Publication No. FHWA NHI 03-002
October, 2002

U.S. Department
of Transportation

**Federal Highway
Administration**

Bridge Inspector's Reference Manual



BIRM

Volume 2

NATIONAL HIGHWAY INSTITUTE
Training Solutions for Transportation Excellence

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Section 8
Inspection and Evaluation of Common Steel Superstructures

Topic 8.1 Fatigue and Fracture in Steel Bridges

8.1.1 Introduction

Over the past 40 years, several hundred steel bridges have developed cracks, mostly due to fatigue stress. Although these localized failures have been extensive, only a few U.S. bridges have actually collapsed as a result of steel fractures.

The first was the Silver Bridge over the Ohio River at Point Pleasant, West Virginia. This structure was an eyebar chain suspension bridge with a 700-foot (213 m) main span that collapsed without warning on December 15, 1967 (see Figure 8.1.1).



Figure 8.1.1 Silver Bridge Collapse

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



Figure 8.1.2 Mianus River Bridge Failure

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

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

Fracture Critical Member

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

Fatigue

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

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

Reviewing Member Forces

For a bridge member to be classified as fracture critical, it must meet two criterion. The first criterion deals with the forces in the member. Members that are in tension or members that have fibers or elements that are in tension meet the first

criterion. The five types of member forces, previously covered in Section P.1.5, include:

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

Redundancy

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

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

There are three basic types of redundancy in bridge design:

- Load path redundancy
- Structural redundancy
- Internal redundancy

Load Path Redundancy

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

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



Figure 8.1.3 Load Path Redundant Multi-Girder Bridge

Structural Redundancy

Bridge designs which provide continuity of load path from span to span are referred to as structurally redundant. Some continuous span two-girder bridge designs are structurally redundant (see Figure 8.1.4). In the event of a member failure, loading from that span can be redistributed to the adjacent spans, and bridge failure may not occur.



Figure 8.1.4 Structurally Redundant Continuous Span Bridge

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

Continuous spans are structurally redundant except for the end spans, where the development of a fracture would effectively cause two hinges, one at the abutment and one at the fracture itself. This situation would be unstable. In the interest of conservatism, AASHTO chooses to neglect structural redundancy and classify all two girder bridges as nonredundant.

Internal Redundancy

Internal or member redundancy exists when a bridge member contains three or more elements that are mechanically fastened together so that multiple independent load paths are formed. Failure of one member element would not cause total failure of the member. An example of an internally redundant member is a riveted plate girder (see Figures 8.1.5 and 8.1.6).

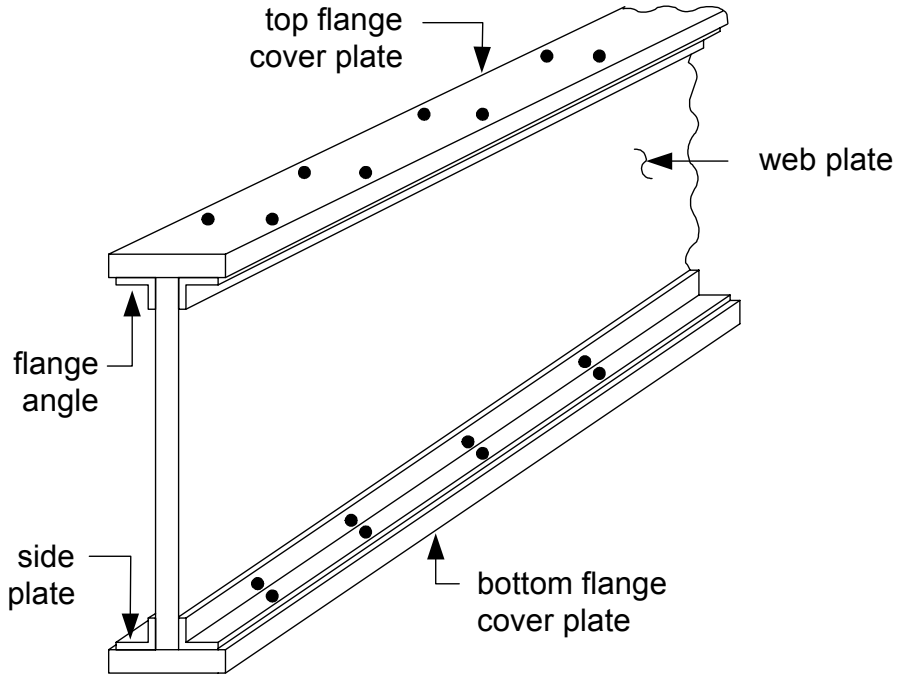


Figure 8.1.5 Internally Redundant Riveted I-Beam

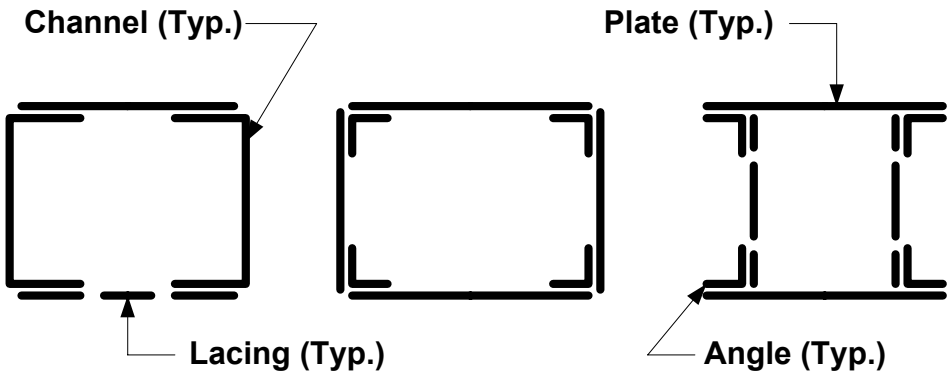


Figure 8.1.6 Internally Redundant Riveted Box Shapes

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



Figure 8.1.7 Patch Plate on Girder Web along Flange Angle

Nonredundant Configuration

Bridge designs that are nonredundant have two or fewer main load-carrying members or load paths. Bridge members that are nonredundant have two or fewer elements. Nonredundant bridge configurations almost always contain fracture critical members.

Bridge inspectors are concerned primarily with load path redundancy. The inspector should neglect structural and internal redundancy and classify all two-girder bridges as nonredundant (see Figure 8.1.8).



Figure 8.1.8 Non-Redundant Two-Girder

8.1.2 Failure Mechanics

As stated earlier, failure mechanics involves describing the process by which a member fails when subjected to fatigue.

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

- Crack initiation
- Crack propagation
- Fracture

Crack Initiation

Cracks most commonly initiate from points of stress concentrations in structural details. Stress concentration can result from flaws, fatigue-prone structure details, or out-of-plane distortions. The most critical conditions for crack initiation at structural details are those combining:

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

Crack Propagation

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

Fracture

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

Fatigue Life

The number of cycles required to initiate a fatigue crack is the fatigue-crack-initiation life. The number of cycles required to propagate a fatigue crack to a

critical size is called the fatigue-crack-propagation life. The total fatigue life is the sum of the initiation and propagation lives.

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

Types of Fractures

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

- Brittle Fracture - Occurs with no warning (i.e., without prior plastic deformation and at average stresses below those of general yielding) (see Figure 8.1.9).

- Ductile Fracture - Generally preceded by local plastic deformation of the net uncracked section. This plastic deformation results in distortion of the member, providing some visual warning of the impending failure (see Figure 8.1.10).



Figure 8.1.9 Brittle Fracture of Cast Iron Specimen



Figure 8.1.10 Ductile Fracture of Cold Rolled Steel

Factors that Determine Fracture Behavior

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

- Ambient temperature – Each heat of steel has a transition temperature below which it becomes brittle

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

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

The risk of a brittle fracture is greatly increased when the fracture behavior factors include:

- Cold temperature
- Rapid loading
- High constraint

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

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

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

Fracture Toughness

The fracture toughness is a measure of the material's resistance to crack extension. Fracture toughness can be defined as the ability to carry load and to absorb energy in the presence of a crack. Like the previously discussed brittle-to-ductile failure transition, the fracture toughness for a given steel type varies with:

- Temperature
- Loading rate
- Degree of constraint

In general, a steel plate may experience a brittle failure when:

- The temperature is low
- The structure is rapidly loaded
- The member is thick

Conversely, the fracture becomes increasingly ductile when:

- The temperature is higher
- The loading rate is slower
- The member is thinner

Trucks crossing a bridge stress the steel at rates intermediate between those of slow and rapid loading. Therefore, with actual bridges in service, the toughness of the steel is primarily a function of steel type, temperature, and constraint. In general, thick welded members made of steel with low toughness are more likely to fracture on cold days.

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

8.1.3 Factors Affecting Fatigue Crack Initiation

Most critical conditions for fatigue crack initiation are those which involve a combination of flaws and stress concentrations. Stress concentrations have resulted from out-of-plane distortion of elements of bridge members, and cracks have occurred due to such distortion. Good detailing can reduce the number and severity of these stress concentrations, but the need to connect girders, stringers, floorbeams, diaphragms, bracing, truss members, hangers, and other members makes it impossible to completely avoid them. Bridge structures, particularly those that are welded, cannot be fabricated without details that cause some level of stress concentrations.

Welds

Conditions of stress concentration are often found in weldments, which are known to be prone to crack initiation. Welds are the connections of metal parts formed by heating the surfaces to a plastic (or fluid) state and allowing the parts to flow

together and join with or without the addition of filler metal. The term base metal refers to the metal parts that are to be joined. Filler metal, or weld metal, is the additional molten metal generally used in the formation of welds. The complete assembly is referred to as a weldment.

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

- Groove Welds – Groove welds, which are sometimes referred to as butt welds, are used when the members to be connected are lined up edge to edge or are in the same plane (see Figure 8.1.11). Full penetration groove welds extend through the entire thickness of the piece being joined, while partial penetration groove welds do not. Weld reinforcement is the added filler metal that causes the throat dimension to be greater than the thickness of the base metal. This reinforcement is sometimes ground flush with the base metal to qualify the joint for a greater fatigue strength category (see Section 8.1.5 for descriptions of AASHTO Fatigue Categories).

- Fillet welds – Fillet welds connect members that either overlap each other or are joined edge to face of plate, as in plate girder assembly of web and flange plates (see Figure 8.1.12). Fillet welds are the most common type of weld because large tolerances in fabrication are allowable when members are lapped over each other instead of fitted together as in groove welds.

- Plug welds - Plug and slot welds pass through holes in one member to another, with weld metal filling the holes and joining the members together (see Figure 8.1.13). Plug welds have sometimes been used to fill misplaced holes. These repairs are very likely to contain flaws and microcracks that can result in the initiation of fatigue cracking. Plug welds are no longer permitted by AASHTO for bridge construction.

- Tack welds - Tack welds are small welds commonly used to temporarily hold pieces in position during fabrication or construction(see Figure 8.1.14). They are often made carelessly, without proper procedures or preheating, and can be a fatigue-prone detail.

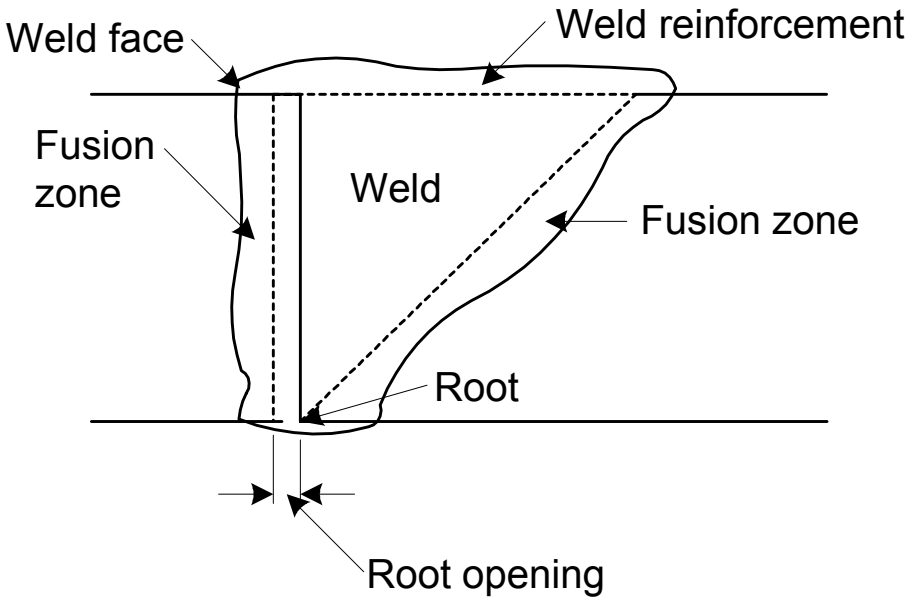


Figure 8.1.11 Groove Weld Nomenclature

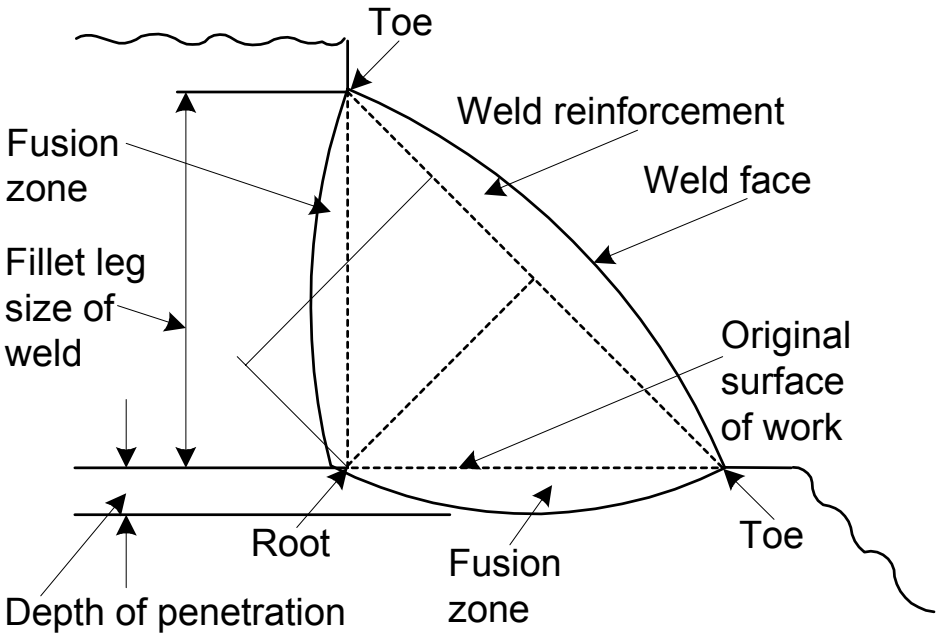


Figure 8.1.12 Fillet Weld Nomenclature

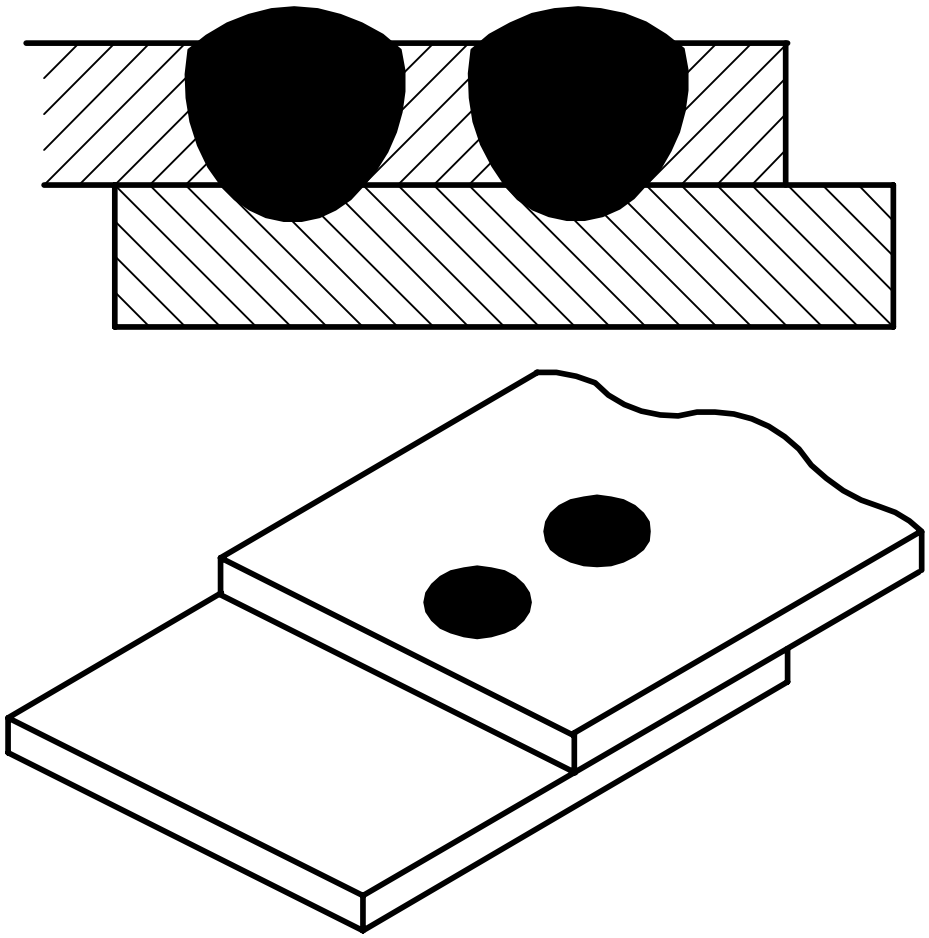


Figure 8.1.13 Plug Weld Schematic



Figure 8.1.14 Tack Weld

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

The joint geometry is also used to describe the weld. Some common weld joints include (see Figure 8.1.15):

- Butt
- Lap
- Tee
- Edge
- Corner

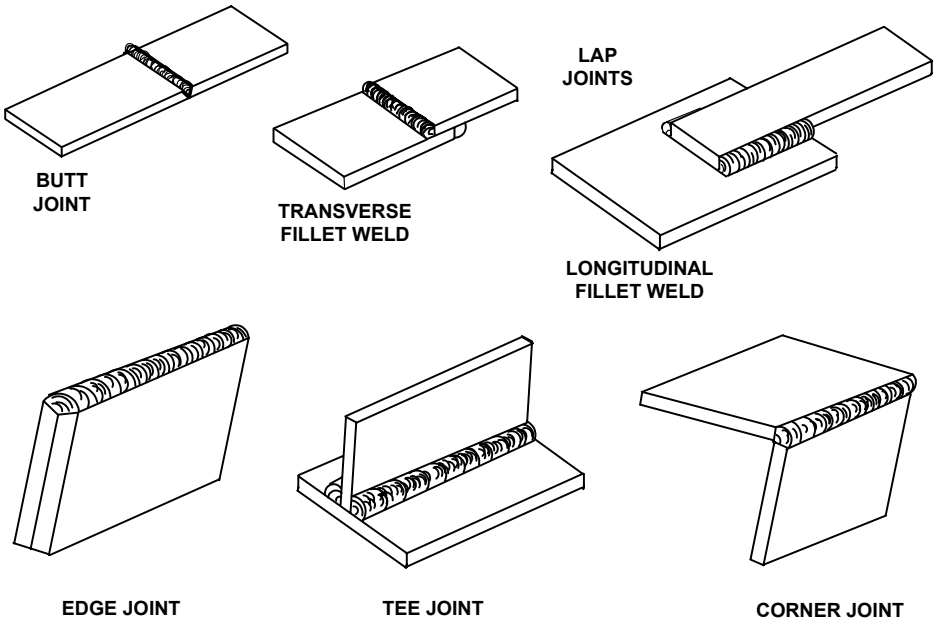


Figure 8.1.15 Types of Welded Joints

All welding processes result in high built-in residual tension stresses, which are at or near the yield point in the weldment and in the base metal adjacent to it.

Load-induced stress concentrations also often occur at welded bridge connections, where these residual tensile stresses are high. This combination of stress concentration and high residual tensile stress is conducive to fatigue crack initiation.

Such cracks typically begin either at the weld periphery, such as at the toe of a fillet weld, where there typically can be sharp discontinuities, or else at an internal discontinuity such as a slag inclusion or porosity.

In the initial stages of fatigue crack growth, much of the fatigue life is expended by the time a crack has propagated out of the high residual tensile stress zone.

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

- Quality of material
- Fabrication methods
- Erection techniques
- In-service conditions

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

Material Flaws

Material flaws may exist in different forms:

- External flaws (e.g., surface laps)
- Internal flaws (e.g., nonmetallic inclusions, laminations and "rolled-in" plate defects (see Figures 8.1.16 & 8.1.17)).



Figure 8.1.16 Centerline Crack in Steel Slab

The centerline crack in Figure 8.1.16 may have resulted from a shrinkage cavity like that shown in Figure 8.1.17 which was not forged and melded completely in the hot rolling process.

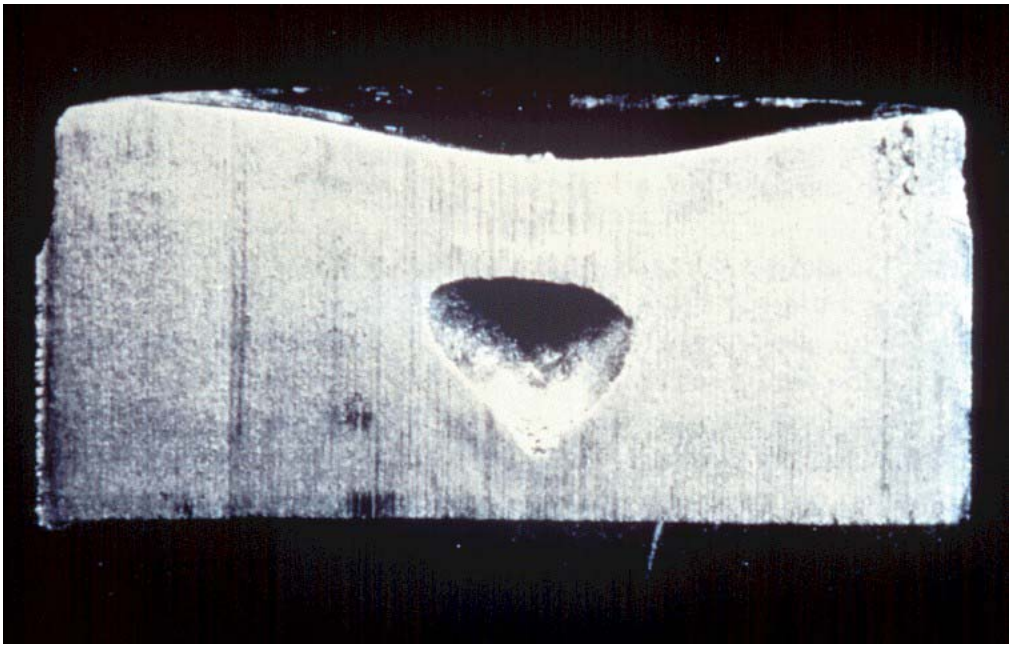


Figure 8.1.17 Shrinkage Cavity in Steel Billet

Fabrication Flaws

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

- Incomplete Penetration – Incomplete penetration occurs when the weld metal fails to penetrate the root of a joint or fails to fuse completely with the root face of the base metal (see Figure 8.1.18). Incomplete penetration is not permitted in any degree. Incomplete penetration welds cause a local stress riser at the root of a weld and can reduce the load carrying capacity of the member. A stress riser is a detail that causes stress concentration.

- Lack of fusion – Lack of fusion is a condition in which boundaries of unfused metal exist either between base metal and weld metal or between adjacent layers of weld metal (see Figure 8.1.19). Lack of fusion is generally a result of poor welding techniques, can seriously reduce the load carrying capacity of the member, and could be a point of crack initiation at a lower stress.

- Slag inclusions – Slag inclusion occurs when nonmetallic matter is inadvertently trapped between the weld metal and the base metal (see Figure 8.1.20). Slag from the welding rod shield may be forced into the weld metal by the arc during the welding operation. If large, irregular inclusions or lengthy lines of inclusions are present, crack initiation at a lower stress could begin and the strength of the weld may be considerably reduced. However, small isolated globe shaped inclusions do not seriously affect the strength of a weld, but can be a point of crack initiation.

- Porosity – Porosity is the presence of cavities in the weld metal caused by entrapped gas and takes the form of small spherical cavities, either scattered through out the weld or clustered in local regions (see Figure

8.1.21). It is tolerated if the amount does not exceed specified quantities relative to weld size.

