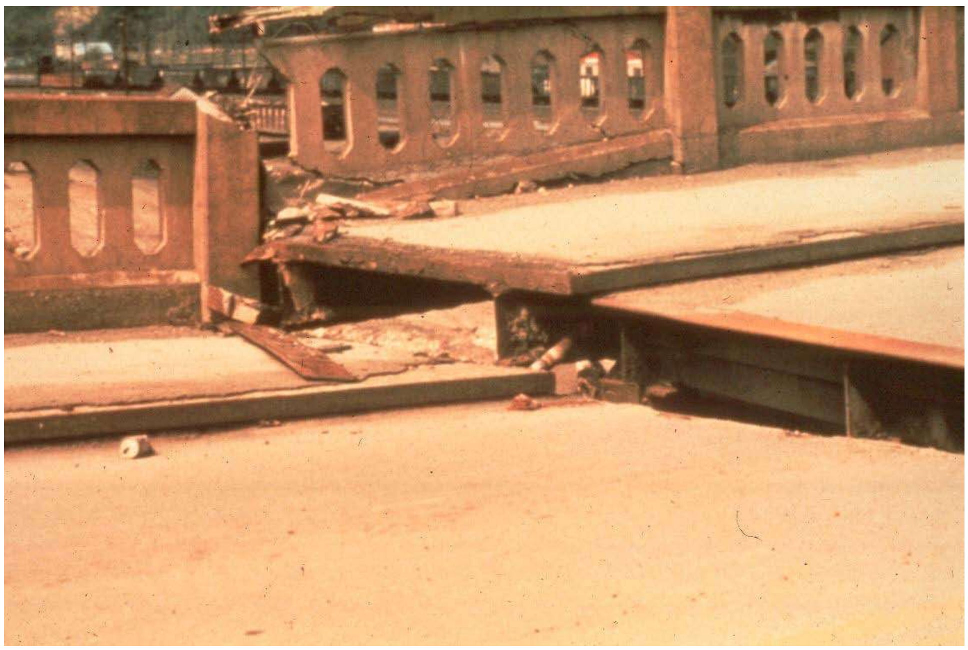

Figure 12.2.46 Superstructure Evidence of Pier Settlement

Inspection for lateral movement, or sliding, includes:

- Check the general alignment.
- Check the bearings for evidence of lateral displacement.
- Investigate the deck joints. The deck joint openings should be consistent with the recorded temperature.
- Inspect for cracking or spalling that may otherwise be unexplained; in the case of inspections after earthquakes, such damage is readily apparent (see Figure 12.2.47).
- Check for scour or undermining around the pier or bent footing (see Figures 12.2.48 and 12.2.49). Refer to Topic 13.2 for a more detailed description of scour and undermining. Refer to Topic 13.3 for a more detailed description of underwater inspection.

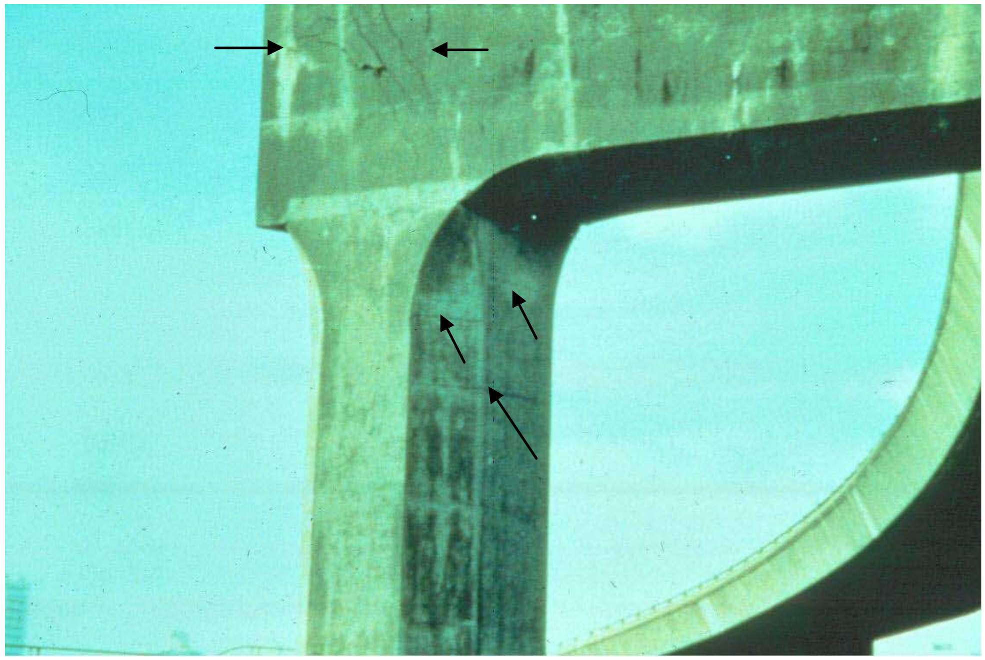

Figure 12.2.47 Cracks in Bent Cap due to Lateral Movement of Bent during Earthquake

Figure 12.2.48 Pier Movement and Superstructure Damage due to Scour/Undermining

Figure 12.2.49 Tipping of Bent due to Scour/Undermining

Inspection for rotational movement, or tipping, includes:

- Checking vertical alignment of the pier using a plumb bob or level.
- Investigating the clearance between the ends of the simply-supported beams at piers.
- Inspect for unusual cracking or spalling.

Bearing Areas

Differential settlement or excessive longitudinal or transverse forces, such as those experienced during an earthquake, may cause rotational movement (tipping) and lateral (horizontal) movement of piers or bents.

High bearing zones include the bridge seats, the pier cap, the pier shaft or bent column/footing connection, and the area where the footing is supported by earth or deep foundations. In timber piers or bents, look for crushing. Look for cracking or spalling in concrete and masonry members. Examine steel members for buckling or distortion.

Shear Zones

Vertical forces cause high shear zones in pier caps close to points of support. Horizontal forces cause high shear zones on the bottom of the pier shaft or bent column. In timber piers or bents, look for splitting. Look for diagonal cracks in concrete and masonry. Examine steel members for buckling or distortion.

Flexural Zones

Check the pier cap for signs of overstress in the positive and negative bending moment regions. High flexural moments caused by horizontal forces occur at the bottom of the pier shaft or bent column. High flexural moments may be occurring at the footing toe/pier shaft. Moments cause compression and tension depending on the load type and location of the member neutral axis. Look for deficiencies caused by overstress due to compression or tension caused by flexural moments. Check compression areas for splitting, crushing or buckling. Examine tension members for cracking or distortion.

Areas Exposed to Drainage

Water can leak through the deck joints. Examine areas below deck joints for signs of water leakage, and dirt and debris build-up. Look for material deficiencies caused by exposure to moisture, such as corrosion and section loss on steel, spalls and delaminations on concrete and decay on timber. Examine the piers and bents at the ground level or water level for similar deteriorations.

Areas Exposed to Traffic

Check for collision damage from vehicles passing adjacent to structural members.

Damage to concrete piers or bents may include spalls and exposed reinforcement and possibly steel reinforcement section loss. Steel piers or bents may experience cracks, section loss, or distortion which needs to be documented. Timber piers and bents may experience cracks, section loss, distortion or loose connections which need to be documented.

Areas Previously Repaired

Examine thoroughly any repairs that have been previously made. Determine if repaired areas are sound and functioning properly.

For concrete members, effective repairs and patching are usually limited to protection of exposed reinforcement (see Figure 2.1.50). For steel members, document the location and condition of any repair plates and their connections. For timber members, document the location and condition of repaired areas and their connections.

Figure 12.2.50 Repaired Concrete Column Bent

Scour and Undermining

Scour is the removal of material from a streambed as a result of the erosive action of running water. Scour can cause undermining or the removal of supporting foundation material from beneath the piers or bents when streams or rivers flow adjacent to them. Refer to Topic 13.2 for a more detailed description of scour and undermining.

Inspection for scour includes probing around the pier or bent footing for signs of undermining. Sometimes silt loosely fills in a scour hole and offers no protection or bearing capacity for the pier or bent footing.

Problematic Details and Fracture Critical Members

Steel piers or bents may contain problematic or fatigue prone details. Closely examine these details for section loss due to corrosion and cracking.

Steel piers or bents may be considered to be fracture critical (see Figure 12.2.51). See Topic 6.4 for a detailed description of details and fracture critical members.

Figure 12.2.51 Fracture Critical Steel Bent

Dolphins and Fenders

The condition of dolphins and fenders are checked in a manner similar to that used for inspecting the main substructure elements.