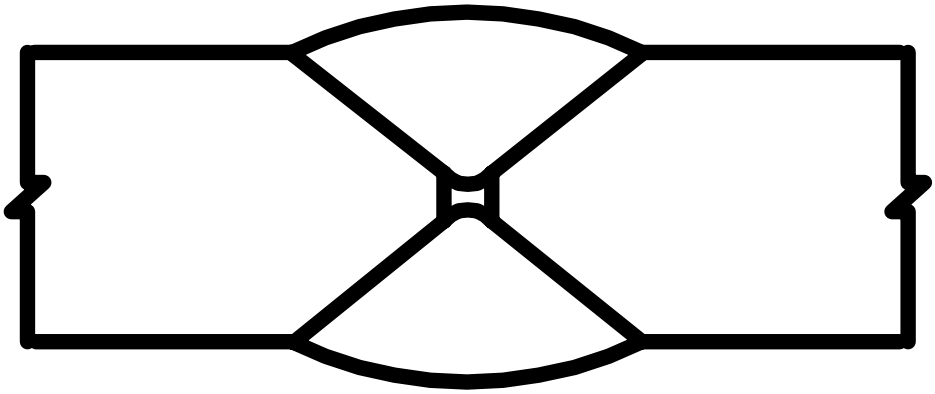


Figure 8.1.18 Incomplete Penetration of a Double V-Groove Weld

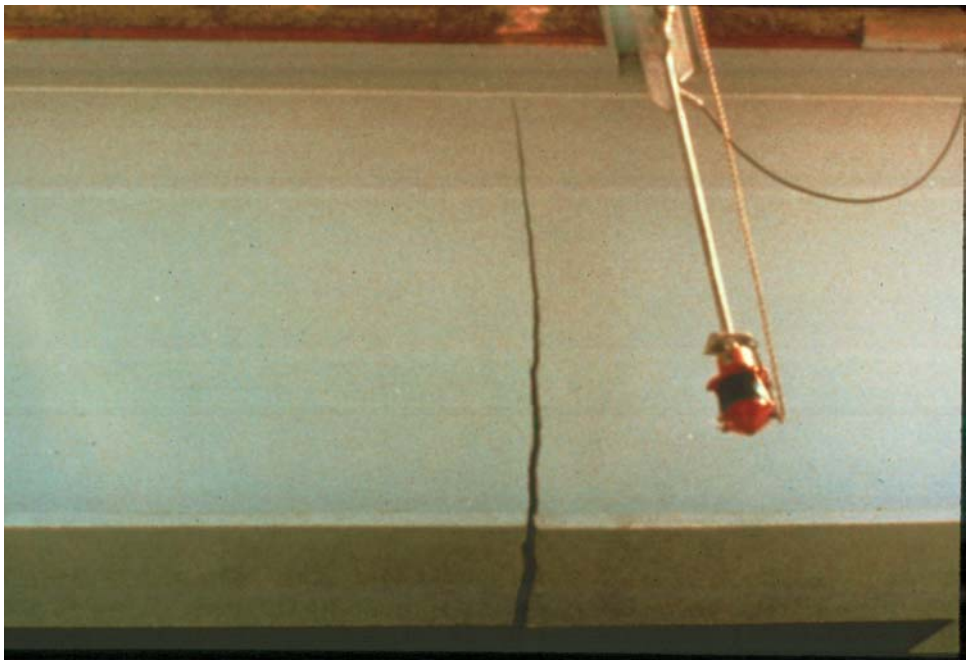


Figure 8.1.19 Crack Initiation from Lack of Fusion in Heat Affected Zone of Electroslag Groove Weld of a Butt Joint

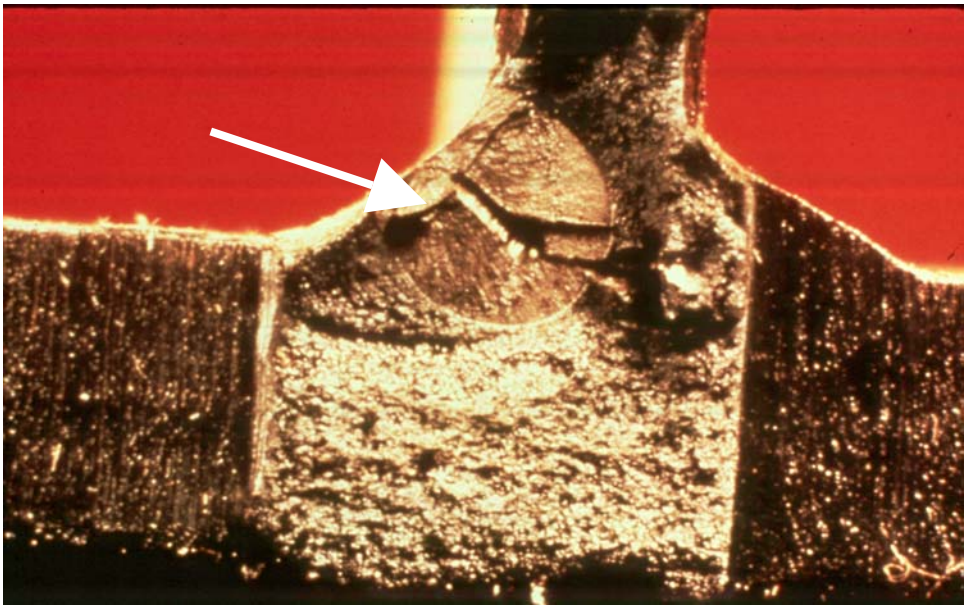


Figure 8.1.20 Web to Flange Crack due to Fillet Weld Slag Inclusion

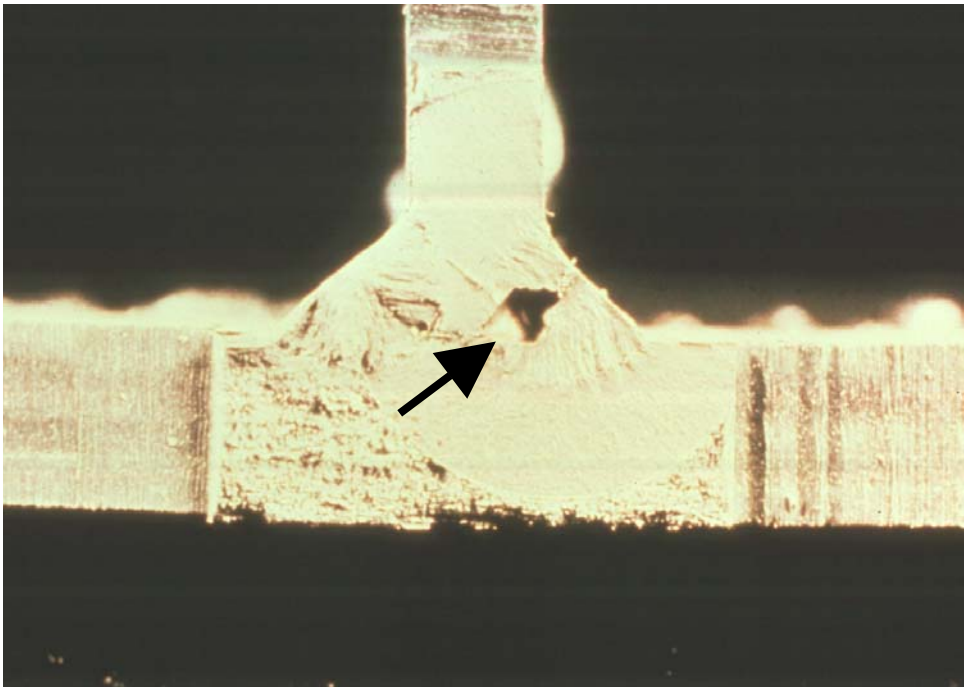


Figure 8.1.21 Crack Initiation from Porosity in Longitudinal Web-to-Flange Fillet Weld of Plate Girder

Plug welds are sometimes found in bridge members. In most cases, they were made to fill mislocated bolt holes. Such welds are highly restrained and often will contain incomplete penetration, lack of fusion, slag inclusions, and porosity. There have been many instances where a crack and fracture have occurred because of a plug weld (see Figure 8.1.22).



Figure 8.1.22 Crack Resulting from Previously Plug Welded Hole (Now Bolted)

Visible weld flaws include:

- Undercut – The condition in which a local reduction in a section of base metal occurs alongside the weld deposit. This may happen either on the surface of the base metal at the toe of the weld, or in the fusion face of multiple pass welds due to overheating. This groove creates a mechanical notch, which is a stress riser (see Figure 8.1.23). When an undercut is controlled within the limits of specifications and does not constitute a sharp or deep notch, it is not seen as a serious defect.

- Overlap – Overlap is a weld flaw at the toe of a weld in which the weld metal overflows onto the surface of the base metal without fusing to it due to insufficient heat (see Figure 8.1.24). This condition may exist intermittently or continuously along the weld joint. Discontinuity at the toe of a weld acts as a stress riser and reduces the fatigue strength of the member.

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

- Beam Coping – When flange/web copings do not have the proper radius as per AASHTO specifications a stress riser is created (see Figure 8.1.25).

- Flame Cuts – Flame cutting, although fast, creates large surface discontinuities that are stress risers (see Figure 8.1.26). The surface of flame cut plates in tension must be ground smooth in the direction of the tensile stress.

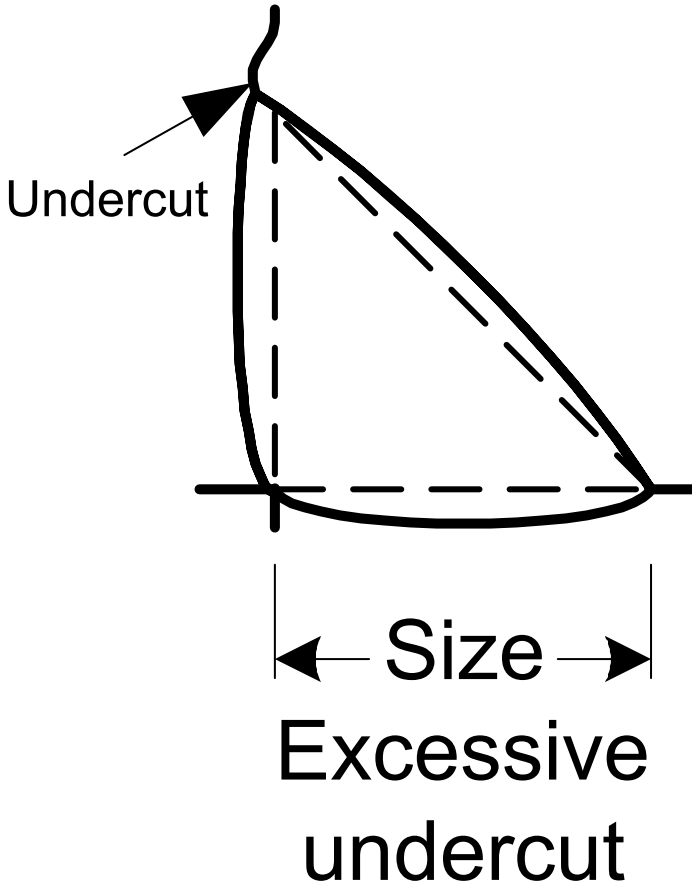


Figure 8.1.23 Undercut of a Fillet Weld

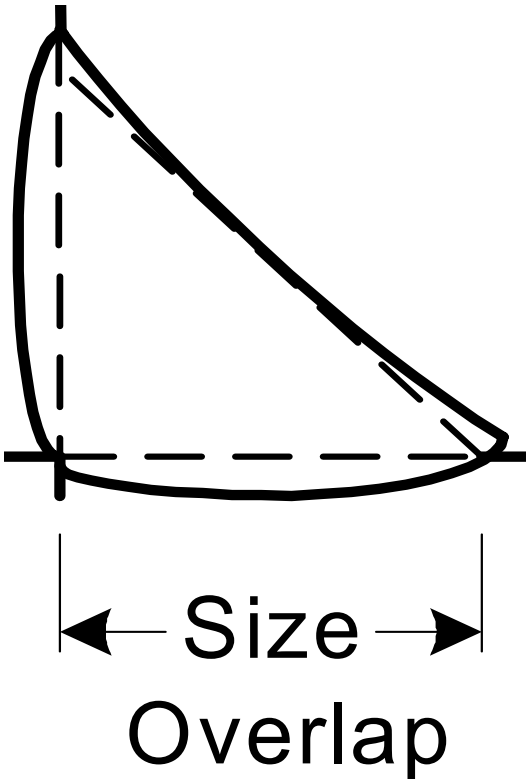


Figure 8.1.24 Overlap of a Fillet Weld

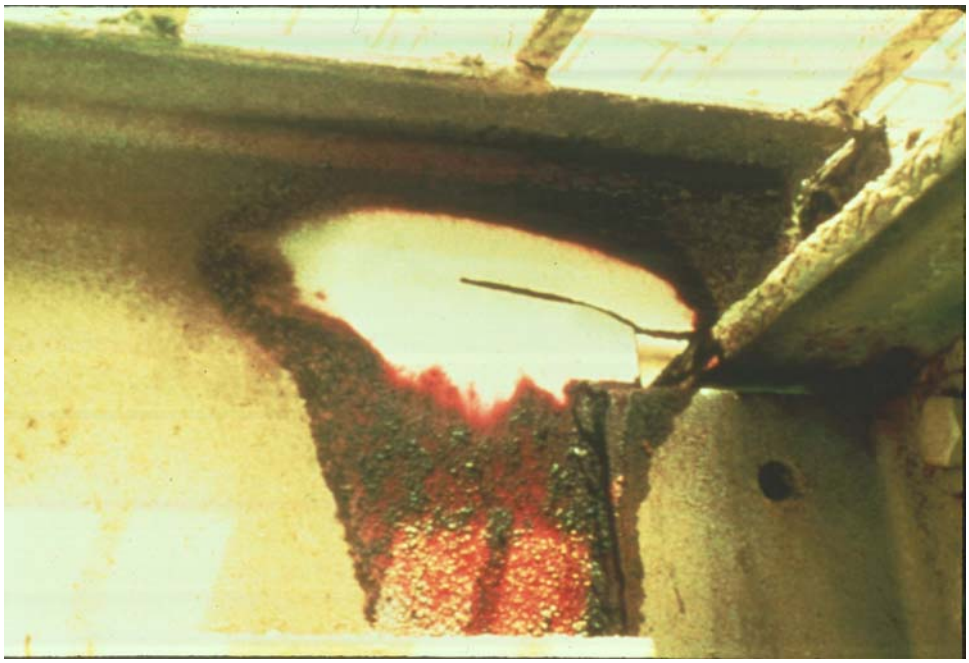


Figure 8.1.25 Crack Initiation at Coped Web in Stringer to Floorbeam

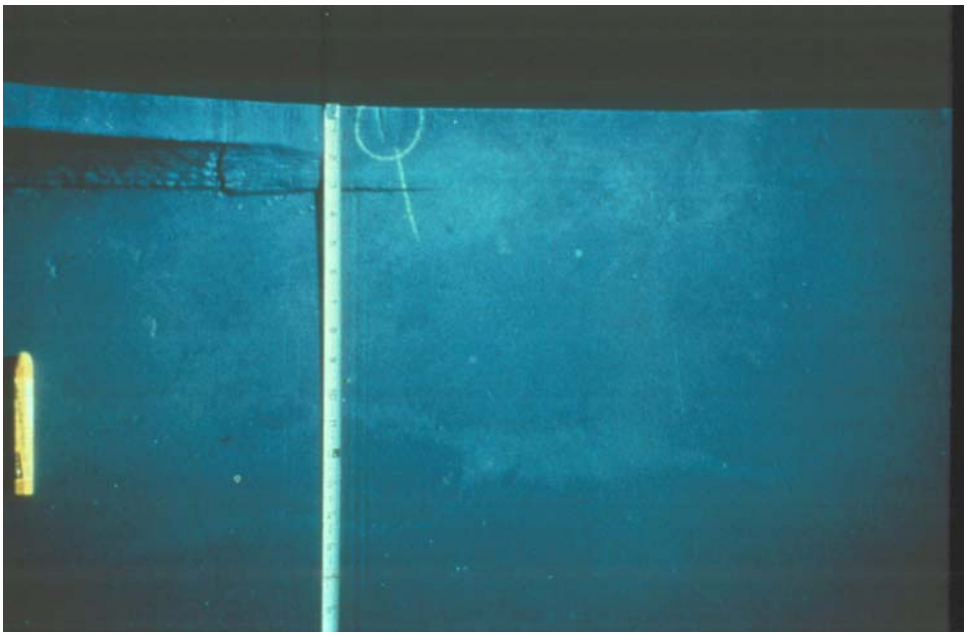


Figure 8.1.26 Insufficiently Ground Flame Cut of Gusset Plate for Arch to Tie Girder Connection

Transportation and Erection Flaws

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

- Nicks, Notches, and Indentations – Beam handling devices such as lifting tongs develop intense pressure at the point of contact and can cause measurable indentations and gouges.
- Chain marks – When transporting steel beams, chains are commonly used to secure the beam to the truck.
- Out-of-Plane Bending Forces – Beams must be securely blocked to resist cyclic sidesway movement during truck or rail transport. There have been extreme cases where cracks have initiated in beams before they have been erected.
- Tack Welds – Tack welds were very common in the mid 1950's and through the early 1960's. When left in place and exposed to tensile stress, they are a potential crack initiation location.

In-Service Flaws

Once the structure is placed in service, some members may be prone to collision damage by errant vehicles which may nick, tear, and excessively strain the steel (see Figure 8.1.27). Improper heat straightening may damage the steel. Deep corrosion pits can develop in structures that are improperly detailed for corrosion control, poorly maintained, or left unpainted. Also, any indiscriminate welding of conduit supports, lighting attachments, and ladder brackets to steel members can cause areas of high tensile stresses.



Figure 8.1.27 Severe Collision Damage on a Fascia Girder

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

8.1.4 Factors Affecting Fatigue Crack Propagation

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

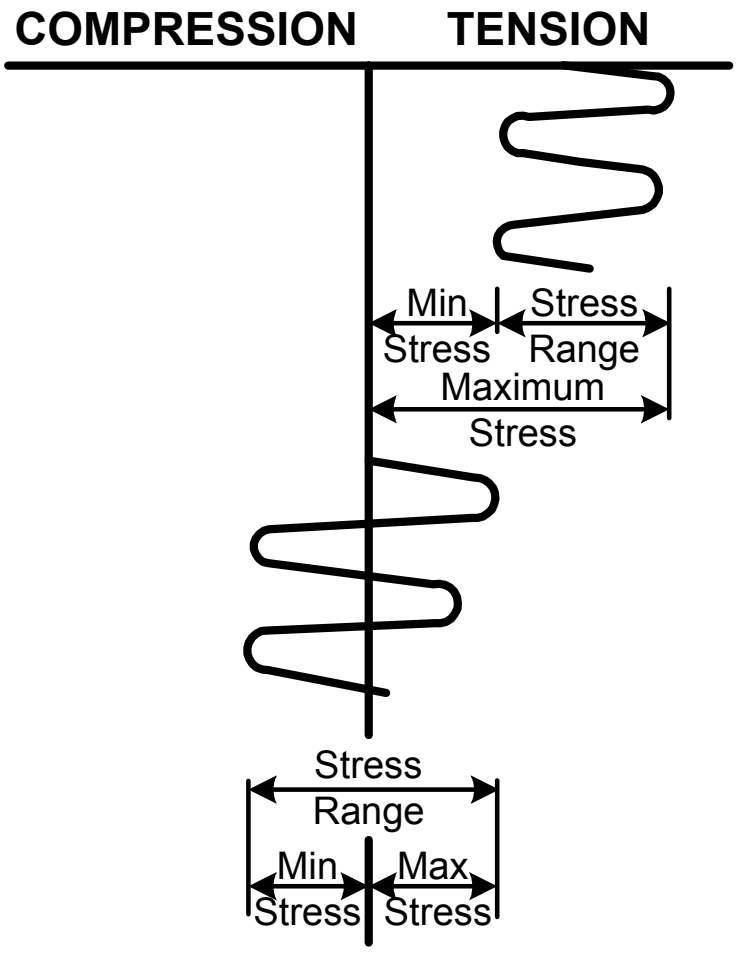
- Stress range
- Number of cycles
- Type of details

Stress Range

The stress range is defined as the algebraic difference between the maximum stress and the minimum stress calculated at the detail under consideration. In other words, it is the value of the cyclic stress caused by a truck crossing the bridge (see Figure 8.1.28). The weight of the bridge produces a constant stress instead of a cyclic stress. Therefore, it does not affect the crack propagation life. Only stress ranges in tension or stress reversal can drive fatigue cracks to failure. Stress ranges in compression may cause cracks to grow to some extent at weldments

where there are high residual tensile stresses. However, these "compression" cracks eventually arrest, and they do not induce fracture of the member.

Figure 8.1.28 Applied Tensile and Compressive Stress Cycles



Number of Cycles

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

Types of Details

The term "type of detail" refers to the stress condition in a member or connection (see Figure 8.1.29). Examples of types of details include:

- Flange cover plates
- Transverse stiffeners
- Flange groove welds
- Bolted joints
- Longitudinal stiffeners
- Connection plates (vertical or horizontal) attached to webs or flanges

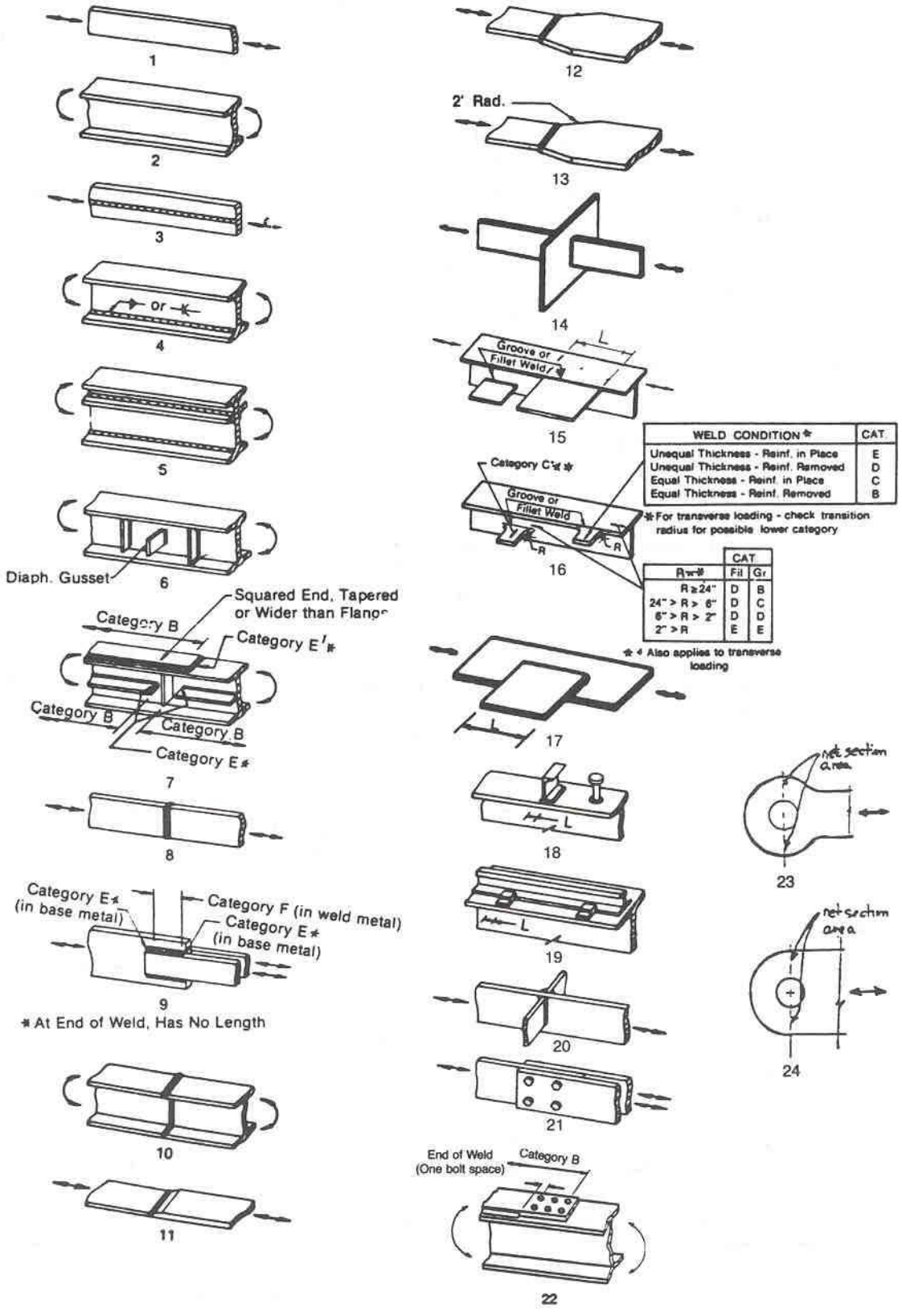


Figure 8.1.29 AASHTO Table 10.3.1C - Illustrative Examples of Bridge Details

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

Flange Crack Failure Process

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

One or more cracks can initiate from microscopic flaws or defects at the weld toe of the transverse end weld (see Figure 8.1.30). Such cracking may then advance in two stages:



Figure 8.1.30 Part-Through Crack at a Cover Plated Flange

Stage 1

In the first stage, a part-through surface crack is only barely visible as a hairline on the bottom of the flange at the toe of weld.

As stress is applied, the small cracks that have initiated join each other and begin to form a larger part-through surface crack (see Figure 8.1.31).



Figure 8.1.31 Part-Through Crack Growth at Cover Plate Welded to Flange

The crack front develops a thumbnail or half penny shape as it propagates in the thickness direction of the flange until reaching the inside surface.

Once it breaks through the thickness of the flange, the shape rapidly changes into that of a three-ended crack.

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

Stage 2

During the second stage, the crack then propagates with two fronts moving across the flange width and one front moving into the web until the member fractures (see Figure 8.1.32).

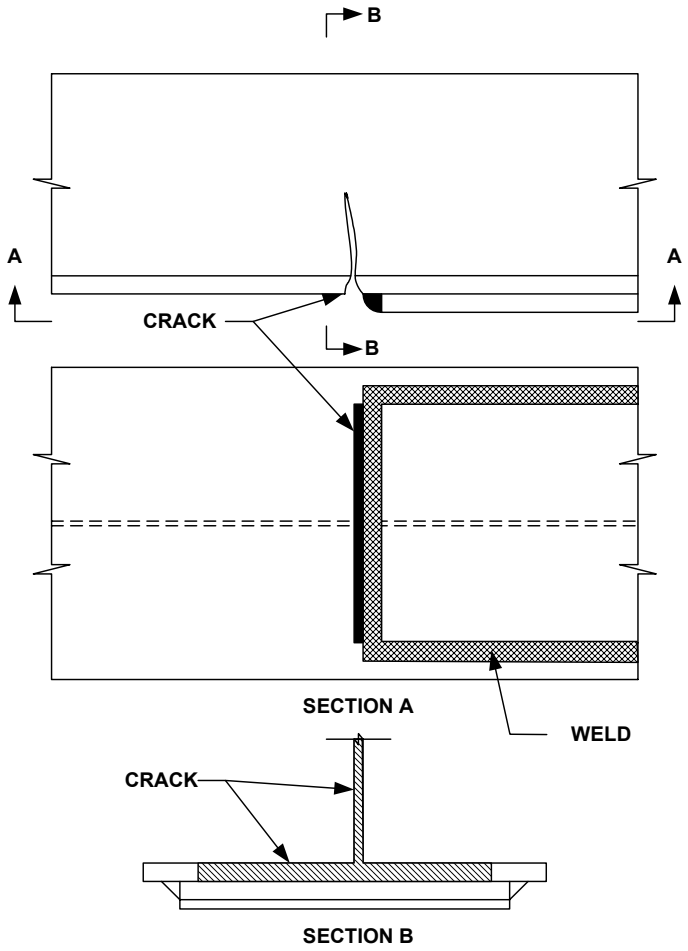


Figure 8.1.32 Through Crack Growth at Cover Plate Welded to Flange

The crack is readily visible as a through-the-thickness crack on both the inside and outside surfaces of the flange (see Figure 8.1.33).