In concrete pier protection members, check for spalling and cracking of concrete or corrosion of the reinforcing steel (see Figure 12.2.52). Investigate for hourglass shaping of piles due to abrasion at the waterline, and check for structural damage caused by marine traffic.

In steel pier protection members, observe the splash zone (up to two feet above high tide or mean water level) carefully for corrosion. Where there are no tides, check the area from the mean water level to two feet above it. Examine steel members for corrosion, and check for structural damage (see Figure 12.2.53).

In timber pier protection members, observe the portions between the high waterline and the mud line for marine borers, caddisflies, and decay, and check for structural damage (see Figure 12.2.54). Check for hourglass shaping of piles at the waterline.

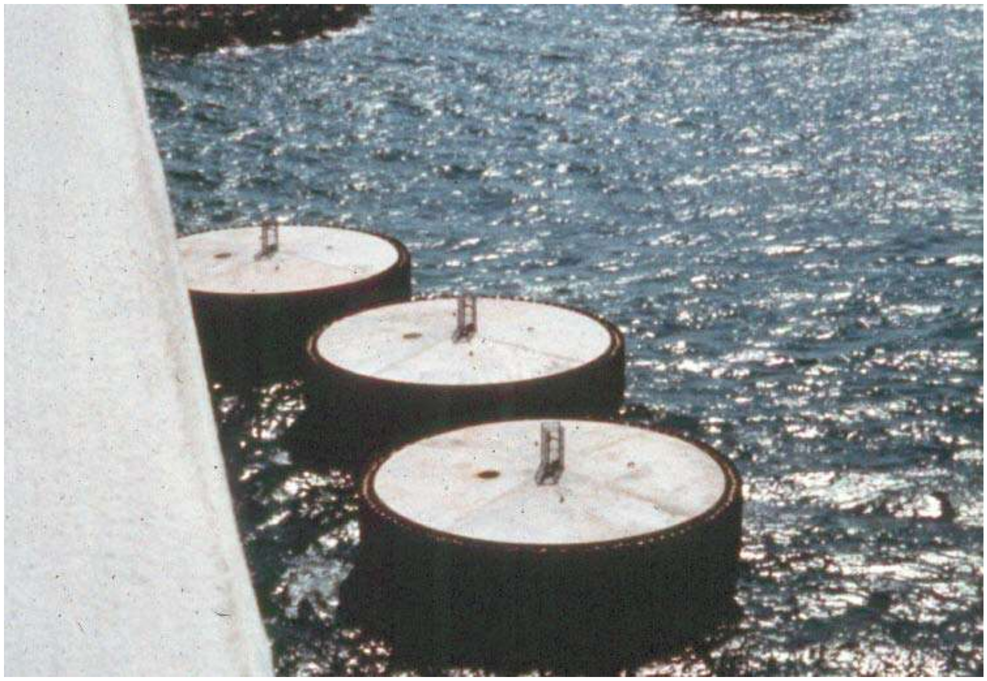

Figure 12.2.52 Concrete Dolphins

Figure 12.2.53 Steel Fender

Figure 12.2.54 Timber Fender System with Deteriorated Piles

12.2.4 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of substructures. 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 condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment method.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the entire substructure including abutments and piers. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 60) for additional details about NBI component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating. Recognize that piers may be affected by scour or other conditions that may only be able to be accessed and evaluated by a separate underwater inspection. Therefore, the results of both the routine and underwater inspection, if applicable, are integrated and evaluated together to arrive at the correct component condition rating for the substructure. Note the findings of the underwater inspection in the narrative portion of the routine inspection report as documentation and justification for the determined substructure component condition rating code.

Element Level Condition State Assessment

In an element level condition state assessment of a pier or bent structure, possible AASHTO Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Substructure	
202	Steel Column/Pile Extension
207	Steel Column Tower (Trestle)
225	Steel Submerged Pile
231	Steel Pier Cap
204	Prestressed Concrete Column/Pile Extension
226	Prestressed Concrete Submerged Pile
233	Prestressed Concrete Pier Cap
205	Reinforced Concrete Column/Pile Extension
210	Reinforced Concrete Pier Wall
220	Reinforced Concrete Pile Cap/Footing
227	Reinforced Concrete Submerged Pile
234	Reinforced Concrete Pier Cap
206	Timber Column/Pile Extension
208	Timber Column Tower (Trestle)
228	Timber Submerged Pile
212	Timber Pier Wall
235	Timber Pier Cap
213	Masonry Pier Wall
211	Other Pier Wall

BME No.	Description
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating
521	Concrete Protective Coating

The unit quantity for the pier cap elements is feet, measured horizontally across the pier cap. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for columns and piles is each, and the total quantity is placed in one of the available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coatings is square feet, with the total area distributed among the four available conditions states depending on the extent and severity of the deficiency. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of the piers and bents..

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
361	Scour
363	Steel Section Loss
364	Steel Out-of-Plane (Compression Members)
367	Substructure Traffic Impact (load capacity)

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

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Chapter 13
Inspection and Evaluation of Waterways

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Chapter 13
Inspection and Evaluation of Waterways

Topic 13.1 Waterway Elements

13.1.1 Introduction

Rivers are the most dynamic geomorphic system that engineers have to cope with in the design and maintenance of bridges. The geomorphic features of the river can change dramatically with time. During major floods, significant changes can occur in a short period of time. While rivers are dynamic or can move locations, bridges do not move locations.

There are several ways in which channels can change and thereby jeopardize the stability and safety of bridges. The channel bed can scour (degrade) so that bed elevations become lower, undermining the foundation of the piers and abutments. Deposition of sediment on the channel bed (aggradation) can reduce conveyance capacity through the bridge opening. Flood waters are then forced around the bridge, attacking roadway approaches, channel banks, and flood plains. Another consequence of aggradation is that the river stage may be increased to where it exerts lateral thrust and lift on the deck and girders of the bridge (see Figures 13.1.1 and 13.1.2). The other primary way in which bridges can be adversely affected by a waterway is through bank erosion or avulsion, causing the channel to shift laterally. These phenomena of aggradation, degradation or scour, bank erosion, and lateral migration can be a result of natural or induced causes and can adversely affect the bridge (see Figure 13.1.3). Topic 13.2 presents detailed descriptions of waterway deficiencies.

Of all the bridges in the National Bridge Inventory (NBI), approximately 86% are built over waterways. Bridge inspectors need to understand the relationship between the bridge and waterway elements. This understanding involves being able to recognize and identify the streambed, embankments, floodplain, and streamflow so that an accurate assessment and record of the present condition of the bridge and waterway can be determined.

Figure 13.1.1 Failure Due to High Water Levels During Hurricane: Aerial View

Figure 13.1.2 Failure Due to High Water Levels During Hurricane: Close-Up View

Figure 13.1.3 Pier Foundation Failure

13.1.2 Properties Affecting Waterways

Safety is a major concern in the inspection of bridges over active waterways. Various properties can affect waterways and structures.

- The size, shape and orientation of the bridge superstructure and foundation units.
- The physical characteristics such as channel sinuosity, slope, streambed and bank material classification and bank geometry and vegetative cover.
- The geomorphic history of the waterway (history of changes in the location, shape, and elevation of the channel).
- The hydraulic forces imposed on the bridge by the streamflow.
- Changes in the river channel or flow due to development projects (such as dams, diversions, urbanization and channel stabilization) or natural phenomena.
- The condition of hydraulic control structures that have been utilized to help protect the bridge and adjacent channel.
- Changes in the sediment balance in the stream due to nearby streambed gravel mining or landslides.

13.1.3 Purpose of Waterway Inspections

There are three major purposes for conducting waterway inspections.

- Identify critical damage
- Record existing channel conditions
- Monitor channel changes

Identify Critical Damage

Waterway inspections are needed to identify conditions that cause structural collapse of bridge structures. Deficient piling along with damage or deterioration to foundation members can only be detected during a waterway inspection. Entering the water and probing around the foundations is necessary to detect loss of foundation support.

Record Existing Channel Conditions

Waterway inspections are conducted to create a record of the existing channel conditions adjacent to the bridge. Conditions such as channel opening width, depth at substructure elements, channel cross-section elevations, water flow velocity, and channel constriction and skew are noted and compared to previously recorded conditions.

Accessing the waterway to measure and record channel conditions may be restricted by several factors including channel width and depth, flow velocity, or pollution. These factors may require the bridge inspector to return to the site during a period of low flow. Alternatively the inspector may need to consider using an alternate means of waterway access, such as a boat, or an alternative inspection technique, such as underwater diving inspection.