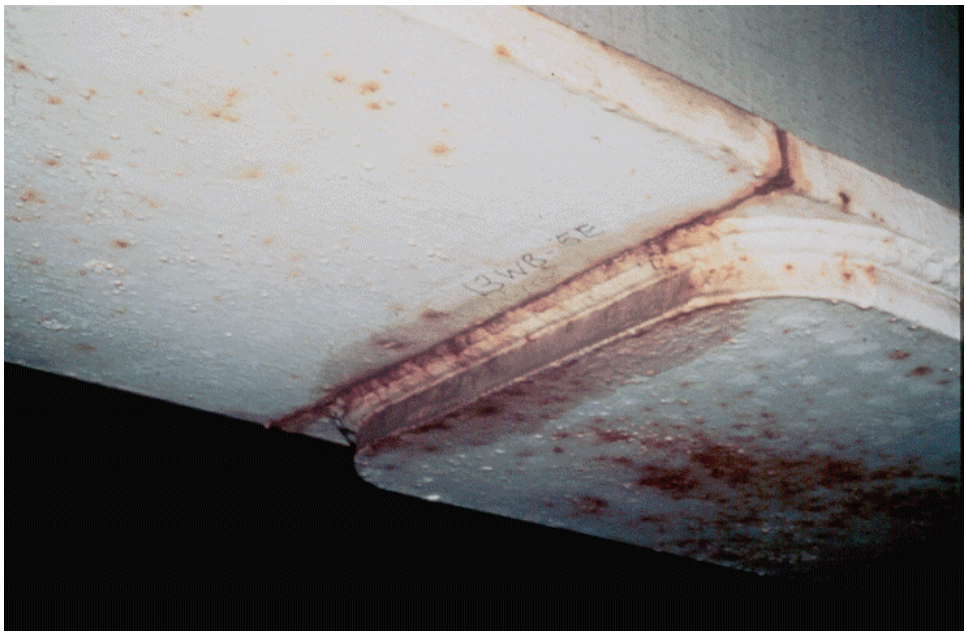


Figure 8.1.33 Through Crack at a Cover Plated Flange

Only about **5%** of the fatigue life is left for growing the Stage 2 through crack (see Figure 8.1.34).

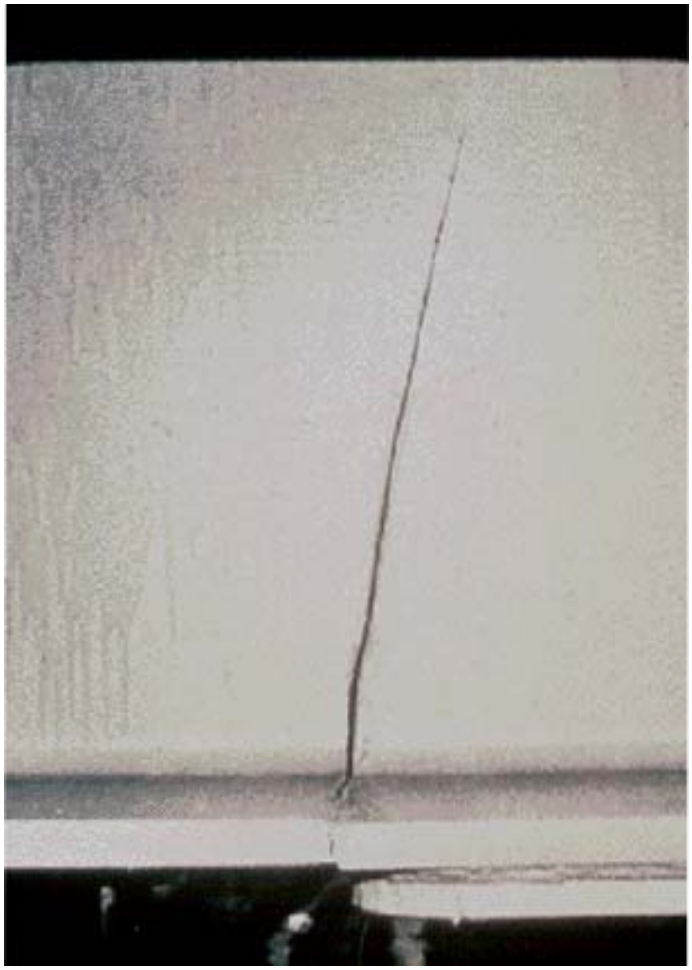


Figure 8.1.34 Through Crack has Propagated into the Web

Inspection

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

When the fatigue life is finally used up, that is the fatigue crack has grown to a critical size and stress intensity, then fracture will occur. The brittle fracture surface will appear crystalline or uneven, and will often reveal a herringbone pattern oriented toward the point of fracture initiation (see Figure 8.1.35).

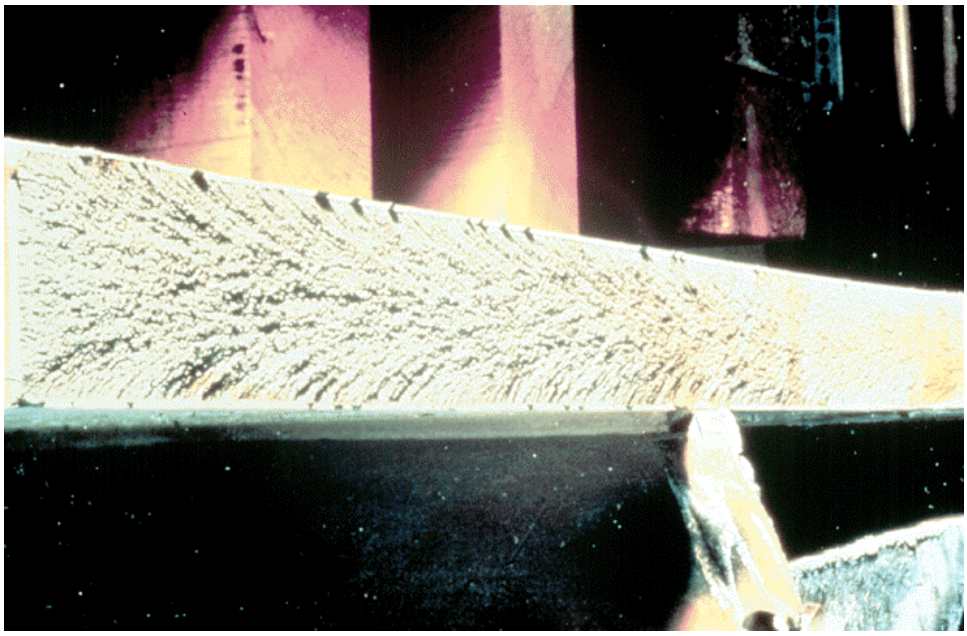


Figure 8.1.35 Brittle Fracture – Herringbone Pattern

Web Crack Failure Process

A common location for initiation of a web crack is at the weld toe of a transverse stiffener that is welded to the web of a beam. This type of crack grows in three stages (see Figure 8.1.36):

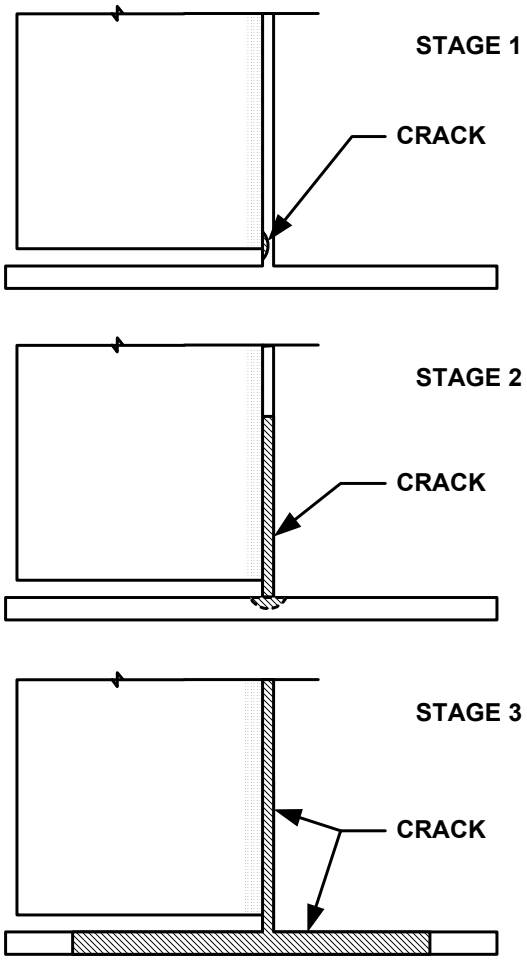


Figure 8.1.36 Crack Growth at Transverse Stiffener Welded to Web

Stage 1

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

A part-through stiffener crack is often just barely visible as a hairline along the toe of the weld (see Figure 8.1.37).

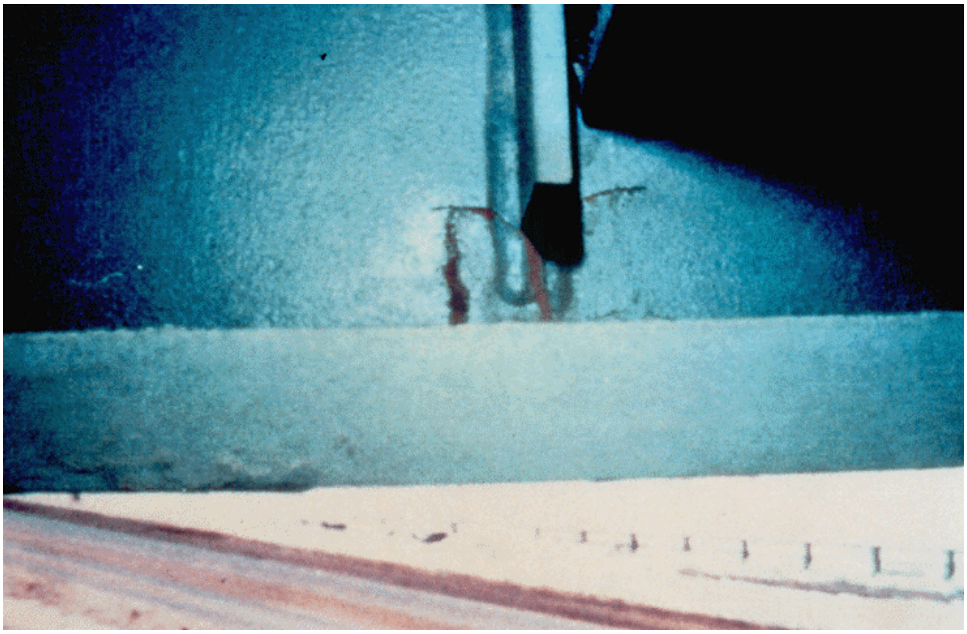


Figure 8.1.37 Part-Through Web Crack

This type of stiffener crack expends about 80% of its fatigue life propagating in Stage 1.

Stage 2

After it breaks through the web, the shape changes into a two-ended (or more) through crack which propagates up and down the web (see Figure 8.1.38).

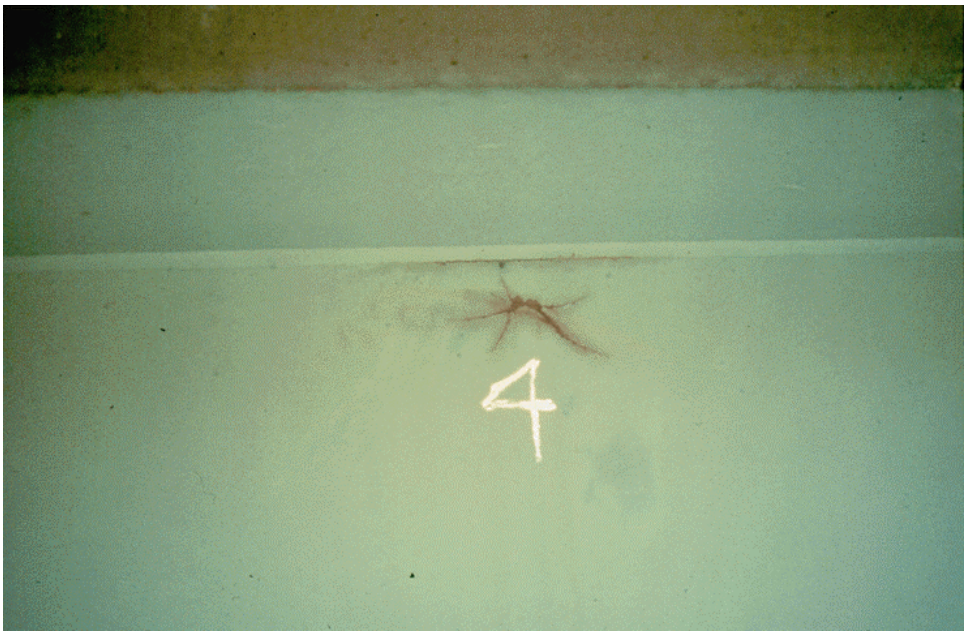


Figure 8.1.38 Through Crack in the Web

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

The stiffener crack expends about 15% of the fatigue life propagating in Stage 2.

Stage 3

Eventually, the lower front will reach the bottom flange, and the three-ended crack will then propagate with two fronts moving across the flange and one front moving farther up the web, until the member fractures (see Figure 8.1.39).



Figure 8.1.39 Through-Crack Ready to Propagate into the Flange

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

The stiffener crack expends its remaining 5% of the fatigue life propagating in Stage 3.

When steel lacks sufficient toughness and perhaps is also at a low temperature, a member can prematurely fracture at an early stage of crack growth.

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

8.1.5 AASHTO Detail Categories for Load-Induced Fatigue

For purposes of designing bridges for fatigue caused by in-plane bending stress, the details are grouped into categories labeled A to E' (see Figure 8.1.40). The classification of details by category does not apply to details that crack due to out-of-plane distortion. Each letter represents a rating given to a detail that indicates its level of fatigue strength. The details assigned to the same category have about equally severe stress concentrations and comparable fatigue lives. The alphabetical classification by the severity of the stress concentration is a useful method of identifying fatigue strength.

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

Category A

Category A refers to "base metal" or plain material with rolled or cleaned surfaces away from welded, riveted, or bolted connections. This condition has the longest fatigue life. It is not common practice to examine these base metal regions for fatigue cracks unless the regions are susceptible to distortion, because cracks usually develop at nearby connection details with lower fatigue strength categories.

Category B

Category B includes the following welds, structural details, and high strength bolted joints:

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

Category B'

Category B' is a new (1987) subcategory including details similar to those of Category B, but more sensitive to fatigue:

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

Category C

Category C includes transverse stiffeners, very short attachments, and transverse groove welds with reinforcement not removed:

- Base metal at welds connecting transverse stiffeners or vertical gusset plates to connection and gusset plates of girder webs or flanges
- Transverse full penetration groove welds, weld reinforcement not removed, but with weld soundness established by NDT
- Groove or fillet welded horizontal gusset or attachment, the length of which (in the direction of the main member) is less than 2 inches (50 mm)
- Groove welded attachments 6 to 24 inches (150 to 610 mm) in length with transition radius
- Intersecting plates connected by fillet welds with the discontinuous plate not more than 1/2 inch (13 mm) thick
- Shear connectors

Category D

Category D includes welded short attachments, welded connections with sharp transition curves, and riveted joints:

- Welded attachments with a groove or fillet weld in the direction of the main member between 2 inches and 4 inches (50 and 100 mm) long but less than 12 times the plate thickness
- Groove welded attachments with transition radius between 2 and 6 inches (50 and 150 mm)
- Groove welded attachments with unequal plate thickness, weld perpendicular to attachment, weld reinforcement removed, and a transition radius of at least 2 inches (50 mm)
- Fillet weld 2 inches (50 mm) or larger and welds ground smooth
- Riveted connections, net section

Category E and E'

Categories E and E' include details that have the lowest fatigue strength in comparison to those in other categories. Generally, for welded details in this group with the same configurations, Category E' applies if the flange plate thickness exceeds 0.8 inch (20 mm) or if the attachment plate thickness is 1 inch (25 mm) or more. Of all the details, those in categories E and E' are the most susceptible to fatigue crack growth. These details should be examined at every inspection and include:

- Ends of partial length cover plates on girder or beam flanges
- Welded attachment, with groove or fillet weld in the direction of the main members, more than 4 inches (100 mm) or 12 times the plate thickness; this is a common detail on bridge girders (e.g., the gusset plate which connects lateral bracing to the girders)
- Welded attachment with curved transition radius
- Welded attachment with loads transverse to welds
- Intermittent fillet welds
- Shear stress on the throat of a fillet weld (Formerly classified as Category F)

TABLE 10.3.1B

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
Plain Member	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less.	T or Rev[a]	A	1,2
Built-Up Members	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds (with backing bars removed) or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3,4,5,7
	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds with backing bars not removed, or by continuous partial penetration groove welds parallel to the direction of applied stress.	T or Rev	B′	3,4,5,7
	Calculated flexural stress at the toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at ends of partial length welded coverplates with high-strength bolted slip-critical end connections. (See Note[b])	T or Rev	B	22
	Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends, or wider than flange with welds across the ends:			
	(a) Flange thickness ≤ 0.8 in.	T or Rev	E	7
	(b) Flange thickness > 0.8 in.	T or Rev	E′	7
	Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	T or Rev	E′	7
Groove Welded Connections	Base metal and weld metal in or adjacent to full penetration groove weld splices of rolled or welded sections having similar profiles when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	8,10
	Base metal and weld metal in or adjacent to full penetration groove weld splices with 2 ft radius transitions in width, when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	13
	Base metal and weld metal in or adjacent to full penetration groove weld splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of the applied stress, and weld soundness established by nondestructive inspection:			
	(a) AASHTO M 270 Grades 100/100W (ASTM A 709) base metal	T or Rev	B′	11,12
	(b) Other base metals	T or Rev	B	11,12
	Base metal and weld metal in or adjacent to full penetration groove weld splices, with or without transitions having slopes no greater than 1 to 2½, when the reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8,10,11,12
Groove Welded Attachments—Longitudinally Loaded[c]	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is less than 2 in.	T or Rev	C	6,15
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15

Figure 8.1.40 AASHTO Table 10.3.1B - Fatigue Categories of Bridge Details

TABLE 10.3.1B (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	15
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	15
	Base metal adjacent to details attached by full or partial penetration groove welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without end welds ground smooth.	T or Rev	E	16
Groove welded Attachments—Transversely Loaded[c,d]	Detail base metal attached by full penetration groove welds with a transition radius, R, regardless of the detail length and with weld soundness transverse to the direction of stress established by nondestructive inspection:			
	—With equal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	—With equal plate thickness and reinforcement not removed	T or Rev		16
	(a) Transition radius ≥ 6 in.		C	
	(b) 6 in. > Transition radius ≥ 2 in.		D	
	(c) 2 in. > Transition radius ≥ 0 in.		E	
	—With unequal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii with unequal plate thickness and reinforcement not removed.	T or Rev	E	16
Fillet Welded Connections	Base metal at details connected with transversely loaded welds, with the welds perpendicular to the direction of stress:			
	(a) Detail thickness ≤ 0.5 in.	T or Rev	C	14
	(b) Detail thickness > 0.5 in.	T or Rev	See Note[e]	
	Base metal at intermittent fillet welds.	T or Rev	E	—
	Shear stress on throat of fillet welds.	Shear	F	9
Fillet Welded Attachments—Longitudinally Loaded[c,d]	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, is less than 2 in. and stud-type shear connectors.	T or Rev	C	15,17,18,20
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	15,17
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress greater than 12 times the plate thickness or greater than 4 in.:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	7,9,15,17
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E′	7,9,15

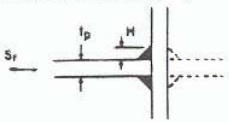
Figure 8.1.40 AASHTO Table 10.3.1B - Fatigue Categories of Bridge Details, continued

TABLE 10.3.1B (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 10.3.1A)	Illustrative Example (See Figure 10.3.1C)
	Base metal adjacent to details attached by fillet welds with a transition radius, R, regardless of the detail length:			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Fillet Welded Attachments—Transversely Loaded with the Weld in the Direction of Principal Stress[c,f]	Detail base metal attached by fillet welds with a transition radius, R, regardless of the detail length (shear stress on the throat of fillet welds governed by Category F):			
	—With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 2 in.		D	
	(b) 2 in. > Transition radius ≥ 0 in.		E	
	—For all transition radii without the end welds ground smooth.	T or Rev	E	16
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connecting materials.	T or Rev	B	21
	Base metal at net section of high-strength bolted bearing-type connections.	T or Rev	B	21
	Base metal at net section of riveted connections.	T or Rev	D	21
Eyebar or Pin Plates	Base metal at the net section of eyebar head, or pin plate	T	E	23, 24
	Base metal in the shank of eyebars, or through the gross section of pin plates with:			
	(a) rolled or smoothly ground surfaces	T	A	23, 24
	(b) flame-cut edges	T	B	23, 24

[a] "T" signifies range in tensile stress only, "Rev" signifies a range of stress involving both tension and compression during a stress cycle.
[b] See Wattar, Albrecht and Sahli, *Journal of Structural Engineering*, ASCE, Vol. III, No. 6, June 1985, pp. 1235–1249.
[c] "Longitudinally Loaded" signifies direction of applied stress is parallel to the longitudinal axis of the weld. "Transversely Loaded" signifies direction of applied stress is perpendicular to the longitudinal axis of the weld.
[d] Transversely loaded partial penetration groove welds are prohibited.
[e] Allowable fatigue stress range on throat of fillet welds transversely loaded is a function of the effective throat and plate thickness. (See Frank and Fisher, *Journal of the Structural Division*, ASCE, Vol. 105, No. ST9, Sept. 1979.)

$$S_r = S_r^C \left(\frac{0.06 + 0.79H/t_p}{1.1 t_p^{1.6}} \right)$$



where S_r^C is equal to the allowable stress range for Category C given in Table 10.3.1A. This assumes no penetration at the weld root.
[f] Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.

Figure 8.1.40 AASHTO Table 10.3.1B - Fatigue Categories of Bridge Details, continued

8.1.6 Fatigue Crack Categories

Out-of-Plane Distortion

The largest category of fatigue cracking is a result of out-of-plane distortion across a small gap, usually a segment of a girder web (see Figure 8.1.41). In general, the cracks form in planes parallel to the stresses from loading and are not detrimental to the performance of the structure, providing they are discovered and retrofitted

before becoming perpendicular to the primary stresses from loads.

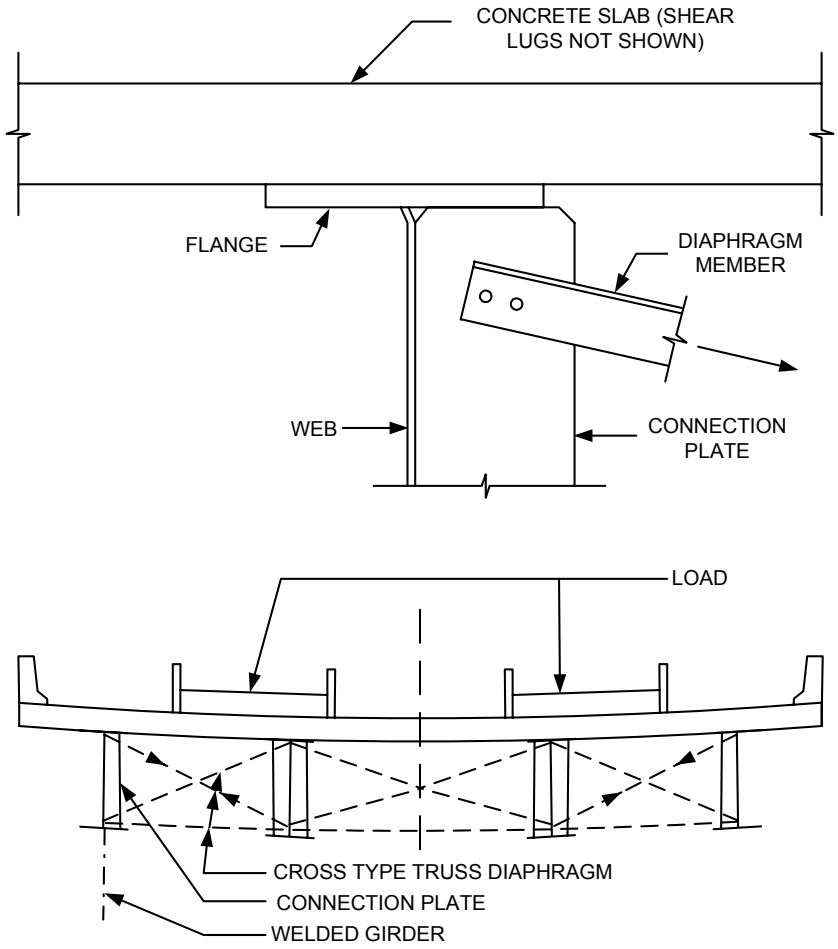


Figure 8.1.41 Out-Of-Plane Distortion in Web Gap at End of Transverse Connection Plate

Two very common instances are in the web gap at unconnected cross-frame connection plates, and at similar web gaps at floorbeam connections to main girders.

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

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

When distortion-induced cracking develops in a bridge, there are usually large numbers of cracks that form before corrective action is taken, because the cyclic stresses are often very high. As a result, many cracks form simultaneously in the structural system (see Figure 8.1.42).



Figure 8.1.42 Crack Near Top Flange at a Diaphragm Connection Plate which is on the Opposite Side of the Web

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

Even though distortion induced cracks are usually parallel to the primary stress direction, they can turn perpendicular. Drilled holes are often used as crack stoppers.

Details and Defects

Low fatigue strength details and initial defects make up the next largest category of cracked members.

Low Fatigue Strength Details

Low fatigue strength details, such as cover-plated beams and welded web and flange gusset plates, should be carefully inspected on bridges that have experienced large numbers of stress cycles (see Figure 8.1.43).