Monitor Channel Changes

Current waterway inspection data should be compared to previous inspection data in order to identify channel changes. This "tracking" of channel change over time is an important step in ensuring the safety of the bridge. Over time, vertical changes, due to either degradation or aggradation processes, or horizontal alignment changes, due to lateral migration of the channel, could result in foundation undermining, bridge overtopping, or even collapse of the structure. If major changes are found, a formal scour analysis of the site, involving a multi-disciplinary team of engineers, may be needed to estimate floodwater elevations, velocities, angle of attack, and potential scour depths. Potential threats to bridge members caused by channel changes can thus be dealt with before damage actually occurs. See Topic 13.2 for the inspection and evaluation of waterways.

13.1.4 Channel Characteristics

According to the Hydraulic Design Series Number 6 (HDS-6) Highways in the River Environment, channels are typically well-defined and confine the streamflow during normal flow conditions (see Figure 13.1.4).

Elements of a Channel

- Streambed - the bottom or floor of the channel.
- Streambank - the sloped sides of the channel, which extend from the streambed to the surrounding ground elevation (floodplain).
- Streamflow - the water, suspended sediment, and any debris moving through the channel.
- Thalweg elevation – lowest elevation of the stream.

Elements of floodplains are presented in Topic 13.1.5.

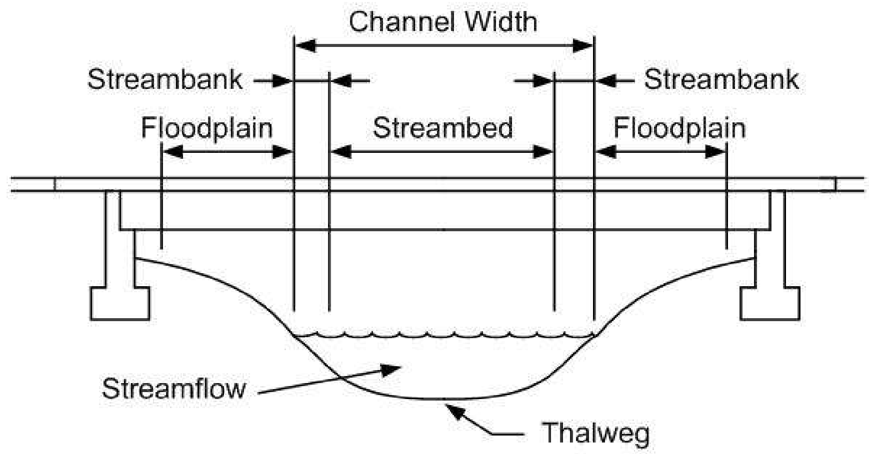

Figure 13.1.4 Typical Waterway Cross Section Showing Well Defined Channel Depression

Types of Channels

Knowledge of the type and profile of a waterway or river channel is essential to understand the hydraulics of the channel and its potential for change. The type of river may dictate certain tendencies or responses that may be more adverse than others. To aid in this understanding, various key river classes are briefly explained. Rivers can be broadly classified into four categories:

- Meandering rivers
- Braided rivers
- Straight rivers
- Steep mountain streams

Meandering Rivers

Meandering rivers consist of a series of bends connected by crossings. In general, pools exist in the bends. The dimensions of these pools vary with the size of the river, flow conditions, radius of the curvature of the bends, and type of bed and bank material. Such rivers are fairly predictable and experience relatively slow velocities. Figure 13.1.5 shows some differences between the various river categories. Figure 13.1.6 illustrates the major characteristics of a meandering river.

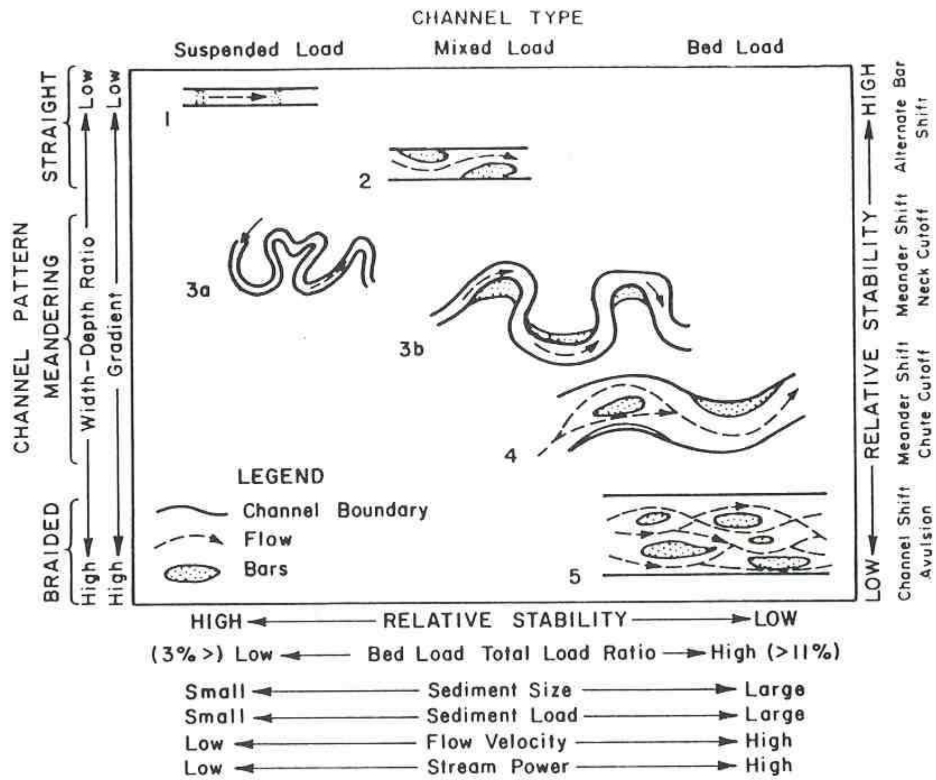

Figure 13.1.5 Plan View of Rivers

Braided Rivers

Braided rivers consist of multiple channels that are intertwined in braided form. At flood stages, the appearance of braiding is less noticeable. The bars dividing the multiple channels may become submerged, and the river will appear to be relatively straight. Braided rivers have steeper slopes and experience higher streamflow velocities which may cause larger scour or undermining problems.

Braided rivers can change rapidly, causing different velocity distributions, partial blockages of portions of the waterway beneath bridges, and larger quantities of debris that can be a hazard to bridges and cause accelerated scour. Figure 13.1.4 illustrates the plan view of typical rivers, including meandering, straight, and braided. This figure also relates form of river to channel type based on sediment load and relative stability of river type.

Straight Rivers

Straight rivers are something of an anomaly. Most straight rivers are in a transition between meandering and braided types. In straight rivers, any development that would flatten the gradient would accelerate change from a straight system to a meandering system. Conversely, if the gradient were increased, the channel may become braided. Therefore, in order to maintain the straight alignment over a normal range of hydrologic conditions, it may become necessary to utilize channel hydraulic control structures (Topic 13.1.7). The characteristics of straight rivers are identified in Figure 13.1.5.

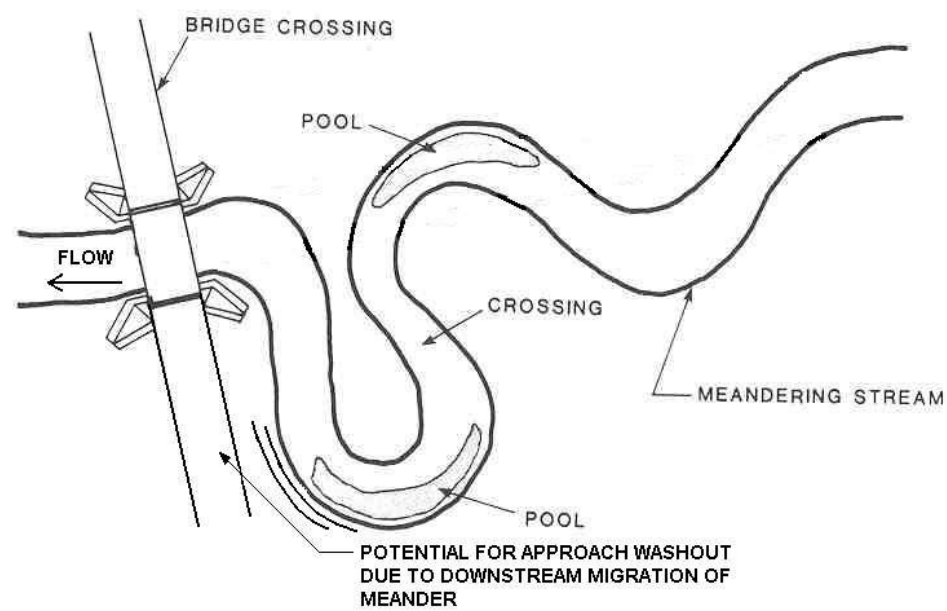

Figure 13.1.6 Meandering River

Steep Mountain Streams

Steep mountain streams are controlled by geologic formations, rock falls, and waterfalls. They experience very small changes in either plan form or profile when subjected to the normal range of discharges. The bed material of such river systems can consist of gravel, cobbles, boulders, or some mixture of these different sizes. Even though these rivers are relatively stable, they can experience significant velocity and flow changes during episodic flood events.