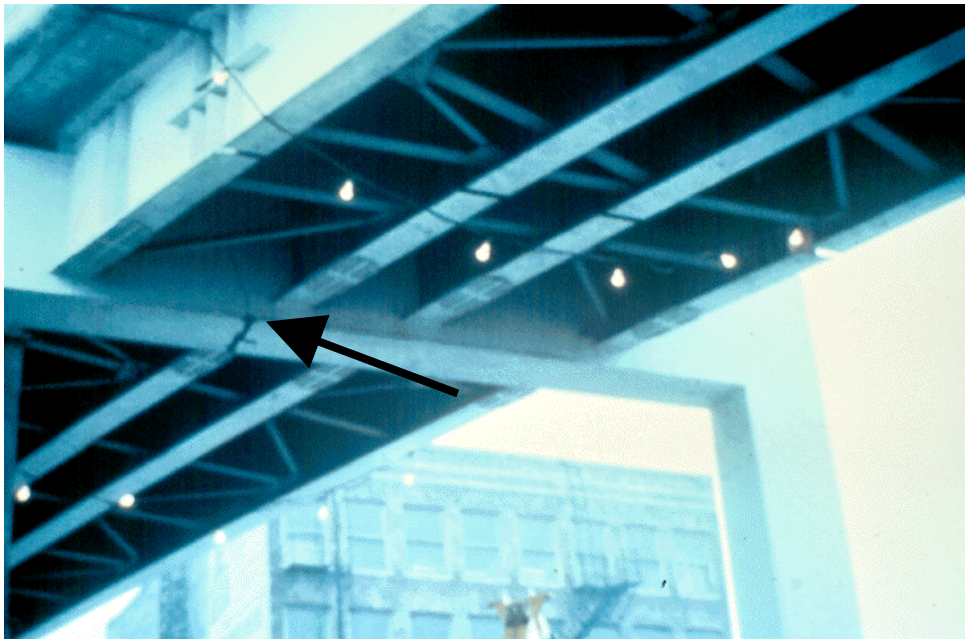


Figure 8.1.43 Category E Flange Penetration Through Web Plate

Initial Defects

Initial defects, in many cases, are cracks resulting from poor quality welds (see Figure 8.1.44). Many of these cracks occurred because the groove-welded element was considered a "secondary" attachment with no established weld quality criteria (e.g., splices in longitudinal web stiffeners). Lack-of-fusion discontinuities in welds can lead to crack initiation. Intersecting welds can provide a path for the crack to travel from secondary members to main members (see Figure 8.1.45).

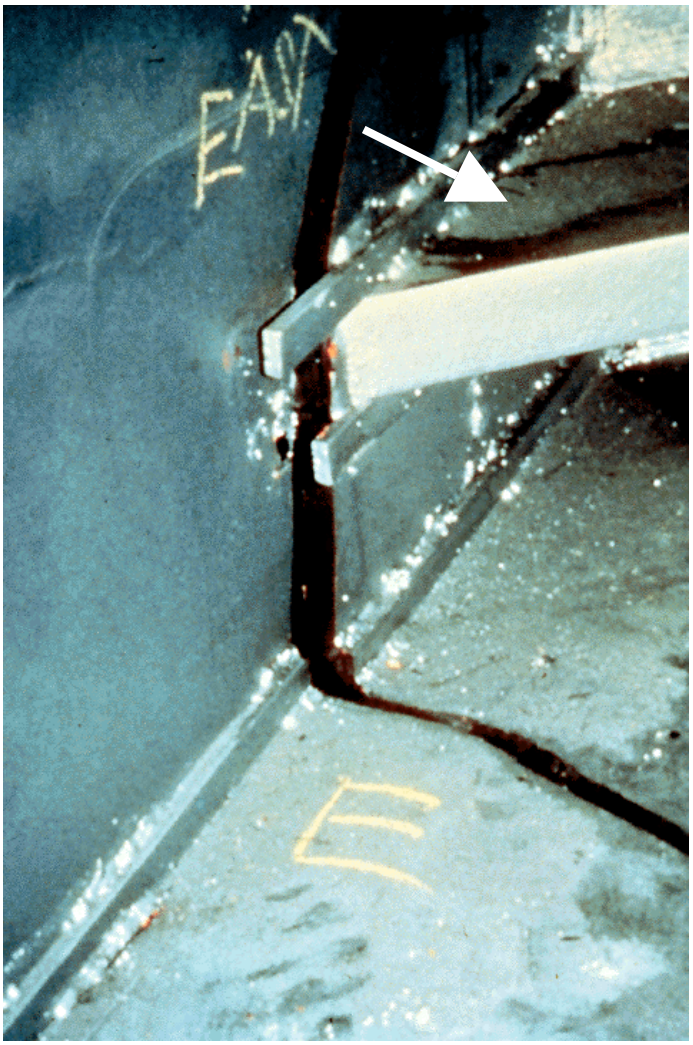


Figure 8.1.44 Poor Quality Welds inside cross girder on Figure 8.1.43.

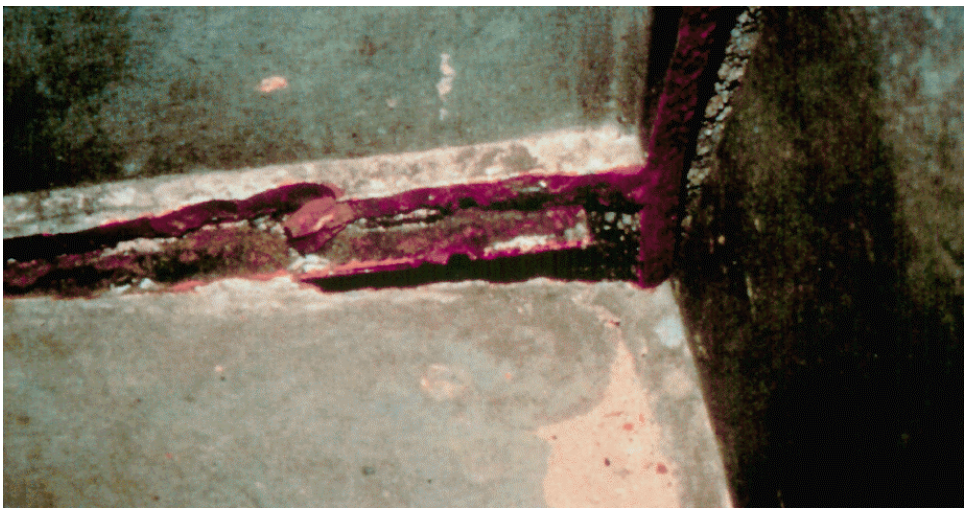


Figure 8.1.45 Intersecting Welds

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

Copes

The use of coped members, such as stringers, floorbeams, and diaphragms, is common in bridge floor systems. Copes are often flame cut, resulting in residual tensile stresses along the cut edges, approaching the yield point (see Figure 8.1.46).

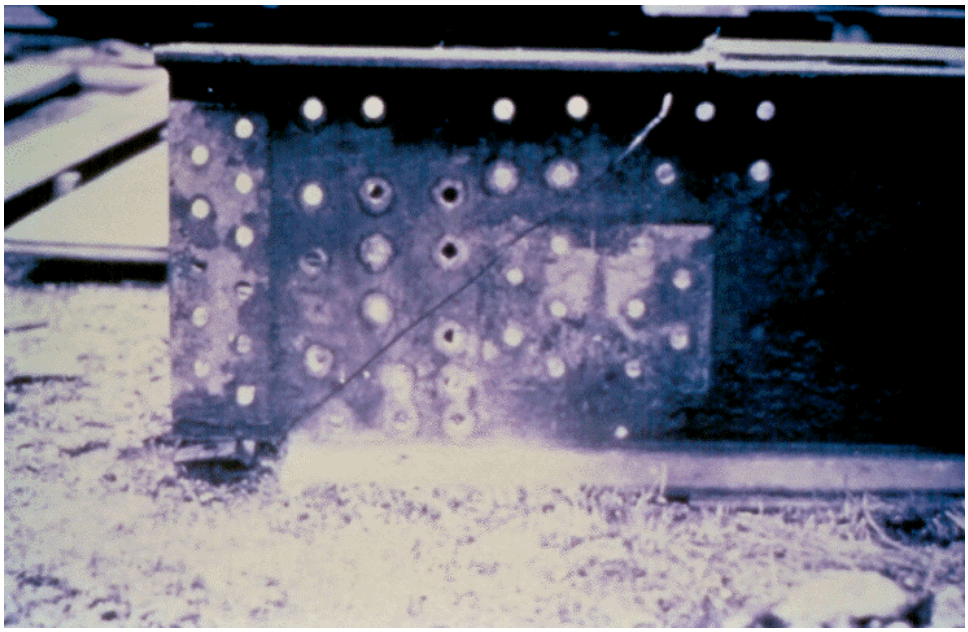


Figure 8.1.46 Fracture of a Coped Member

Flange Terminations

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

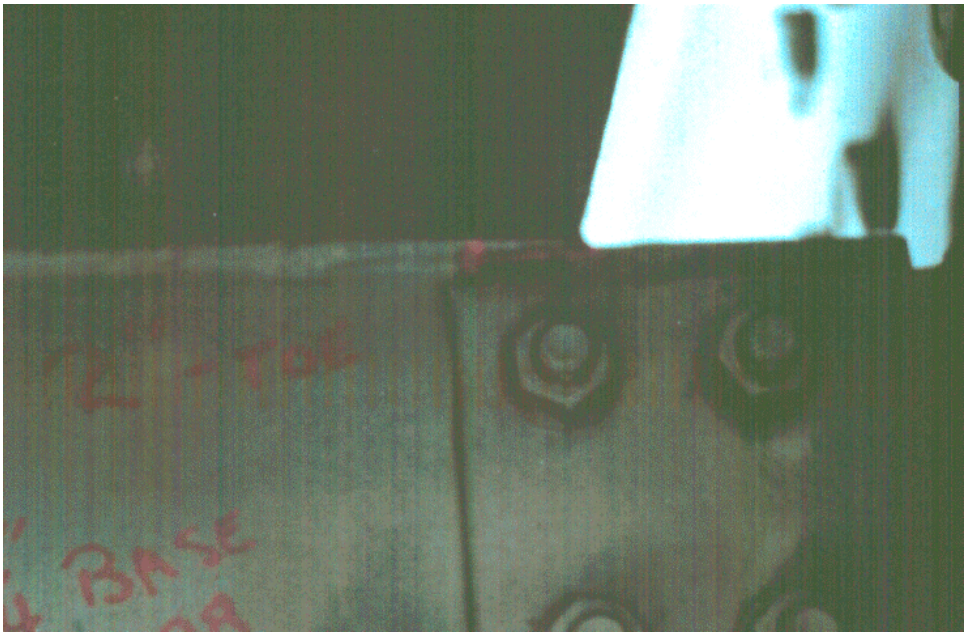


Figure 8.1.47 Flange Termination

End Restraints

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

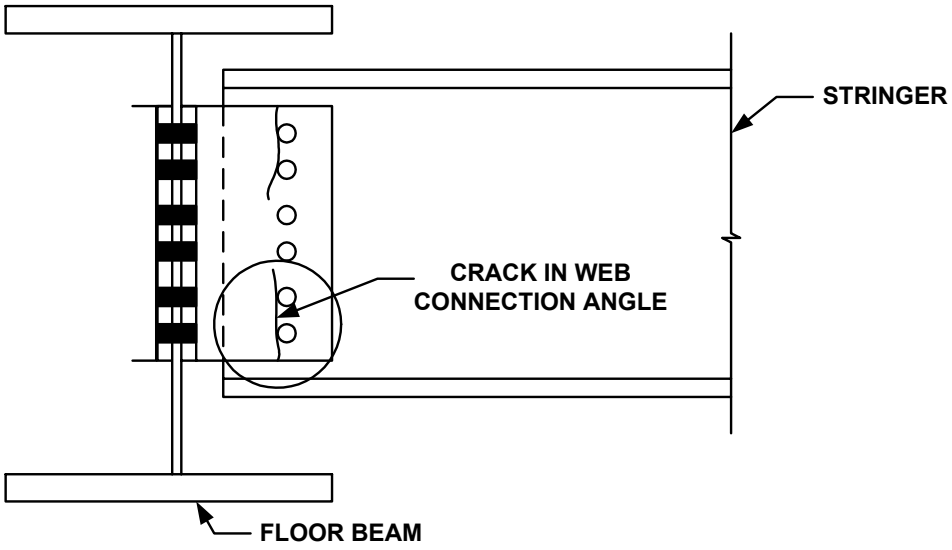


Figure 8.1.48 Cracked Stringer to Floorbeam Connection Angle



Figure 8.1.49 Cracked Stringer to Floorbeam Connection Angle

8.1.7 Inspection Locations and Procedures

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

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

- The degree of redundancy
- The live load member stress
- The propensity of the material to crack or fracture
- The condition of specific FCMs
- The existence of fatigue prone design details
- The previous number and size of loads
- The predicted number and size of loads

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

Fracture Critical Bridge Types

The following is a list of identified fracture critical steel bridge superstructures which are susceptible to failure due to fatigue cracking. The bridges are listed in order, with the most susceptible first:

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

Fatigue cracks can develop in steel bridges as a result of repeated loading. Generally, the stress fluctuation, frequency, and type of detail are the most important factors. Recognizing and understanding the behavior of connections and details is crucial if the inspector is to properly inspect FCMs. This is because connections and details are often the locations of highest stress concentrations.

Inspection of Details

The thoroughness of a fracture critical member inspection should be in the order of their susceptibility to fatigue crack propagation. Therefore, the order should be from the highest susceptibility (E') to the lowest (A).

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

It should be noted that the direction of live load stress at a detail is a major factor in fatigue strength.

Welded Details

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

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

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

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

- Girder webs at floorbeam and diaphragm connections (see Figure 8.1.50)
- Ends of diaphragm connection plates in girder bridges
- Box girder webs at diaphragms
- Lateral bracing gusset plates on girder webs at floorbeam connections
- Floorbeam and cantilever bracket connections to girders
- Pin connected hanger plates and fixed pin plates

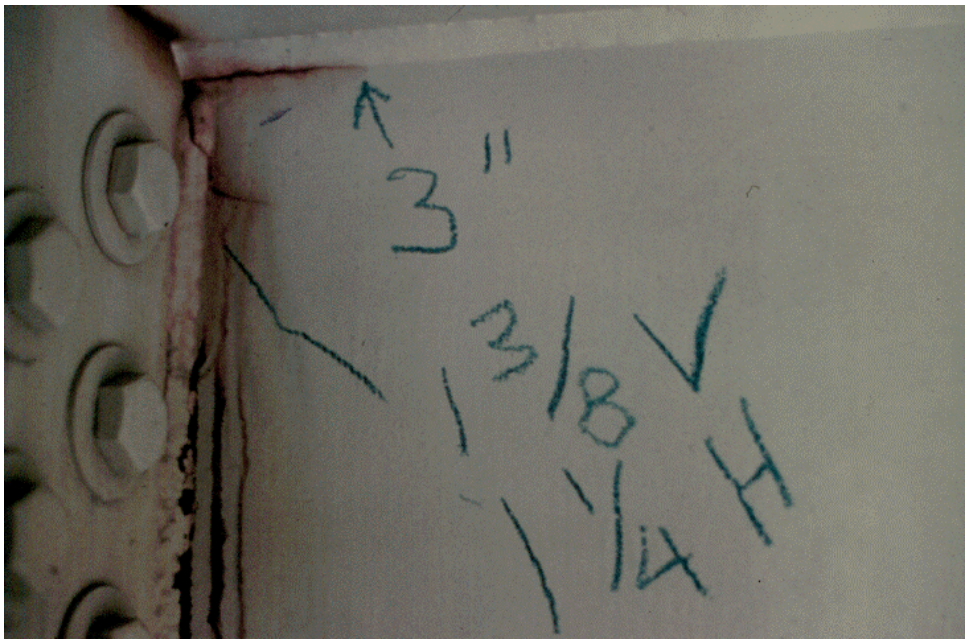


Figure 8.1.50 Cracks at Top of Floorbeam Connection to Girder

For main members, inspect the following locations:

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

For connections and attachments, inspect the following locations:

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

Riveted and Bolted Details

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

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

- Cracking and prying of rivets/bolts at end connections
- End connection angle failure (see Figure 8.1.51)
- Girder webs at floorbeam connections
- Floorbeam connections to girders
- Diaphragm connections to girders
- Cantilever bracket connections to girders
- Truss hangers
- Eyebars
- Tack welds

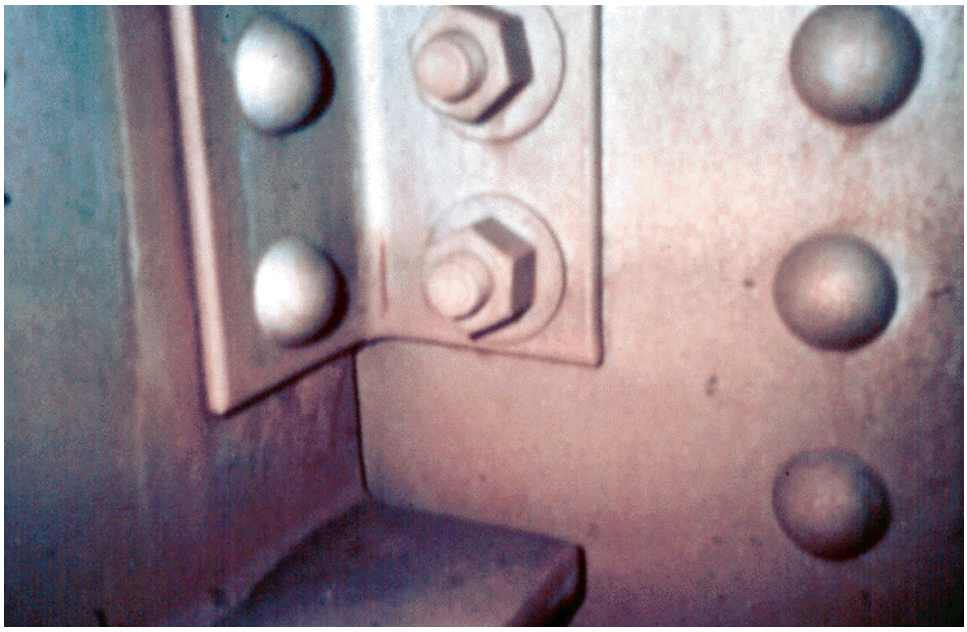


Figure 8.1.51 Cracked Stringer Connection Angle

Procedures

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Section 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

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

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 the re-examination of the actual conditions can be made.

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

Cracks Perpendicular to Primary Stress

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

Cracks Parallel to Primary Stress

Cracks parallel to primary members are less serious than transverse cracks. Cracks parallel to the main direction of stress, do not reduce the capacity load and have less tendency to propagate. These cracks are still serious because they can turn perpendicular to the direction of stress at any time (see Figure 8.1.52).

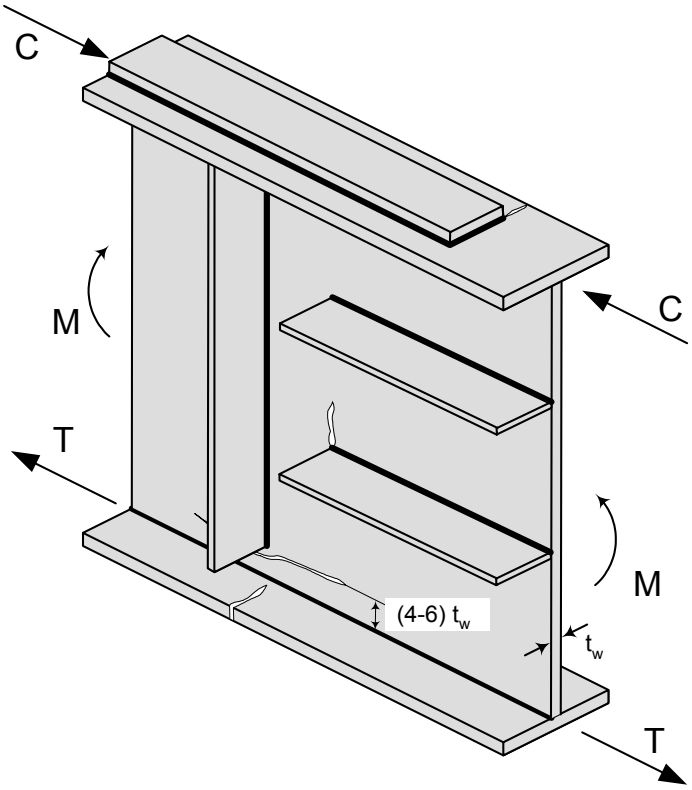


Figure 8.1.52 Cracks Perpendicular or Parallel to Applied Stress

Corrosion

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

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

Defective welds are quite serious but usually will not be detected unless special testing is performed. Defective welds can eventually crack.

Recordkeeping and Documentation

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

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

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

Label the member using paint or other permanent markings: mark the ends of the crack, the date, compare to any previous markings, be sensitive to aesthetics at prominent areas (see Figure 8.1.42). Photograph the member and the defect.

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

Recommendations for Fracture Critical Members

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

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

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

8.1.8 Evaluation

State and federal rating guidelines systems have been developed in order to provide continuity in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Inspection Bridge Management System (BMS).

Application of the NBIS Rating Guidelines and Application of Condition State Assessment

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

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

8.2.1 Introduction

This section will take a detailed look at two different superstructure types: rolled steel multi-beams and fabricated steel multi-girders. Both are similar in a number of different characteristics, but there are some primary differences that make each superstructure type unique.

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

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

Fabricated girders are different in that they are custom made. The width and thickness of the flanges and webs can be varied to the necessary dimensions to optimize the design. Fabricated girders will generally have bearing stiffeners and intermediate stiffeners.

The following sections will review both superstructure types and their characteristics.

8.2.2 Design Characteristics

Rolled Multi-beam

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



Figure 8.2.1 Simple Span Rolled Multi-beam Bridge



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

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



Figure 8.2.3 Rolled Multi-Beam Bridge with a Cover Plate

Steel Multi-girders

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

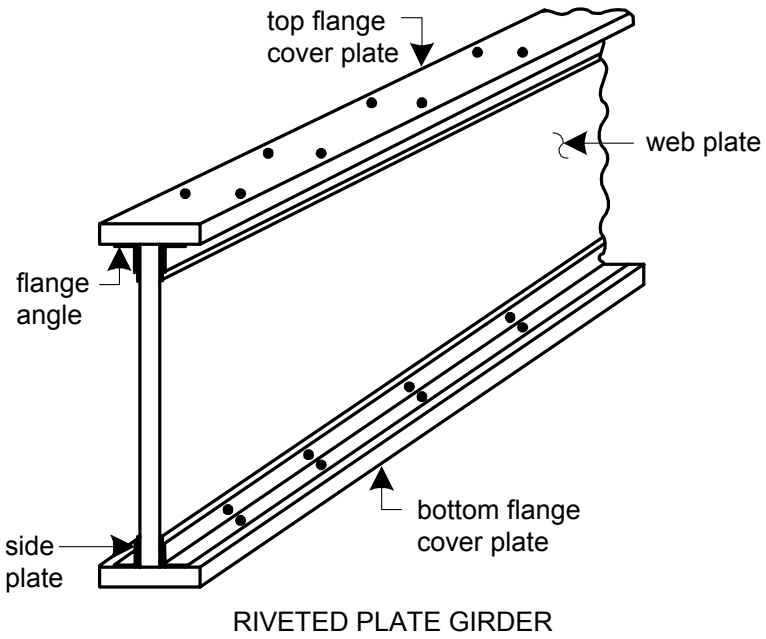


Figure 8.2.4 Built-up Riveted Plate Girder

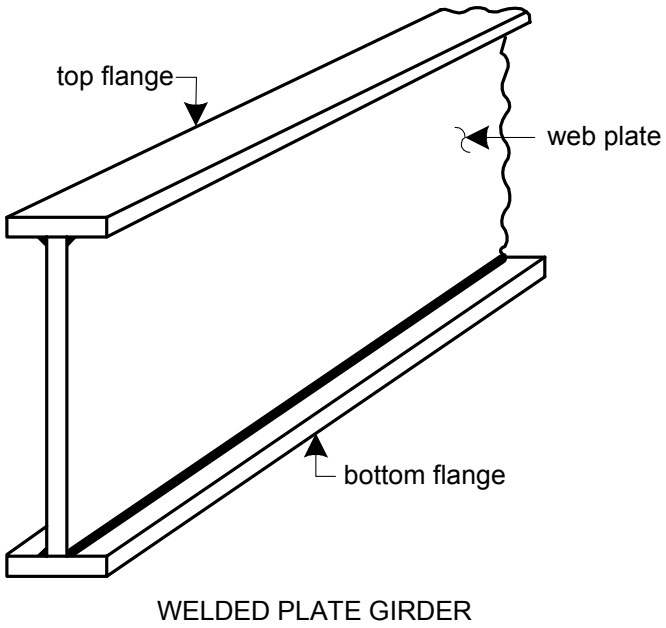


Figure 8.2.5 Welded Plate Girder

This bridge type can be found in single span (see Figure 8.2.6), multiple span, and continuous span designs (see Figure 8.2.7), and it is widely used when curved bridges are required (see Figure 8.2.8). Continuous welded multi-girders can have spans of over 152 m (500 feet). Like the rolled multi-beam bridge, this bridge type often incorporates composite action with the deck. Pin and hanger connections are also common (see Figure 8.2.9).



Figure 8.2.6 Single Span Fabricated Multi-girder Bridge



Figure 8.2.7 Continuous Span Fabricated Multi-girder Bridge



Figure 8.2.8 Curved Fabricated Multi-girder Bridge

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



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

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



Figure 8.2.10 Combination Rolled Beams and Fabricated Girder

Haunched Girder Design In continuous girder designs, additional girder strength is required in negative moment regions. This is accomplished through a method called haunching. Haunching can best be described as increasing the web depth for a specified portion of the girder. The regions above intermediate supports (i.e. piers and

bents) have negative moments larger than the adjacent positive moments. Typically, the girder depth used at the positive moment regions is not sufficient enough to resist these moments, so the web depth needs to be increased. (See Topic P.1 on Moments and Shear) However, instead of increasing the depth for the full length of the girder, the girder is haunched at the intermediate supports for economy. As a result, the girder design is optimized while minimizing the steel costs.

Three methods have been used to haunch girders.

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

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

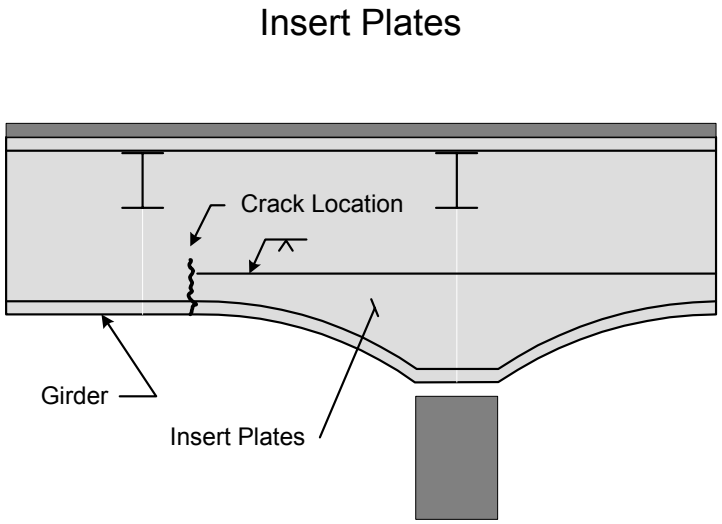


Figure 8.2.11 Web Insert Plate

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

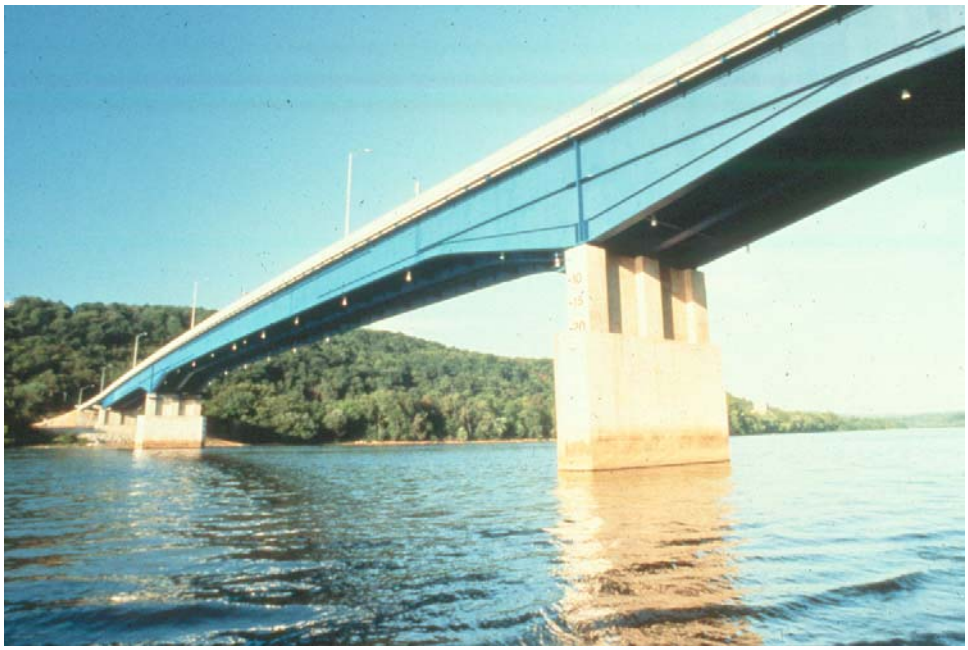


Figure 8.2.12 Fabricated Variable Depth Girder Bridge

Function of Stiffeners

As fabricated girders become larger, the depth of the web plate increases, and it becomes susceptible to web buckling (i.e., failure of the web due to compressive stress). Bridge designers prevent this from occurring by increasing the web thickness or by reinforcing the web with steel stiffener plates. Stiffeners can be either transverse (vertical) or longitudinal (horizontal). They can be placed on one or both sides of the web. The stiffeners limit the unsupported length of the web, which result in increased stability. The stiffeners help hold the web vertically to resist out-of-plane bending.

Primary and Secondary Members

Rolled Multi-beam

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



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

Steel Multi-girders

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



Figure 8.2.14 Curved Multi-girder Bridge

On straight bridges, the secondary members are the diaphragms, which are provided to stabilize the girders during construction of the deck and to help distribute live load (see Figure 8.2.15). Diaphragms can be rolled shapes (e.g., I-beams and channels) or they can be cross frames constructed from angles, tee shapes, and plates. They are usually attached to transverse web stiffeners which are normally referred to as connection plates. On older bridges, secondary members also include lateral bracing, but design specifications no longer require this feature.



Figure 8.2.15 Straight Multi-girder Bridge

Fatigue Prone Details and Failure

Some common areas for fatigue prone details are:

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

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

Fracture Critical Areas

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

8.2.3 Overview of Common Defects

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

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Tack welds in tension zones
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage

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

8.2.4 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

In steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel bridge, including:

- Magnetic particle
- Eddy current
- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Topic 13.3 for Advanced Inspection Techniques.

Locations

Bearing Areas

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

Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion.

Shear Zones

Examine the web areas near the supports for any section loss (see Figure 8.2.16). Shear stresses are greatest near the supports, while bending stresses are least near the supports (except for continuous structures). Therefore, the condition of the web is more critical near the ends than at mid-span. Also investigate the web for crippling due to overloads.

If welded cover plates are present, check carefully at the ends of the cover plates for cracks. If girders have been haunched by the use of insert plates, check the weld between the web and the insert plate.



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

Flexure Zones

The flexure zone of each girder includes the entire length between the supports (see Figures 8.2.17 and 8.2.18). Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, and gouges. Check the flanges in high stress areas for bending or flexure- related damage caused by live loads. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the beams over the intermediate supports have high flexural stresses due to negative moment (see Figures 8.2.19 and 8.2.20). Stresses are reversed in the region over the intermediate supports, and the top flange is in tension (refer to Topic P.1.9 and see Figure P.1.2.1).

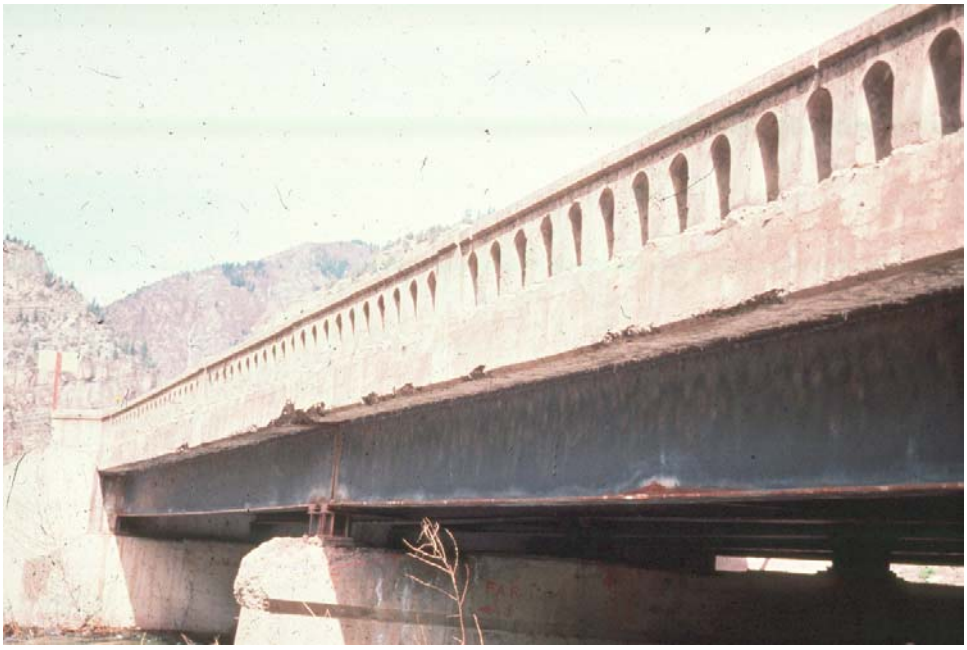


Figure 8.2.17 Midspan Flexural Zone on a Simple Rolled Multi-beam Bridge

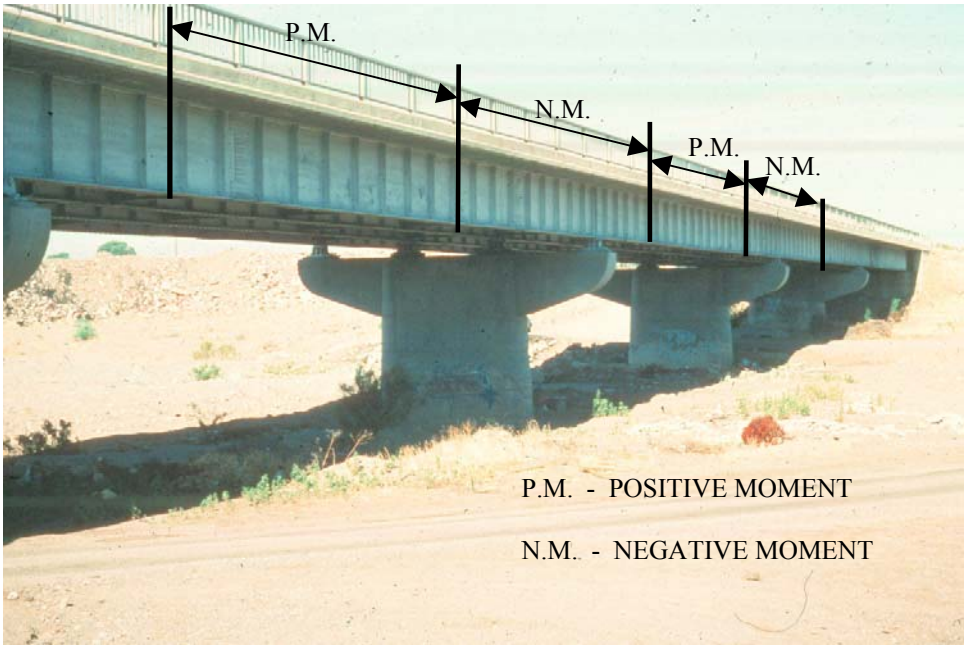


Figure 8.2.18 Midspan Flexural Zone on a Fabricated Multi-girder Bridge



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



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

Secondary Members

Examine the diaphragm and bracing connections for loose fasteners or cracked welds (see Figures 8.2.21 and 8.2.22). This problem is most common on skewed bridges, and it has also been observed on bridges with a high frequency of combination truck loads. Check horizontal connection plates, which can trap

debris and moisture and are susceptible to a high degree of corrosion and deterioration. Check for distorted members. Distorted secondary members may be an indication the primary members may be overstressed or the substructure may be experiencing differential settlement.



Figure 8.2.21 Multi-beam Diaphragm



Figure 8.2.22 Multi-girder Diaphragm

Areas That Trap Water and Debris

Areas that trap water and debris can result in active corrosion cells and excessive

loss in section. This can result in notches susceptible to fatigue or perforation and loss of section. On fabricated multi-girder bridges check:

- along the bottom flanges
- pockets created by diaphragm connections
- lateral bracing gusset plates
- areas exposed to drainage runoff

Areas Exposed to Traffic

Inspect any beams that may have been damaged due to collision very closely. Loss of section, cracking, and any shape distortion should be carefully documented (see Figures 8.2.23 and 8.2.24).



Figure 8.2.23 Collision Damage on a Rolled Multi-beam Bridge



Figure 8.2.24 Collision Damage on a Fabricated Multi-girder Bridge

Fatigue Prone Details

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

If the structure has been painted, breaks in the paint accompanied by rust staining indicate the possible existence of a fatigue crack. Investigate the areas surrounding field splice cover plates on the tension flange (see Figure 8.2.25). The suspected crack area should be cleaned to determine the existence of a crack and its extent. If a crack with rust staining exists in the paint, the fatigue cracks in the steel can already be up to 6 mm (1/4 inch) deep in the beam flange. Check any attachment welds located in the tension zone of the beam, such as lighting brackets, utility attachments, catwalks and signs (see Figure 8.2.26). Also check web stiffener welds, welded flange splices, and intersecting welds (see Figure 8.2.27).

Check for fatigue cracks due to web-gap distortion. Check the web plate for cracks near the top of the diaphragm connection plate, especially in negative moment areas where, in the past, connection plates have not been connected to the girder flange. Cracks have occurred at such locations due to out-of-plane distortions.

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

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

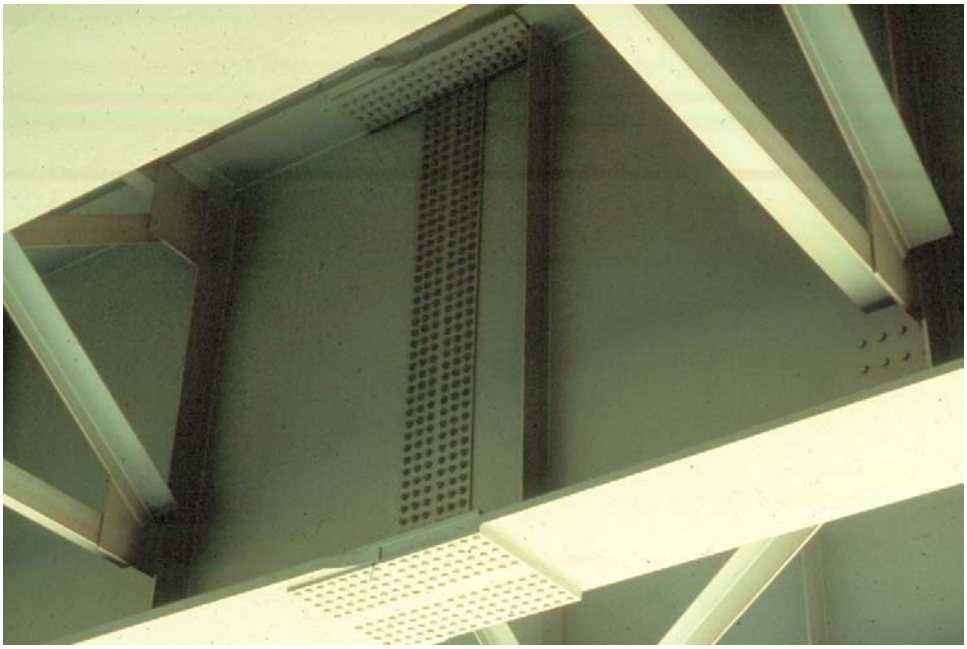


Figure 8.2.25 Field Splice



Figure 8.2.26 Welded Attachment in Tension Zone of a Beam



Figure 8.2.27 Fatigue Crack at Toe of Weld on a Fabricated Multi-girder Bridge

Fracture Critical Members

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

Out-of-plane Distortion

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

Girder Webs at Diaphragm Connections

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

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

Detection of cracks of such length is not difficult. Knowing that unattached ends of diaphragm connection plates are likely locations of fatigue cracks increases the certainty of early detection of these cracks (see Figure 8.2.28).

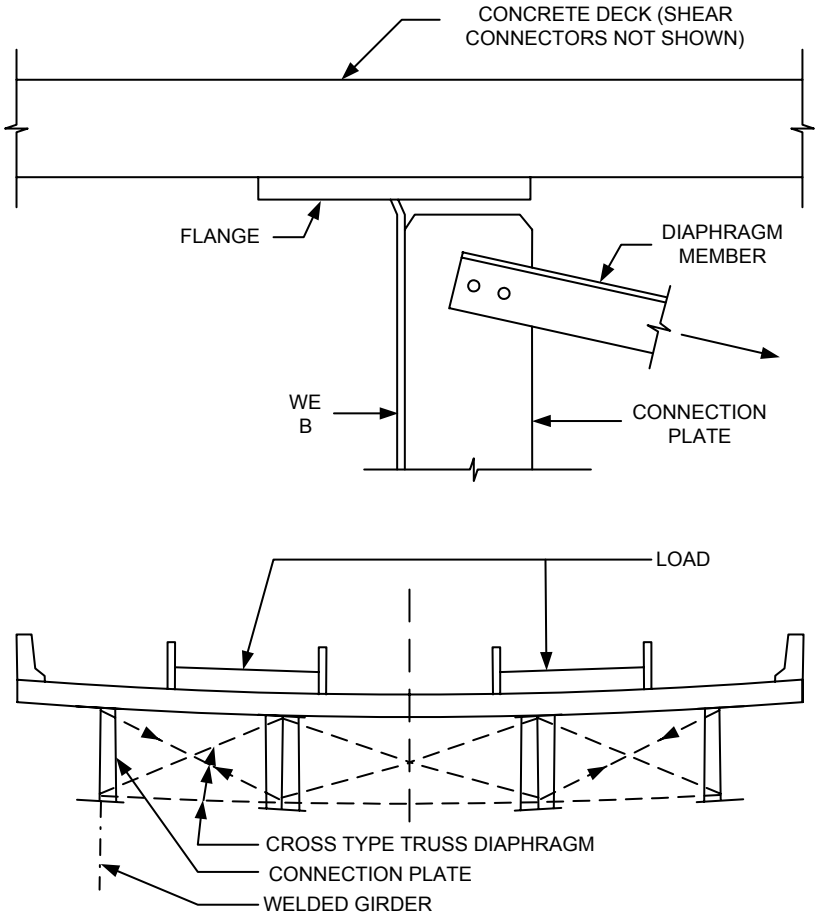


Figure 8.2.28 Out-of-plane Distortion in Web Gap at End of Transverse Connection Plate

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

- Skewed bridges - Another location where fatigue cracking has developed at the unattached lower end of diaphragm connection plate is in skewed bridges. Most of these diaphragms are perpendicular to the girders and thus are subjected to large differential vertical deflections which in turn cause out-of-plane distortion at the lower end of the diaphragm connection plate. If the girder flange is relatively thick and stiff against lateral

displacement, most of the deflection is accommodated by bending of the web plate within the gap between the flange and the end of the connection plate welds. Fatigue cracks may initiate.

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

Ends of Diaphragm Connection Plates in Girder Bridges

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

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

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

Lateral Gussets on Plate Girder Webs at Connections

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

8.2.5 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the girder/beam. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Smart Flags are also used to describe the condition of the steel superstructure, both unpainted and painted.

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

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

The unit quantity for the girder/beam is meters or feet and the total length of all girders/beams must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

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Topic 8.3 Steel Two-girder Systems

8.3.1 Introduction

Two-girder Systems

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

However, unlike the fabricated multi-girder bridge, the two-girder bridge has a floor system of smaller girders and beams that support the deck between the two main girders. The deck, therefore, rests on the top flanges of the girders and floor system.

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



Figure 8.3.1 General View of a Dual Two-girder Bridge

Through Girder Systems

A through girder bridge is similar to a two-girder bridge, in that it has only two main girders, it may be welded or riveted, it has web stiffeners, and it has girder-floorbeam or girder-floorbeam-stringer floor systems. However, the deck on a through girder bridge is placed between the girders which are visible from the roadway (see Figure 8.3.2).

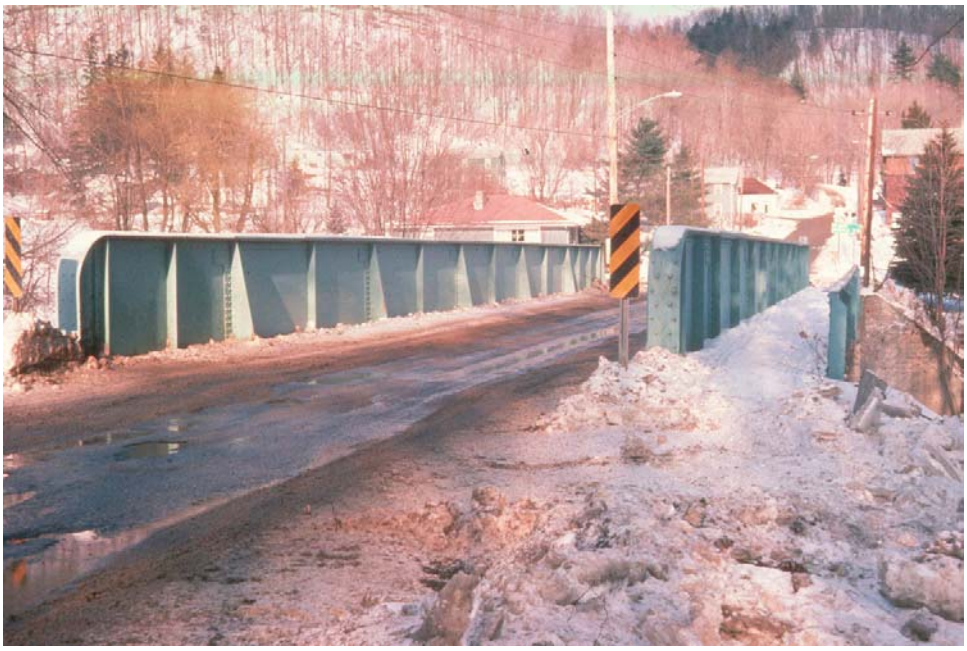


Figure 8.3.2 Through Girder Bridge

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



Figure 8.3.3 Through Girder Bridge with Limited Underclearance

Through girder bridges are most commonly single span structures, although multiple span and continuous configurations were constructed. A rare type of through girder has three or more girders, with the main girders actually separating the traffic lanes (see Figure 8.3.4). These structures are probably converted railroad or trolley bridges. Through girder bridges are commonly riveted.

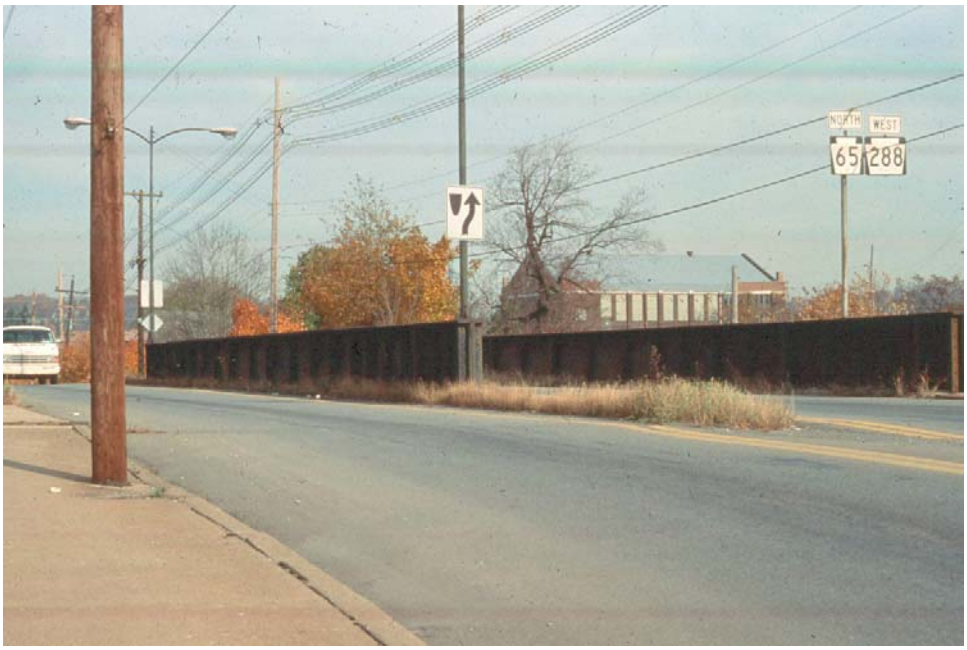


Figure 8.3.4 Through Girder Bridge with Three Girders

8.3.2 Design Characteristics

Floor System Arrangement

Two-girder Systems

In addition to having only two "main" girders, a two-girder bridge has a floor system that supports the deck between the girders. There are two types of floor systems found on two-girder bridges:

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

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



Figure 8.3.5 Two-girder Bridge with Girder-Floorbeam System

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



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

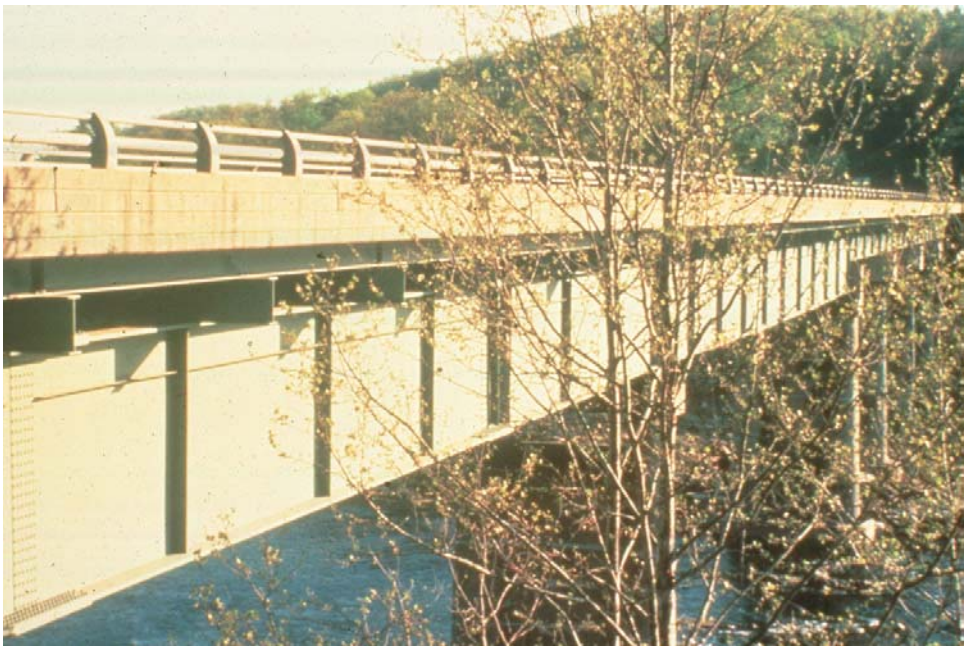


Figure 8.3.7 Two-girder Bridge with GFS System with Stacked Floorbeam and Stringers

Through Girder Systems

A through girder bridge has the same two floor systems as a two-girder bridge with the only difference being the location of the deck.