13.1.5 Floodplain Characteristics

The floodplain is the overbank area outside the channel that carries flood flows in excess of channel capacity (see Figure 13.1.7). It is common to find bridges built within the floodplain. For many structures, the floodplain is quite large, as compared to the channel. Observations made during periods of high water can help the inspector identify the floodplain.

Elements of a Floodplain

- Freeboard – the vertical distance between the design flood water surface and the lowest point of the superstructure to account for waves, surges, drift and other contingencies (see Figure 13.1.5)
- Normal stage – the streamflow stage prevailing during the greater part of the year (between low and high water levels)
- Waterway area – the entire are beneath the bridge which is available to pass flood flows.

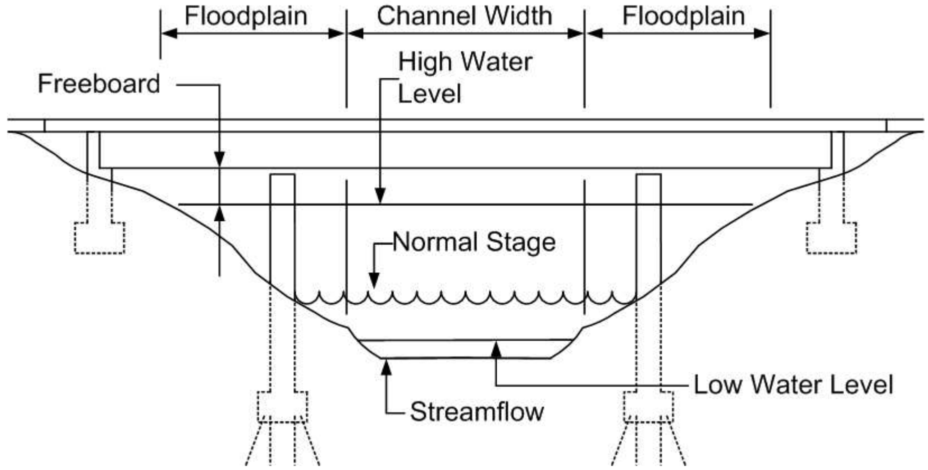

Figure 13.1.7 Typical Floodplain

Channel characteristics are presented in detail in Topic 13.1.4.

13.1.6 Hydraulic Opening

The hydraulic opening is the entire area beneath the bridge which is available to pass flood flows (see Figure 13.1.8). The bottom of the superstructure, the two bridge abutments, and the streambed or ground elevation bounds the hydraulic, or waterway, opening. For multiple spans, intermediate supports such as piers or bents restrict the hydraulic or bridge waterway opening.