Primary and Secondary Members

Two-girder Systems

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



Figure 8.3.8 Underside View of Two-girder Bridge with Lateral Bracing System

Through Girder Systems

The primary members of a through girder bridge are the through girders, floorbeams, and stringers, if present. The secondary members are the lateral bracing (see Figure 8.3.9).

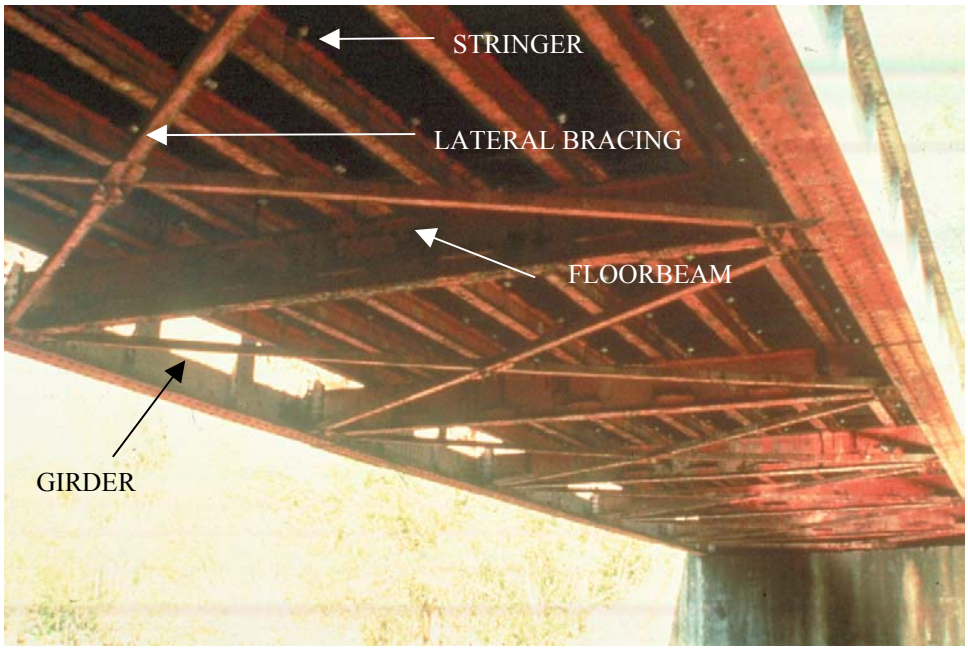


Figure 8.3.9 Underside View of Through Girder Bridge with Lateral Bracing

Fatigue Prone Details and Failure

Some common areas for fatigue prone details are:

- Fabrication welds
- Pin and hanger connections (if present)
- Welded cover plates
- Web stiffener welds
- Welded flange splices
- Intersecting welds
- Attachment welds located in the tension zone
- Web gaps
- Mechanical splices

Inspection of these areas will be discussed further in Topic 8.3.4.

Fracture Critical Areas

Two-girder and through girder bridges do not have load path redundancy. Both are therefore nonredundant and are fracture critical bridge types. The main girders are fracture critical members.

Pin and hanger assemblies in two-girder bridges are fracture critical members (see Figure 8.3.10). Failure of one pin or one hanger will cause collapse of the suspended span since there is no alternate load path (e.g., Mianus River Bridge). Pins in pin and hanger connections have fractured when "frozen" by corrosion. This is a critical situation when it occurs on a two-girder bridge.

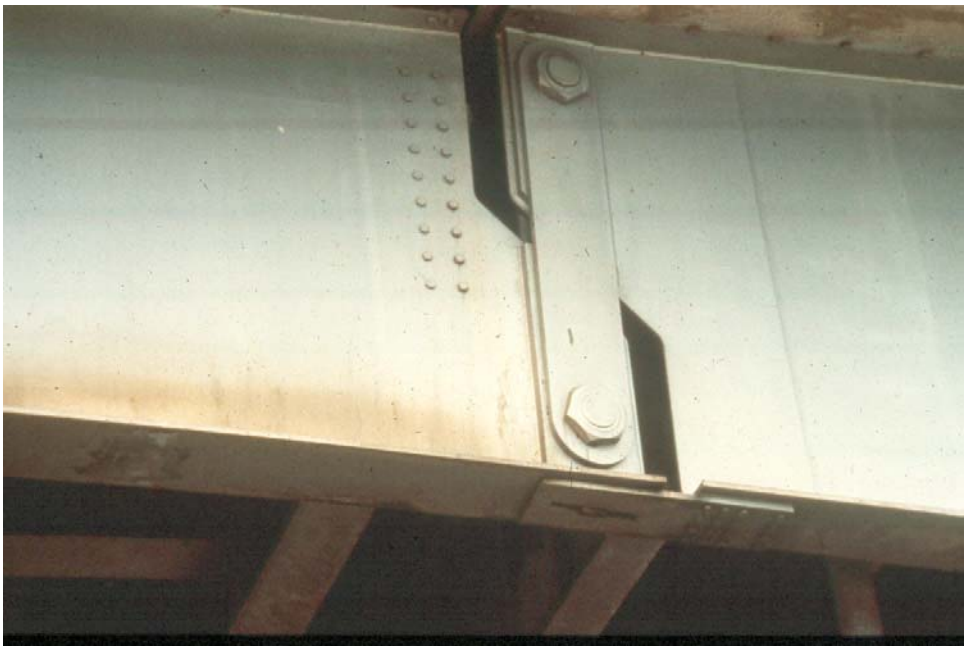


Figure 8.3.10 Two-girder Bridge with Pin and Hanger Connection

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

A fracture critical floorbeam satisfies one or more of the following conditions:

- Flexible or hinged connection to support girders
- Spacing greater than 14'-0"
- No stringers, or
- Stringers are configured as simple beams

8.3.3 Overview of Common Defects

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

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Tack welds in tension zones
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage

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

8.3.4 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

In steel two-girder and through girder systems as well as other steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection, including:

- Magnetic particle
- Eddy current
- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Topic 13.3 for Advanced Inspection Techniques.

Locations

Bearing Areas

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

Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion.

Shear Zones

Examine the web areas of the girders, floorbeams, and stringers near their supports

for corrosion and buckling. This is a critical area, especially if the web is coped or the flange is blocked (see Figures 8.3.11 and 8.3.12).



Figure 8.3.11 Shear Zone on a Two-girder Bridge



Figure 8.3.12 Web Area Near Support on a Through Girder Bridge

Flexure Zones

Check the tension and compression flanges for corrosion and section loss. Also, inspect the flange in high stress areas for bending or flexure-related damage caused by live loads. Check in both the positive and negative moment regions.

Check flange splice welds and longitudinal stiffener splice welds in tension areas (see Figures 8.3.13, 8.3.14, and 8.3.15).



Figure 8.3.13 Flexural Zone on a Two-girder Bridge



Figure 8.3.14 Longitudinal Stiffener in Tension Zone on a Two-girder Bridge



Figure 8.3.15 Flexural Zone on a Through Girder Bridge

Secondary Members

Investigate the diaphragms, if present, and the connection areas of the lateral bracing for cracked welds, fatigue cracks, and loose fasteners. Inspect the bracing members for any distortion or corrosion (see Figures 8.3.16 and 8.3.17).



Figure 8.3.16 Lateral Bracing Connection on a Two-girder Bridge

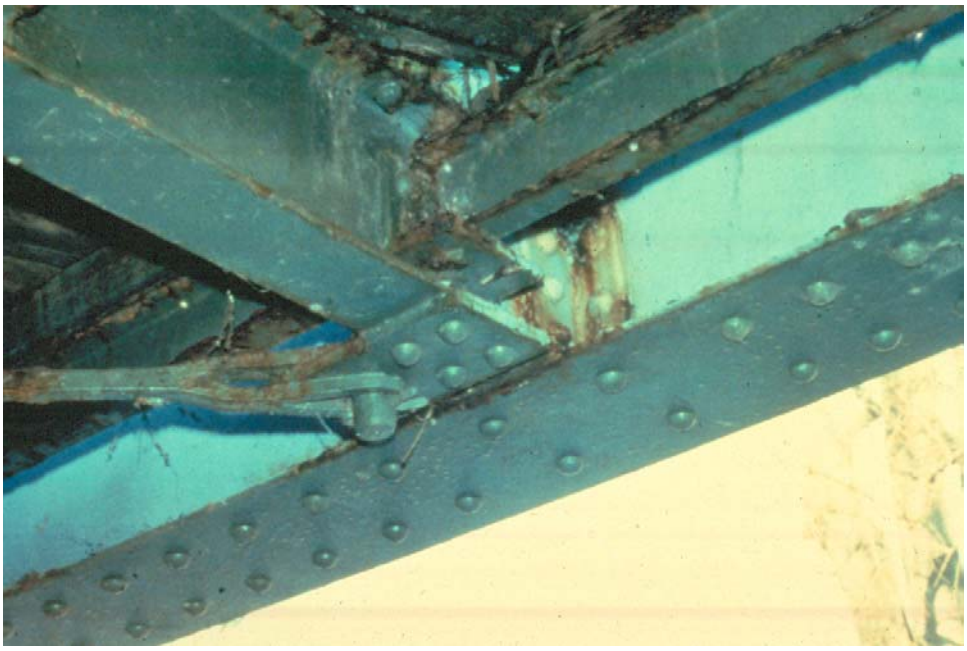


Figure 8.3.17 Lateral Bracing Connection on a Through Girder Bridge

Areas That Trap Water and Debris

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

On two-girder bridges check:

- along the bottom flanges of the girders
- pockets created by girder-floorbeams and floorbeam-stringer connections
- lateral bracing gusset plates
- areas exposed to drainage runoff

On through girder bridges check:

- along the bottom flanges of the girders
- along the girder webs at the curb line
- pockets created by girder-floorbeam connections
- pockets created by floorbeam-stringer connections
- lateral bracing gusset plates
- areas exposed to drainage runoff

Areas Exposed to Traffic

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

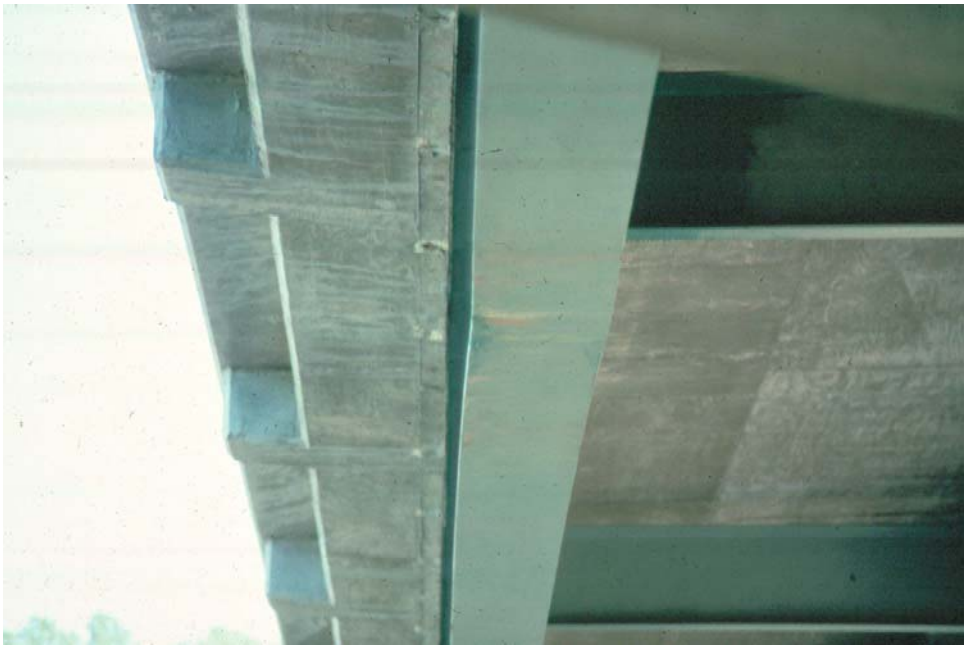


Figure 8.3.18 Collision Damage to a Two-girder Bridge



Figure 8.3.19 Collision Damage to a Through Girder Bridge

Fatigue Prone Details

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

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

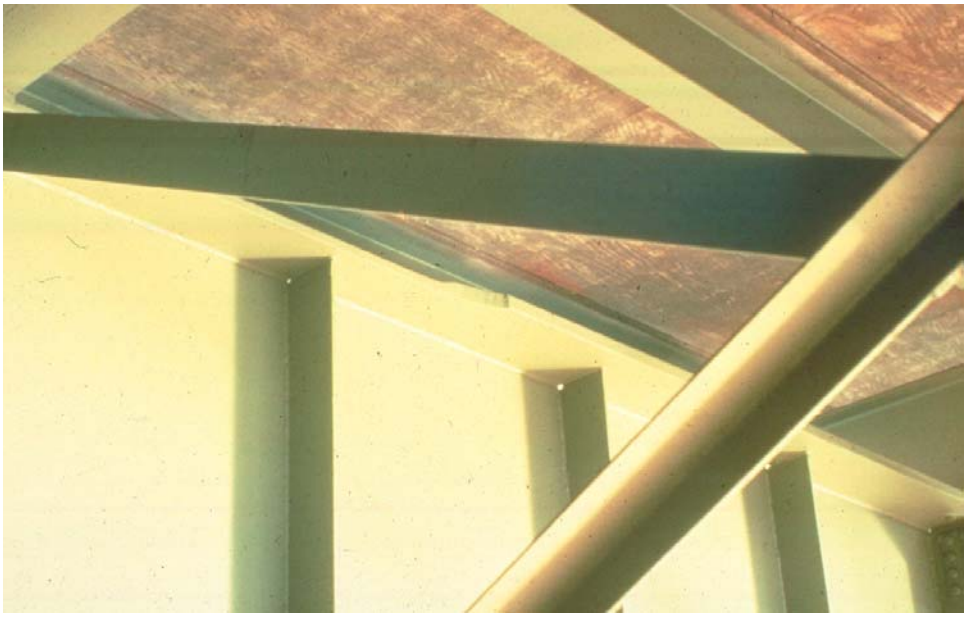


Figure 8.3.20 Web Stiffeners and Welded Flange Splice



Figure 8.3.21 Intersecting Welds



Figure 8.3.22 Tension Zone on a Floorbeam Tie Plate

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

Check the girder webs for cracks at the floorbeam connection plate. Cracks can occur due to out-of-plane distortion of the web, especially if the connection plate is not connected to the top flange. For each crack found, the length, width, orientation and date should be noted on the member and in the inspection notes.

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

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

Out-of-plane Distortion

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

Girder Webs at Floorbeam Connections

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

Sometimes, floorbeam support brackets are welded to the tension flange of the girder (see Figure 8.3.23).

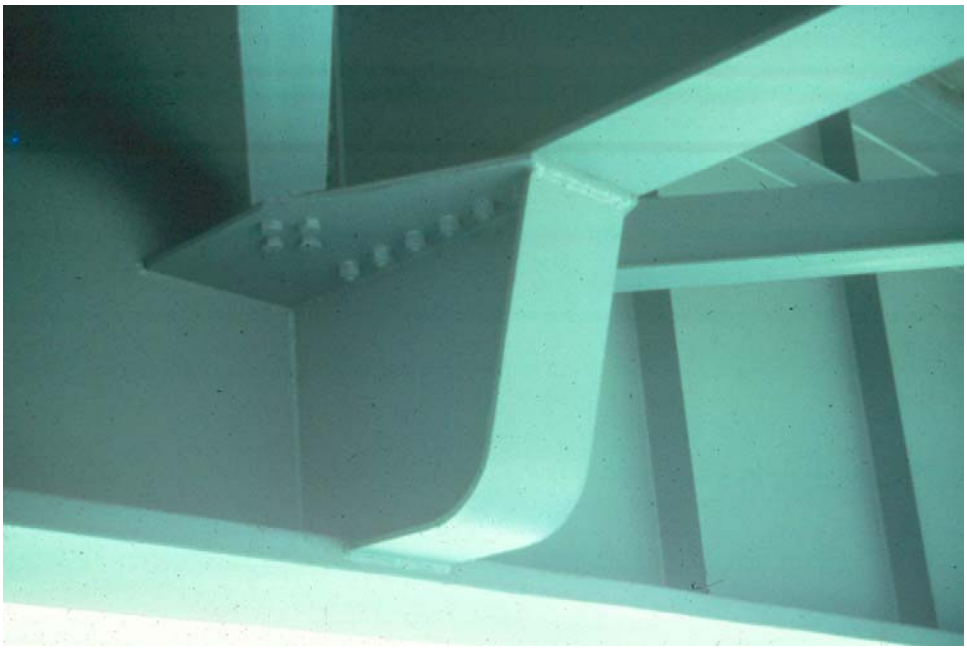


Figure 8.3.23 Floorbeam to Girder Connection

One type of connection detail that has incurred a large number of fatigue cracks is the end of floorbeam connection plates that are not attached to the top tension flange of continuous girder bridges. While the top flange is rigidly embedded in the bridge deck slab, and the connection plate itself is stiff enough to resist rotation and bending from the floorbeam, most of the out-of-plane distortions (perpendicular to the web) concentrate in the local region of the web above the upper end of the connection plate. Fatigue cracks develop in the region as a result of the web plate bending. The cracks are usually horizontal along the web-to-flange welds, and also propagate as an upside-down U along the upper ends of the fillet welds of the connection plate (see Figure 8.3.24). Movement at or near such small cracks often generates oxide powder that combines with moisture to cause apparent bleeding.

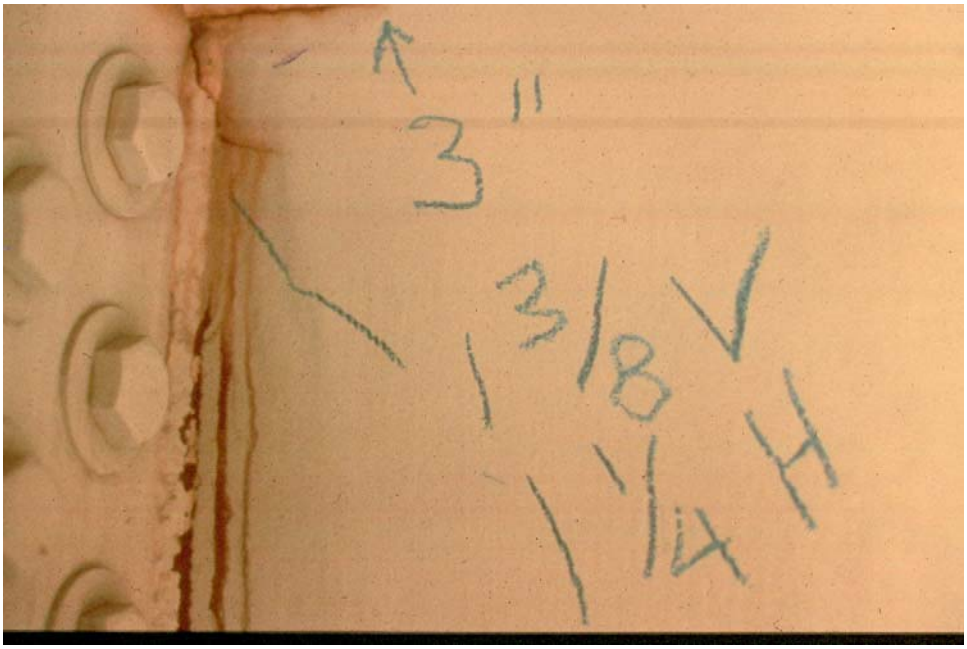


Figure 8.3.24 Cracked Caused by Out-of-plane Distortion

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

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

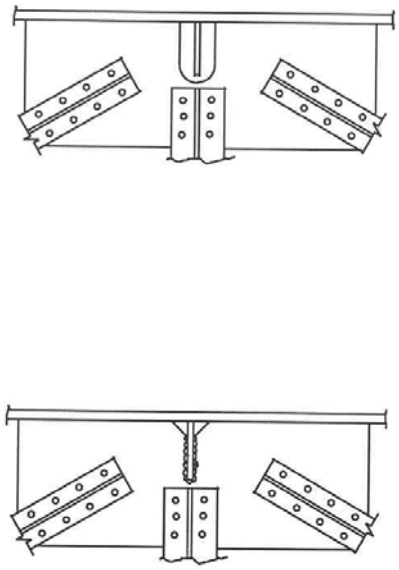


Figure 8.3.25 Lateral Bracing Gusset at Floorbeam or Diaphragm Connection Plate



Figure 8.3.26 Cracked Caused by Out-of-plane Distortion

Lateral Gussets on Plate Girder Webs at Floorbeam Connection

The above photos show examples of lateral gusset out-of-plane bending problems. In addition to possible cracks at the internal gap, the ends of the gusset fillet weld should be equally suspect.

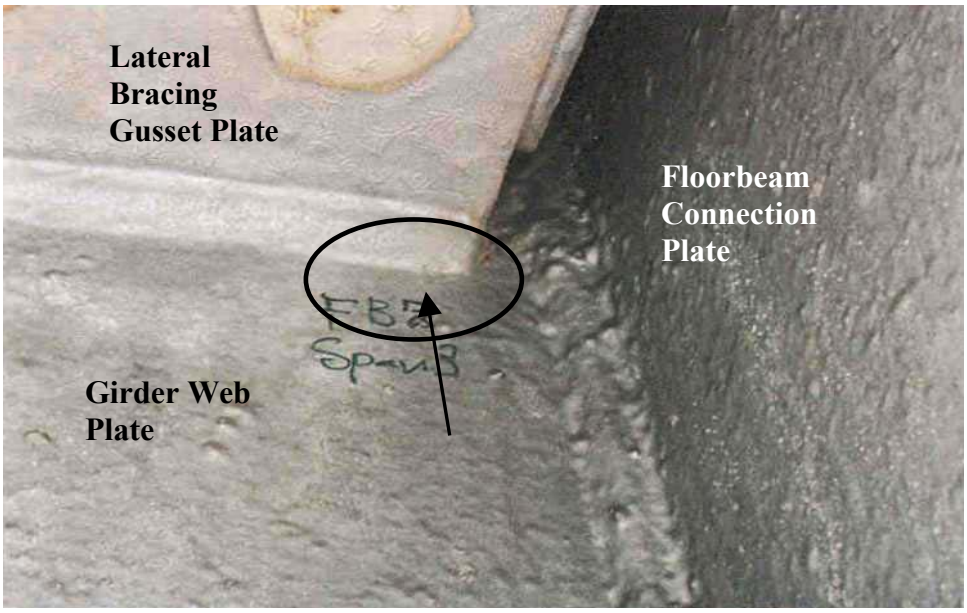


Figure 8.3.27 Crack Caused by Out-of-plane Distortion

Figure 8.3.27 shows another example of a crack in the gap between the lateral bracing gusset and the floorbeam connection plate. In this case, the lateral bracing gusset was bolted to two tabs that were welded to the web on both sides of the connection plate. The crack was very small when detected and the photograph was taken. However, on the opposite side of the web plate, at the elevation of the gusset, a crack more than an inch long was detected along the weld toe of the vertical fillet weld that joins the web and the fascia transverse stiffener in alignment with the floorbeam. This situation of staggered cracks on opposite surfaces of a web plate in a small gap is typical of out-of-plane distortion induced cracks at lateral gusset to floorbeam connection details.

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

Floorbeam and Cantilever Bracket Connection to Girders

In girder and floorbeam bridge systems where the stringers are placed above the floorbeam and the top flanges of the girders are not embedded in the deck slab, serious relative deflections in the direction of the girder could occur at floorbeam and outrigger cantilever bracket connections to the plate girders (see Figure 8.3.28). Fatigue cracks have been detected in the webs of the floorbeams and the cantilever brackets. The situation is not unique to welded floorbeams but also occurs in rolled and built-up members.

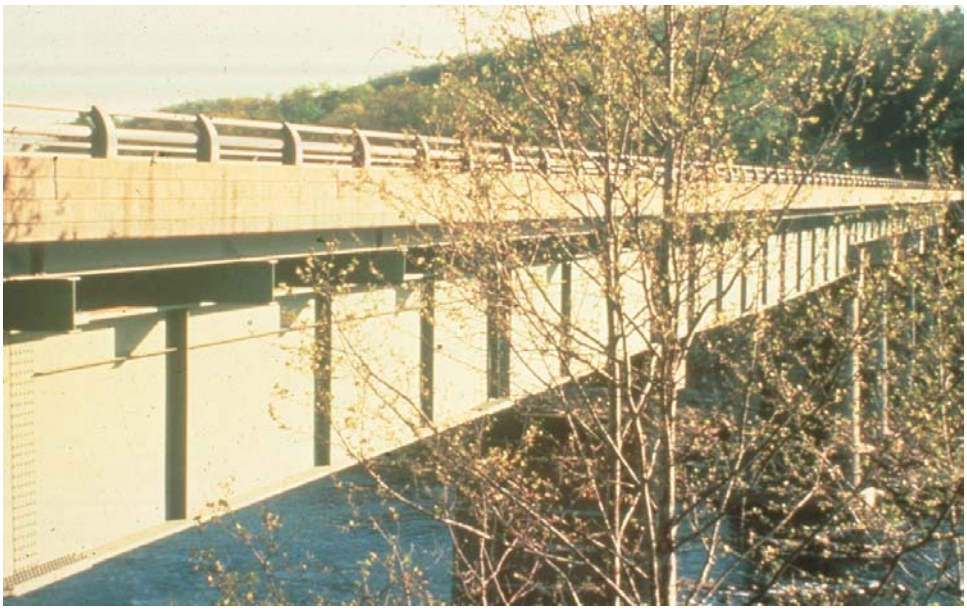


Figure 8.3.28 Cantilever Floorbeam

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

Fracture Critical Members

Since two-girder and through girder bridges are fracture critical, it is important to inspect the main girders thoroughly. Any defects such as cracks, section loss and out-of plane distortions should be measured and documented. All previous reports should be reviewed before performing the inspection to note any areas of particular concern. All reported deficiencies should be checked to ensure no further development has occurred.

8.3.5 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the girder. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Pontis Smart Flags are also used to describe the condition of the steel superstructure, both unpainted and painted.

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

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

The unit quantity for the girder is meters or feet and the total length of all girders must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

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Topic 8.4 Pin and Hanger Assemblies

8.4.1 Introduction

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

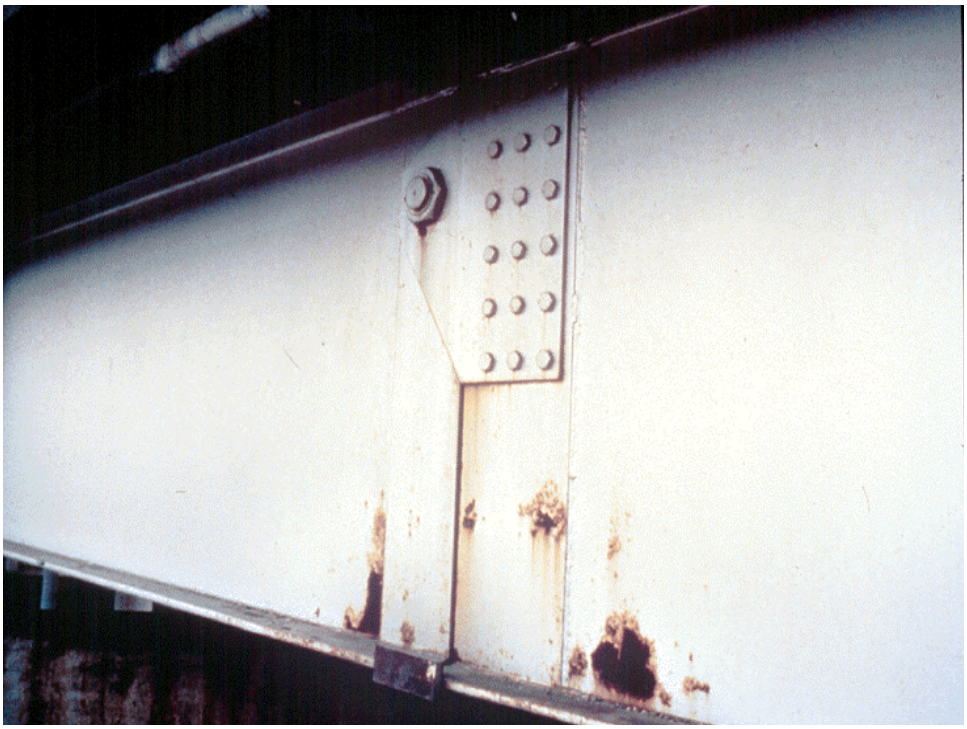


Figure 8.4.1 Single Pin with Riveted Pin Plate



Figure 8.4.2 Typical Pin and Hanger Assembly

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

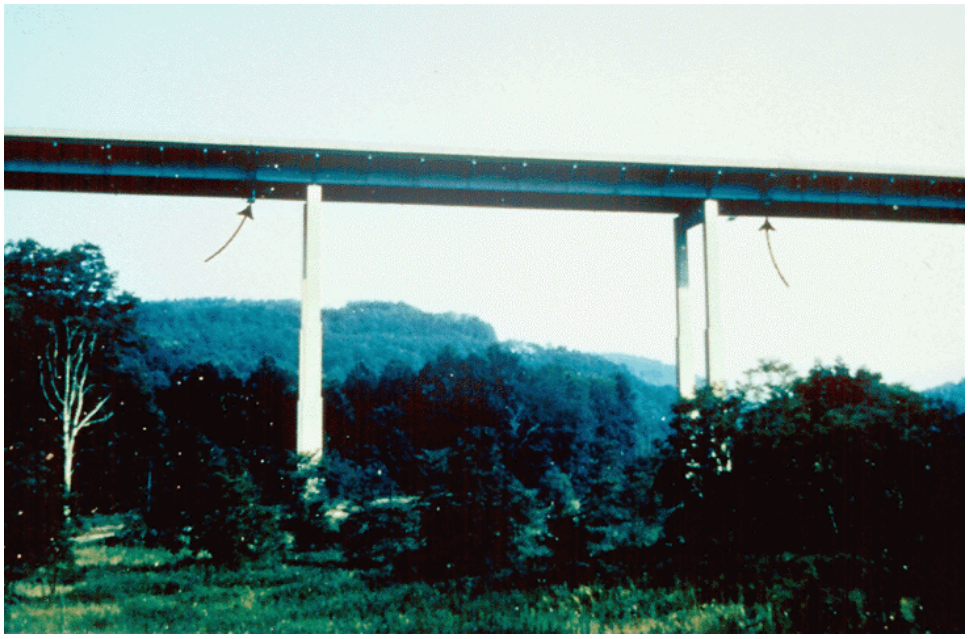


Figure 8.4.3 Pin and Hanger Assembly Locations Relative to Piers

Modern design techniques and computer programs enable the engineer to design multispan bridges without hinges. The problems associated with pin and hanger

details far outweigh any advantages of placing expansion joints away from substructure units.

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

8.4.2 Design Characteristics

Primary and Secondary Elements

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

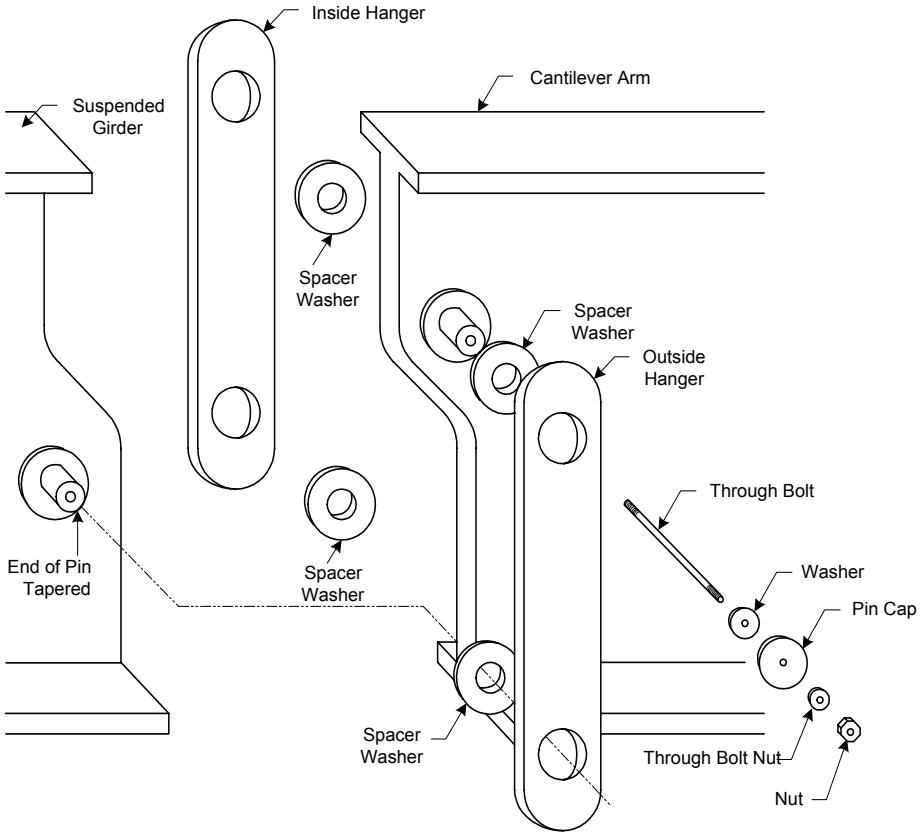


Figure 8.4.4 Pin and Hanger Assembly

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

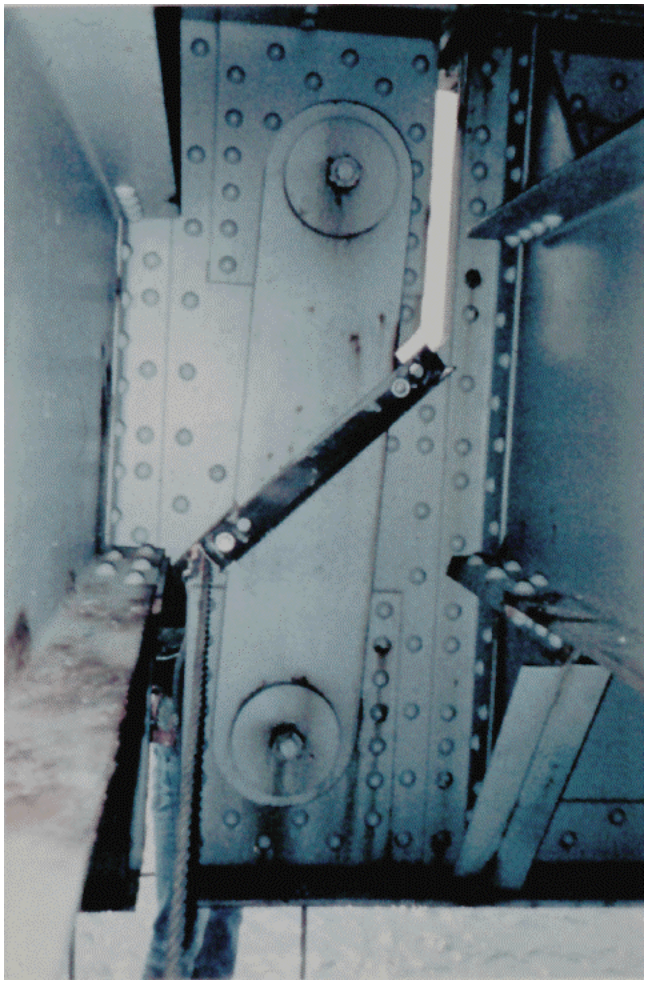


Figure 8.4.5 Pin Cap with Through Bolt

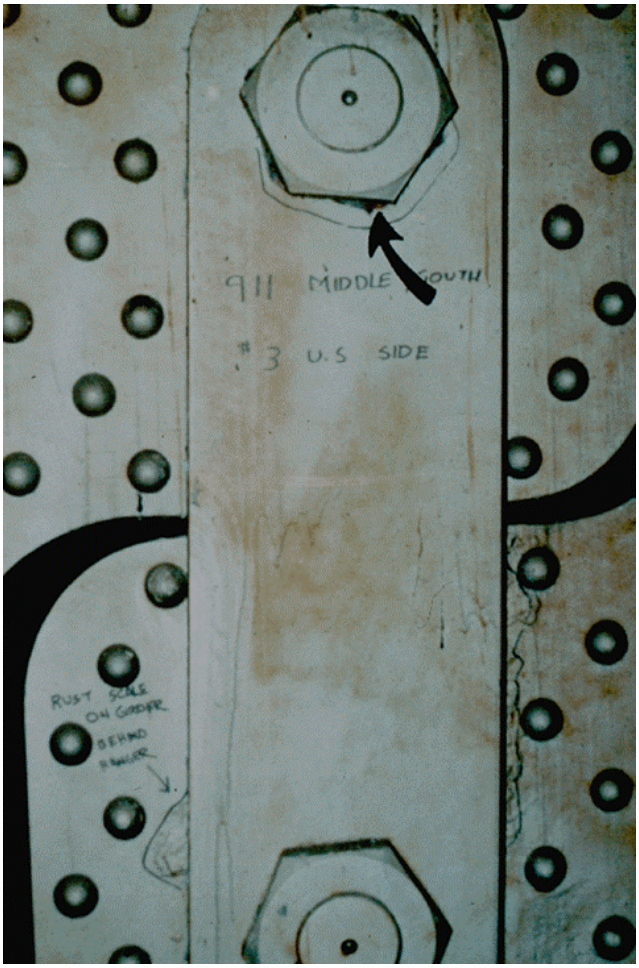


Figure 8.4.6 Threaded Pin with Retaining Nut

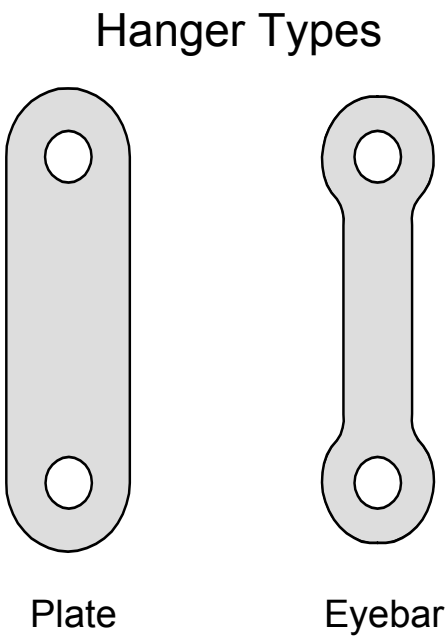


Figure 8.4.7 Plate Type Hanger and Eyebar Shape Link

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

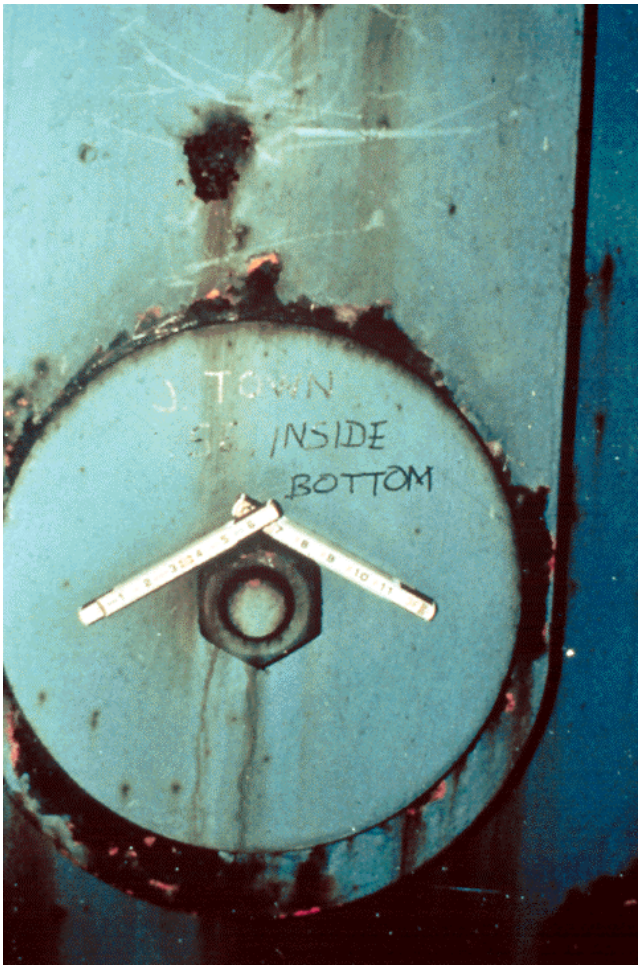


Figure 8.4.8 Pin Cap



Figure 8.4.9 Retaining Nut



Figure 8.4.10 Web Doubler Plates

Forces in a Pin – Design vs. Actual

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

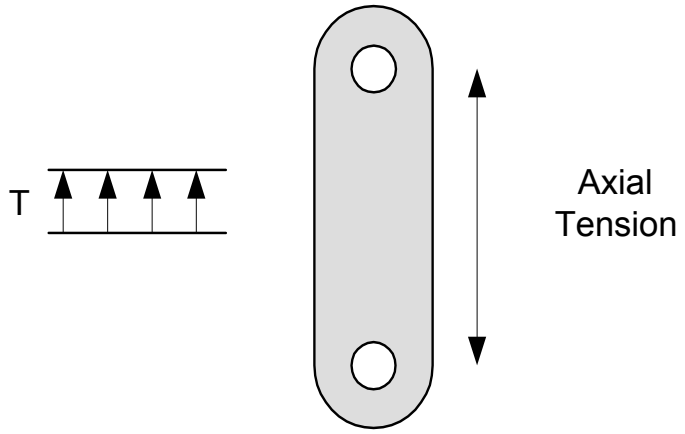


Figure 8.4.11 Design Stress in a Hanger Plate (Tension Only)

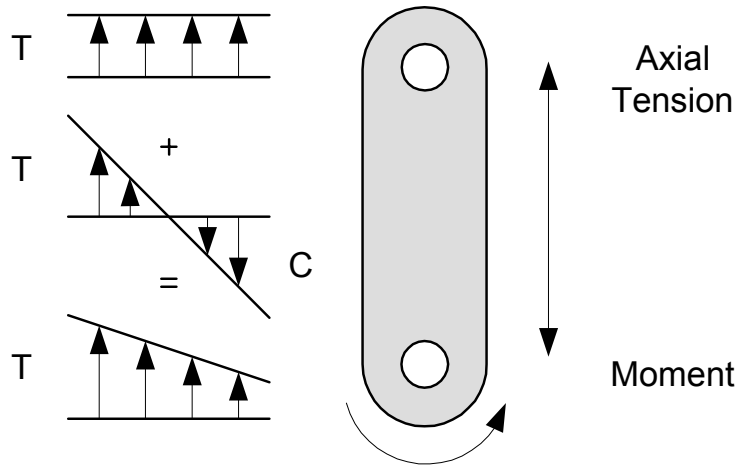


Figure 8.4.12 Actual Stress in a Hanger Plate (Tension and Bending)

Pins are designed for shear and for bearing on the full thickness of the link (see Figure 8.4.13). However, in addition to the designed forces, pins can see very high torsion (twisting) forces if they lose their ability to turn freely (see Figure 8.4.14). Corrosion and rust packing can inhibit or prevent the pins from turning properly. Pins can also be subjected to excessive bearing stress if the link shifts over the pin shoulder (see Figure 8.4.15).

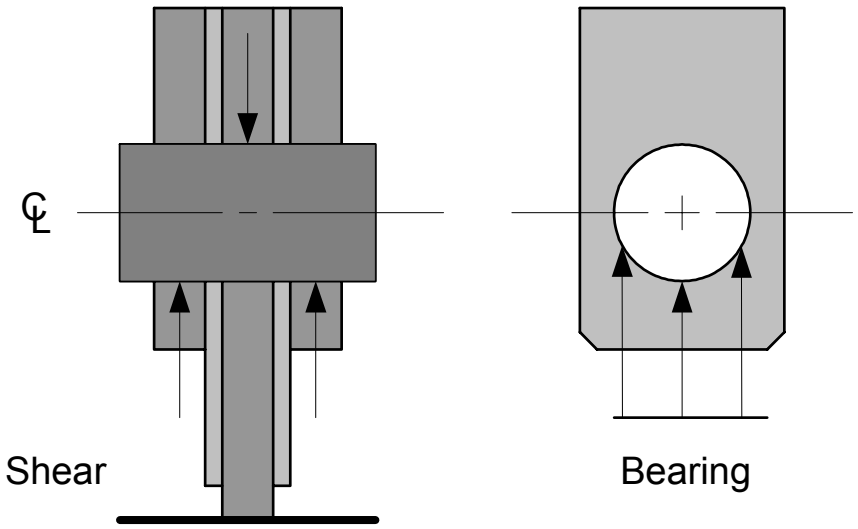


Figure 8.4.13 Design Stress in a Pin (Shear and Bearing)

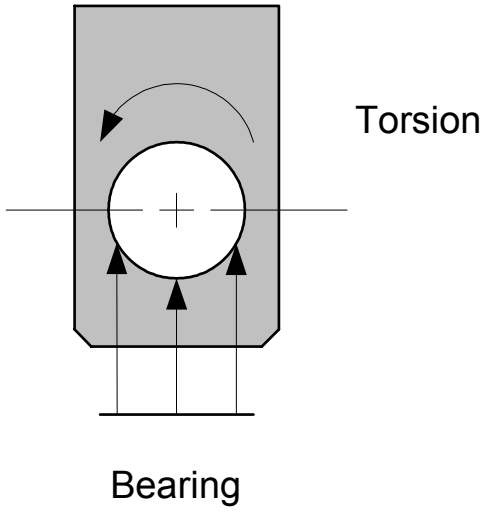


Figure 8.4.14 Actual Stress in a Pin (Shear, Bearing and Torsion)

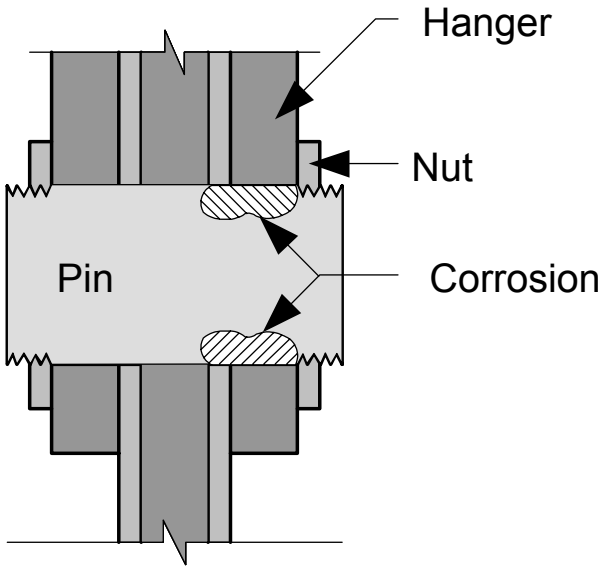


Figure 8.4.15 High Bearing Stress in a Pin due to Corrosion

Fracture Critical Pin and Hanger Assemblies

Pin and hanger assemblies in two-girder bridges are fracture critical members. Failure of one pin or one hanger will cause collapse of the suspended span since there is no alternate load path. The collapse can be catastrophic as demonstrated by the Mianus River Bridge failure shown in Figure 8.4.16. The Mianus River Bridge failed due to the formation of rust between the hangers and the girder webs. As steel rusts, the rust can occupy up to 16 times the original steel volume causing unwanted expansion forces when in a confined space. When rust creates this type of expansion force, bridge inspectors sometimes call it "rust packing". In the case of the Mianus River Bridge, the rust packing pushed the hangers to the ends of the deteriorated pins and the pins eventually failed in bearing.



Figure 8.4.16 Mianus River Bridge Failure

Pin and hanger assemblies in multi-girder structures are not technically fracture critical, since multiple load paths are available. However, they do have the potential for progressive collapse. If all the pin and hanger assemblies at a joint location are frozen and consequently overstressed, the failure of one could cause an adjacent assembly to fail and so on (see Figure 8.4.17).



Figure 8.4.17 Multi-girder Bridge with Pin and Hanger Assemblies

8.4.3 Overview of Common Defects

Common defects that occur on pin and hanger bridges include:

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Tack welds in tension zones
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage

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

8.4.4 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying device after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

In steel pin and hanger bridges as well as other steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel pin and hanger, including:

- Magnetic particle
- Eddy current
- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers

- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Topic 13.3 for Advanced Inspection Techniques.

As mentioned earlier, visual inspection of the pin is not very effective. The majority of the pin is concealed inside the assembly and at best only the surface is available for inspection. Many internal flaws and defects can go undetected if an alternative method for inspection is not used.

Ultrasonic testing is currently the most common means available of checking pins in place (see Figure 8.4.18). For the results to be valid, careful planning and testing by trained individuals is required. For a more detailed look at ultrasonic testing refer to Topic 13.3.



Figure 8.4.18 Ultrasonic Testing of a Pin

Another method for inspecting the pin is to disassemble the pin and hanger unit. However, removal of the retainer nuts or caps should not be attempted unless an alternate means of retaining the hanger on the pin is in place (see Figure 8.4.19).

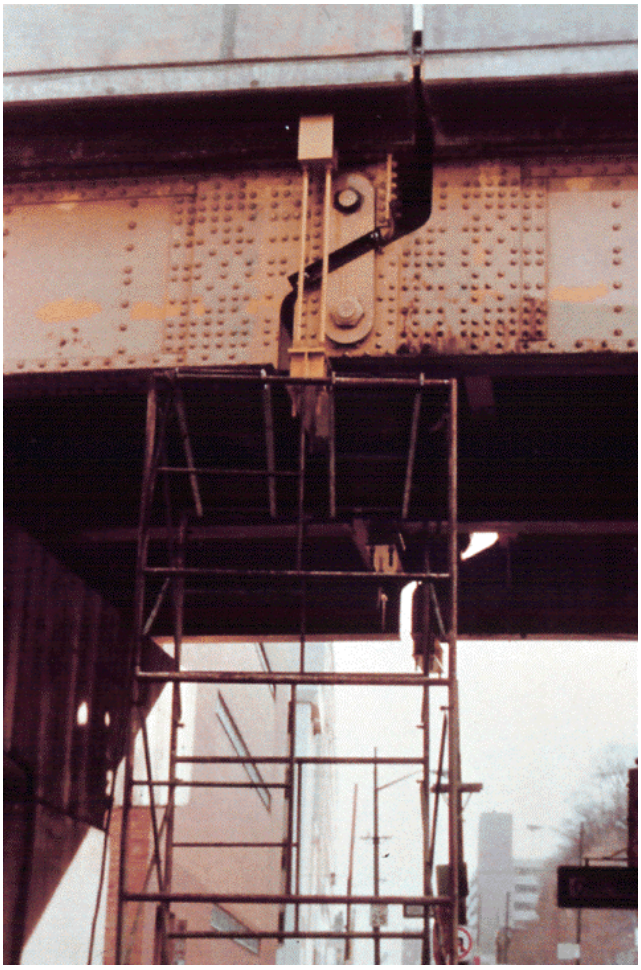


Figure 8.4.19 Alternate Hanger Link Retaining System

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

The bottom line is that disassembly of a pin and hanger joint should be undertaken only after proper engineering design is performed and auxiliary support supplied. It is not a routine bridge inspection procedure.

Locations

General

Pin and hanger connections, when used in suspended span configurations in nonredundant two-girder bridges, are fracture critical. They also occur in cantilevered span arrangements on continuous span structures. This type of connection warrants particular attention. Corrosion can cause fixity at pin and hanger connections. This changes the structural behavior of the connection and is a source of cracking.

Observe and record the general condition of the joint assembly. Check for alignment of the adjacent beam webs and flanges with a straight edge. Note if deck drainage is entering the assembly. Measure the actual dimensions between the pins and also the distance from each pin to the end of the hanger assembly and

compare these values to the as-built dimensions (see Figure 8.4.20).