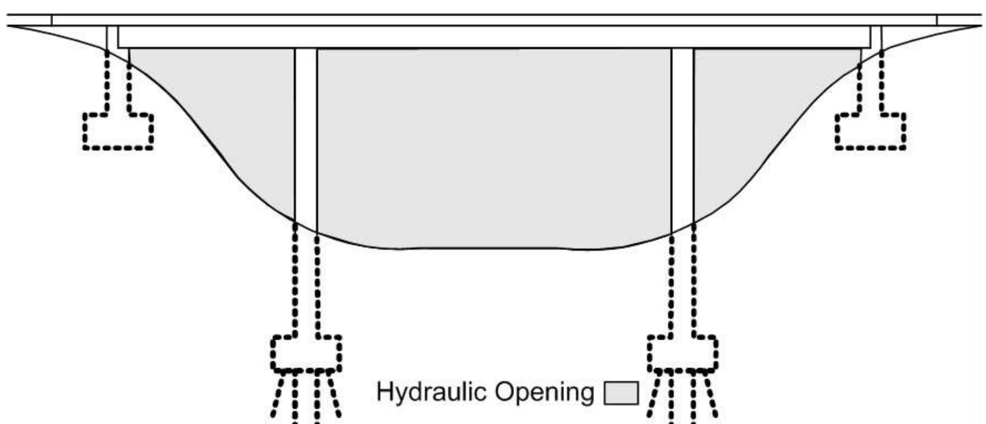

Figure 13.1.8 Hydraulic Waterway Opening

13.1.7 Hydraulic Countermeasures

Hydraulic countermeasures are often utilized to provide protection for bridges against lateral migration of the channel and against high velocity flows and scour. A hydraulic countermeasure is a man-made or man-placed device designed to direct streamflow and protect against lateral migration or scour. These flow hydraulic control countermeasures may be utilized either at the bridge, upstream from the bridge, or downstream from the bridge. Countermeasures are designed by hydraulic and geotechnical engineers and are installed to redirect streamflow and flood flows within the watercourse and through the bridge waterway opening. Hydraulic countermeasures are broken into two distinct categories which are river training structures and armoring countermeasures.

River Control Structures

River control structures are countermeasures designed to modify the flow to help prevent. A couple examples of river training structures are spurs and guide banks. A complete list of the various types of river training structures is located in HEC-23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, 3rd edition.

Spurs

Spurs are linear structures, designed with properly sized and placed rocks, that projects into a channel and placed on the outside bends of the bank to protect the streambank by reducing flow velocity, inducing deposition of sediment or redirecting the flow (see Figure 13.1.10). Common applications occur on meandering streams where they are placed on the outside of the bends to redirect the flow and minimize lateral stream migration.

Guide banks

Guide banks are dikes which extend upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening (see Figure 13.1.11). Scour hole formation occurs at the upstream ends of the guide banks if left unprotected. Common scour prevention devices for guide banks include riprap.

Armoring Countermeasures

Armoring countermeasures tend not to alter the flow significantly, but are design to resist hydraulic stresses of the design flood events. Some examples of armor countermeasures include riprap, gabions, slope stabilization, channel linings and footing aprons. A complete list of the various types of armoring countermeasures is located in HEC-23.

Riprap

Layers or facings of properly sized and graded rock or broken concrete, placed or dumped to protect an abutment, pier or embankment from erosion (see Figure 13.1.9). Riprap has also been used to almost all kinds of armor which include wire-enclosed riprap partially grouted riprap, sacked concrete and concrete slabs. Riprap should be protected against subsurface erosion by filters formed either of properly graded sand/gravel or of synthetic fabrics developed and utilized to replace the natural sand/gravel filter system. It must be placed on an adequately flat slope to be able to resist the anticipated forces of the flowing flood waters. Proper design and placement of riprap is essential. This generally requires placement of the riprap on side-slopes no steeper than 1.5 to 1 vertical (1.5H:1V). Flatter side-slopes of such as 2H:1V to 3H:1V are preferable. Proper design and placement of riprap is essential. Inappropriate installations can aggravate or cause the conditions they were intended to correct or prevent.

Gabions

Rectangular rock- or cobble- filled wire mesh baskets or compartmented rectangular containers, anchored together and generally anchored to the surface they are protecting (see Figure 13.1.12). Gabions may be placed on steeper slopes than riprap or may even be stacked vertically, depending upon the design procedure and site conditions.

Slope Stabilization Methods

Slope stabilization methods consist of the placement of geotextiles, wire mesh, riprap, paving, revetment, plantings or other materials on channel embankments, intended to protect the slope from erosion, slipping or caving or to withstand external hydraulic pressure (see Figure 13.1.13). It is anticipated the various stabilization methods will fill-in with sediment and help sustain plant growth. The roots from the plants contribute to stabilize the embankment or flood plain.

Channel Lining

Channel lining is a concrete pavement that extends across the streambed. Channel linings also may be revetment mats or some other form of bed armoring. A typical revetment mat is formed by interlocking precast concrete blocks linked by cable (polyester or steel) placed on a geotextile fabric. The interlocking matrix allows for use over varying land contours and grades (see Figure 13.1.14). Channel linings may also consist of formed concrete. This type is less flexible and versatile than revetment mats and other bed armoring (see Figure 13.1.15).

Footing Aprons

Footing aprons are protective layers of material surrounding the footing of a substructure unit. Footing aprons usually consist of cast-in-place concrete (see Figure 13.1.16 and 13.1.17). Footing aprons protect footings from undermining. The aprons are not a structural element of the abutment or pier footings and are considered a structural countermeasure instead of a hydraulic countermeasure.

Figure 13.1.9 Crushed Stone Riprap

Figure 13.1.10 Spurs

Figure 13.1.11 Guide banks Constructed on Kickapoo Creek Near Peoria, Illinois