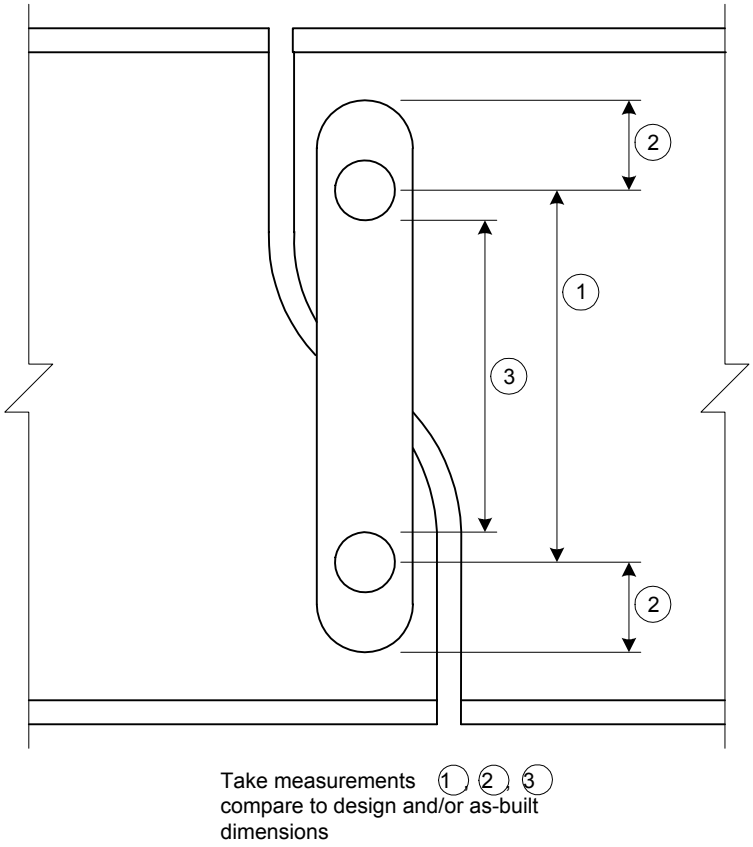


Figure 8.4.20 Pin Measurement Locations

Try to determine if design movement is taking place. Powdery red or black rust where surfaces rub indicates movement (see Figure 8.4.21). It may or may not indicate appreciable section loss. An unbroken paint film across a surface where relative movement should be taking place indicates the joint is frozen.

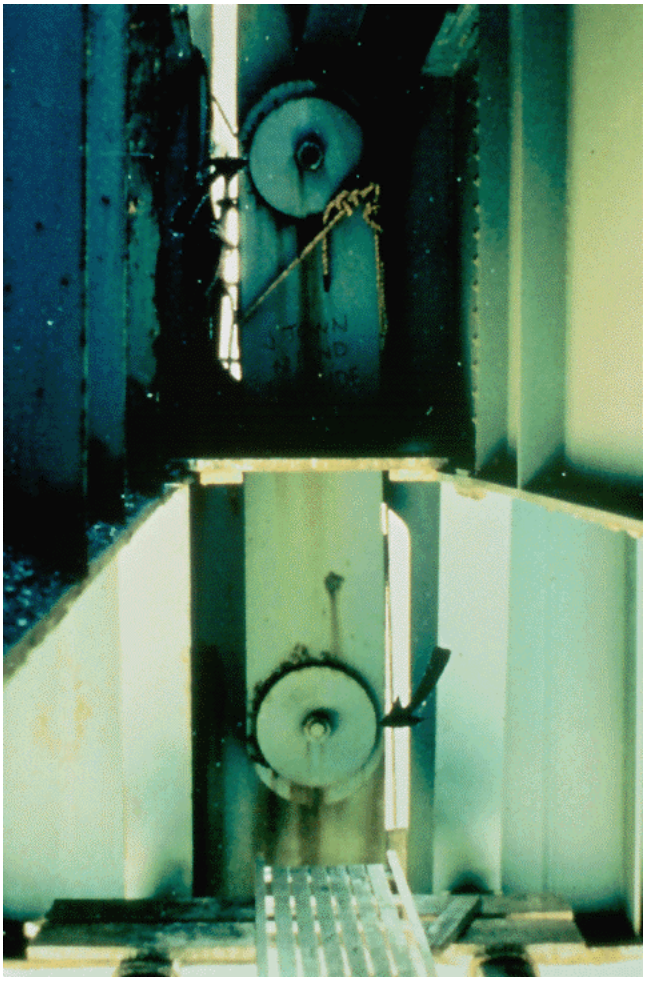


Figure 8.4.21 Rust Stains from Pin Corrosion

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

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

Hangers

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

The hanger plate is as critical as the pin in a pin and hanger connection. It is, however, easier to inspect since it is exposed and readily accessible. Try to determine whether the hanger-pin connection is frozen, as this can induce large moments in the hanger plates.

Examine accessible surface and edges closely for cracks (see Figure 8.4.22). The most critical areas are the ends beyond the pin centerlines and the juncture between the heads and shanks of eyebars. Note surface condition and section loss.

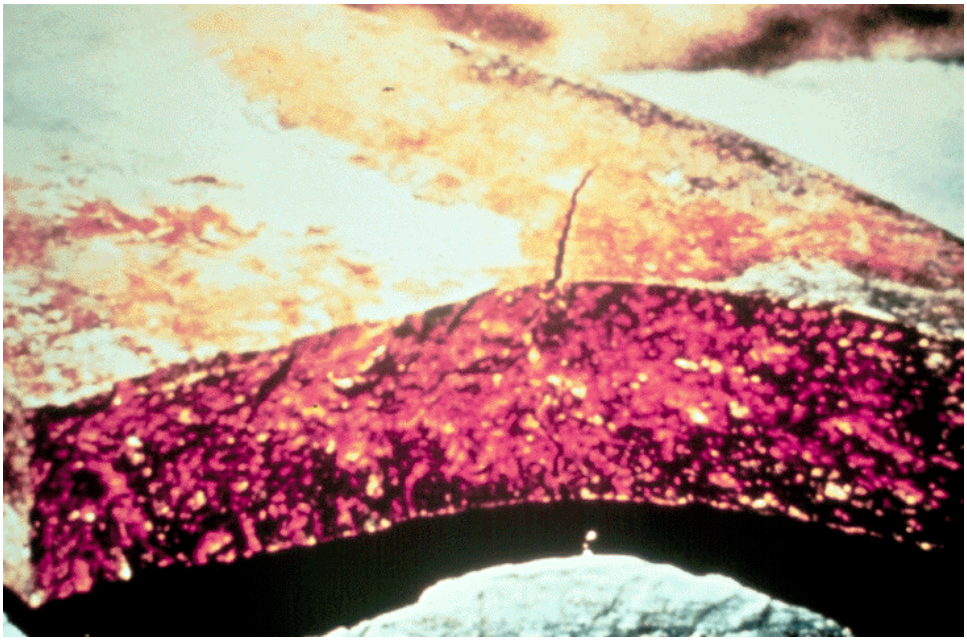


Figure 8.4.22 Close-up of a Crack on a Hanger Plate

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

Examine both sides of the plate for cracks due to bending of the plate from a frozen pin connection. Observe the amount of corrosion buildup between the webs of the girders and the back faces of the plates. Inspect the hanger plate for bowing or out-of-plane distortion from the webs of the girders (see Figure 8.4.23). Any welds should be investigated for cracks. If the plate is bowed, check carefully at the point of maximum bow for cracks that might be indicated by a broken paint film and corrosion.

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

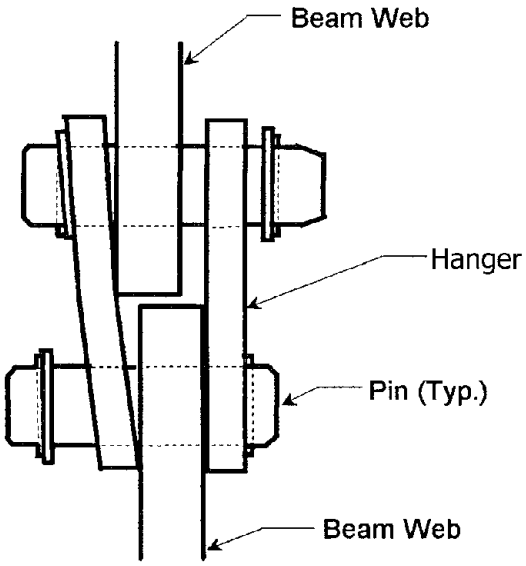


Figure 8.4.23 Bowing Due to Out of Plane Distortion of Hanger

Pins

Rarely is the pin directly exposed in a pin and hanger connection. As a result, its inspection is difficult but not impossible. By carefully taking certain measurements, the apparent wear can be determined. If more than 3 mm (1/8 inch) net section loss has occurred, it should be brought to the attention of the bridge engineer in charge of that structure at once (see Figure 8.4.24). In a girder pin and hanger, wear to the pins and hangers will generally occur in two locations: at the top of the pin and top of the hanger on the cantilevered span and at the bottom of the pin and the bottom of the hanger on the suspended span. Sometimes wear, loss of section, or lateral slippage may be indicated by misalignment of the deck expansion joints or surface over the hanger connection. When inspecting a girder pin and hanger, locate the center of the pin, measure the distance between the center of the pin and the end of the hanger, and compare to the plan dimensions, if available. Remember to allow for any tolerances since the pin was not machined to fit the hole exactly. Generally, this tolerance will be 1 mm (1/32 inch). If plans are not available, compare to previous measurements. The reduction in this length will be the apparent wear on the pin.

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

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



Figure 8.4.24 Corroded Pin and Hanger Assembly

Bearing Areas

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

Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion.

Retrofits

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

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

The first two (rod & saddle and underslung catcher), are added to the structure and only carry load if the pin or hanger in a joint fails (see Figure 8.4.25). The gap between the "catcher" and the girder must be kept as small as possible to limit impact loading. If it is too tight, however, joint movement may be restrained. A neoprene bearing may be included in the assembly to lessen impact. The inspector should find out what the relative positions of the components should be by design and measure the critical points in the field for comparison.

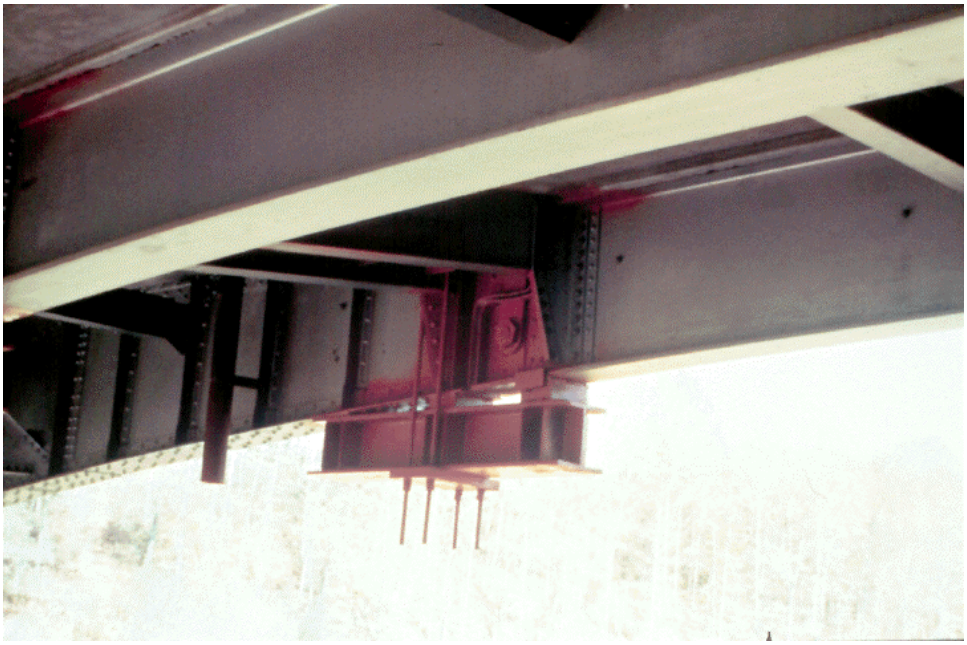


Figure 8.4.25 Underslung Catcher Retrofit

The seated beam and continuity retrofits completely replace the pin and hanger assemblies. Vacant pin holes may be left under some schemes. Inspection of these details should be the same as inspection at field splices and bearings. Additional deflections may be introduced into piers and more movements may take place at expansion bearings when continuity is introduced. The areas should therefore receive extra attention.

Sometimes a pin and hanger assembly is retrofitted by using a bolted field splice. This is done only after a structural engineer analyzes the bridge to determine if the members can support continuous spans instead of cantilevered spans. The inspector must remember to inspect both the positive and negative moment regions of the superstructure.

8.4.5 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of pin and hanger assemblies. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the pin and hanger assembly. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value.

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

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

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

In some states, Element Level Smart Flags are also used to describe the condition of the pin and hanger assembly, both unpainted and painted. For a deficient pin and hanger assembly, the "Frozen and Deformed Pin and Hanger Assembly" Smart Flag may be used.

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Topic 8.5 Steel Box Girders

8.5.1 Introduction

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

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



Figure 8.5.1 Simple Span Box Girder Bridge

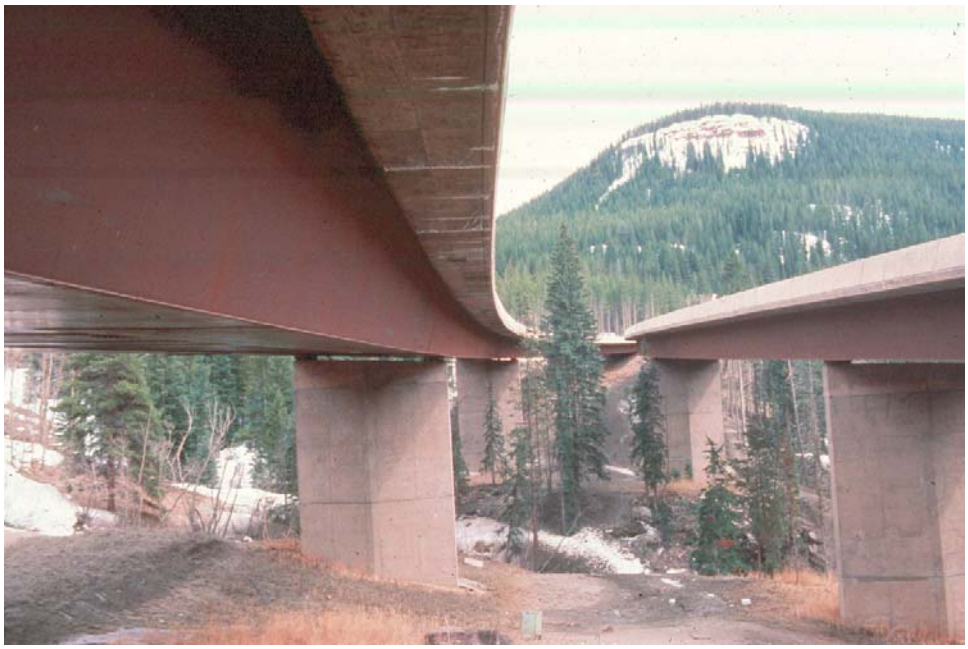


Figure 8.5.2 Curved Box Girder Bridge

8.5.2 Design Characteristics

Configuration

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



Figure 8.5.3 Box Girders With Cantilevered Supports for Deck



Figure 8.5.4 Spread Box Girders

Primary and Secondary Members

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



Figure 8.5.5 Diaphragms – K Bracing and Plate

Function of an Internal Stiffener

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



Figure 8.5.6 External Diaphragm



Figure 8.5.7 Box Girder Access Door

Fatigue Prone Details

Some common areas for fatigue prone details are:

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

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

Fracture Critical Areas

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

Deck Interaction

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

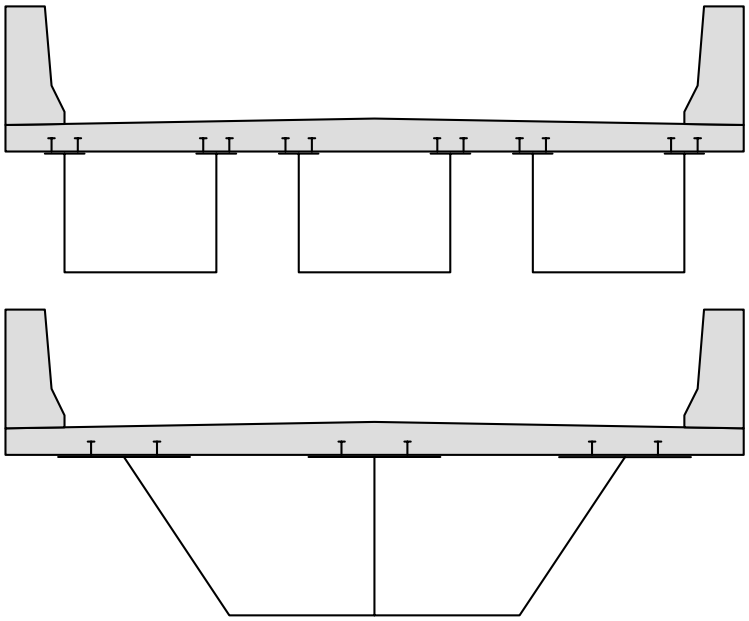


Figure 8.5.8 Box Girder Cross Section with Composite Deck

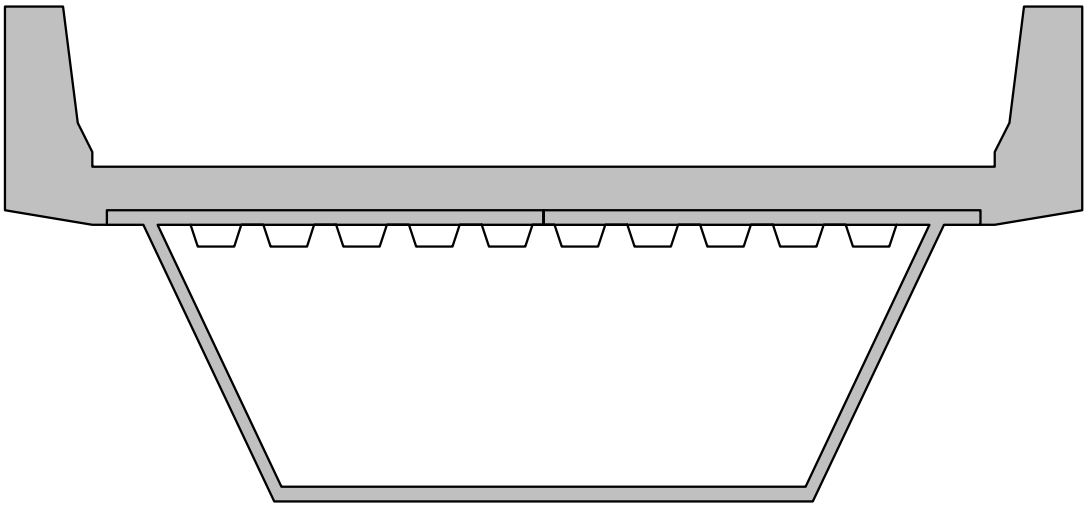


Figure 8.5.9 Box Girder Cross Section with Orthotropic Steel Plate Deck

8.5.3 Overview of Common Defects

Common defects that occur on steel box girder bridges are:

- Failure of the Paint System
- Pitting
- Surface Rust
- Section Loss
- Tack Welds in tension zones
- Fatigue Cracking
- Collision Damage
- Overload Damage
- Heat Damage

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

8.5.4 Inspection Locations and Procedures

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

Locations

Bearing Areas

Examine the web areas over the supports for cracks, section loss and buckling. If bearing stiffeners, jacking stiffeners and diaphragms are present at the supports inspect them for cracks, section loss and buckling also.
Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion.

Shear Zones

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



Figure 8.5.10 Box Girder Shear Zone

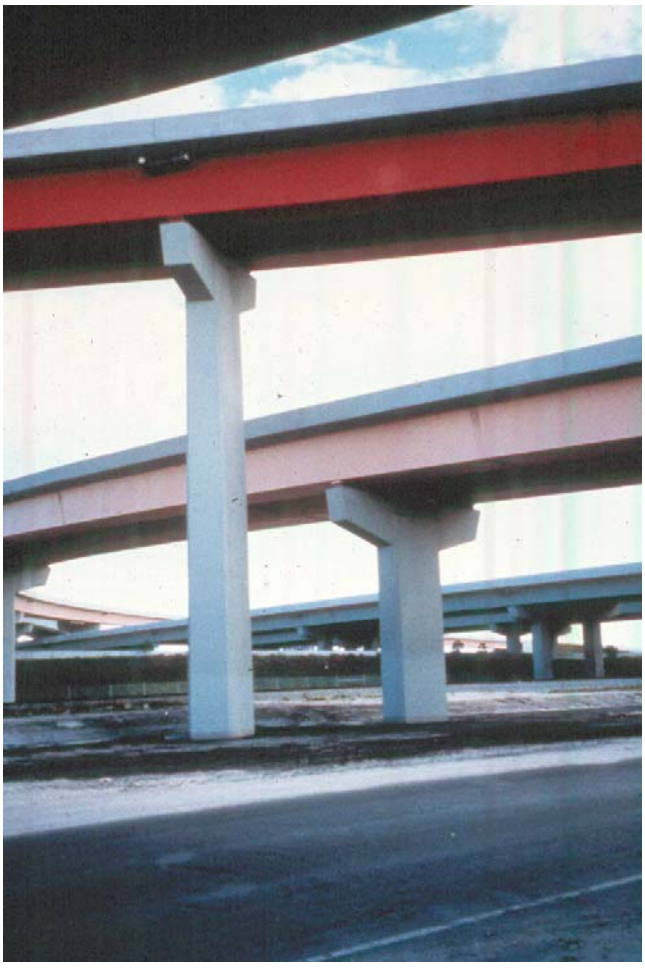


Figure 8.5.11 Continuous Box Girders

Flexure Zones

The flexure zone of a continuous box girder includes its entire length between and over the supports (see Figure 8.5.12). Inspect the tension and compression flange areas for corrosion and loss of section. Examine the flange in high stress areas for flexure and torsion-related damage caused by live loads. Similar to other steel superstructure types, investigate the negative moment regions for flexural damage. Inspect the diaphragms for distortion caused by torsional stresses or differential deflection.



Figure 8.5.12 Box Girder Flexural Zone

Secondary Members

Examine the diaphragm connections for loose fasteners and cracked welds. Check for buckled diaphragms.

Areas That Trap Water and Debris

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

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

Areas Exposed to Traffic

Check the box girder for signs of collision damage. Loss of section, cracking, and

any shape distortion should be carefully documented.

Fatigue Prone Details

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

Check all welds and welded attachments inside the box. This includes web stiffeners, flange stiffeners, diaphragms, lateral bracing, and stay-in-place deck panels. Stiffener weld terminations in tension zones are a special concern. Butt welds joining adjacent longitudinal stiffeners on the bottom flange serve as potential sources for cracks to propagate into the bottom flange. Check all intersecting welds between the webs and flanges, particularly in tension areas. Check field splice areas, especially where stiffeners are welded to the web or bottom flange (see Figure 8.5.13).



Figure 8.5.13 Field Splice

Check for fatigue cracks due to web-gap distortion as discussed in Section 8.2.4.

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

Back-up bars were often spliced with non-NDT-inspected butt welds. These welds may be flawed and deserve special care in inspection if present in tension areas (see Figure 8.5.14).

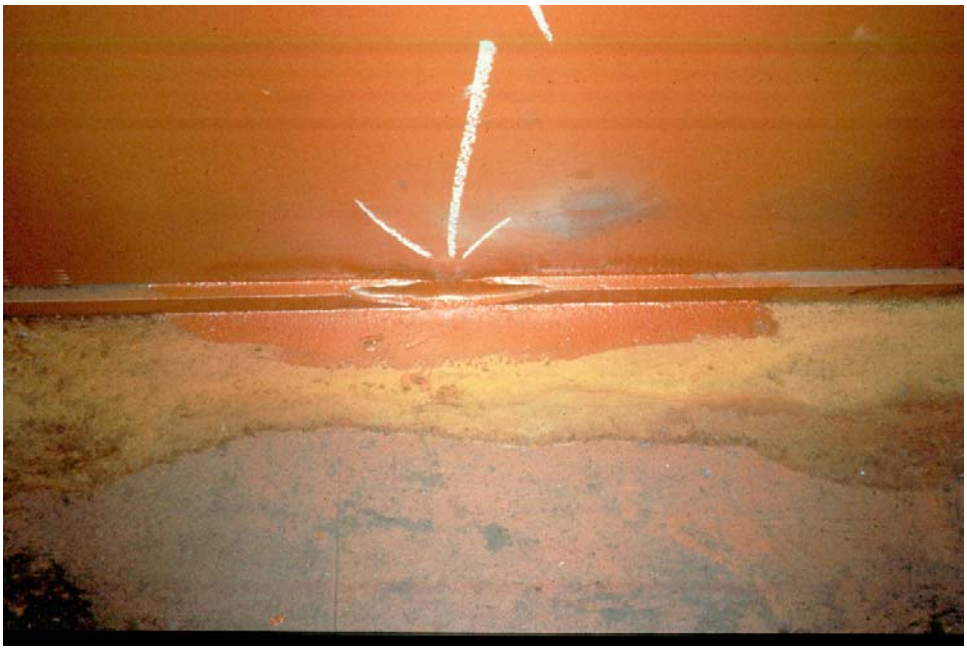


Figure 8.5.14 Butt Welds in Back-up Bars Ground Out as Retrofit

Fracture Critical Members

The redundant nature of a box girder bridge depends primarily on the number of box girders in the span. If two or less box girders are used, then the structure is considered nonredundant and the box girders are fracture critical members (see Figure 8.5.15). If three or more box girders are used, then the structure is generally considered redundant (see Figure 8.5.16). However, if the spacing of the box girders is large, the structure may not be redundant. A structural engineer should determine the fracture critical nature of the member.



Figure 8.5.15 Non-redundant Box Girder Bridge



Figure 8.5.16 Redundant Box Girder Bridge

Out-of-Plane Distortions

Webs of box girders at unattached ends of diaphragm connection plates are susceptible to the same kind of out-of-plane distortions and fatigue cracking

and fatigue cracks in the web at the ends of unattached connection plates will occur at more diaphragms in box girder than in plate girder bridges.



Figure 8.5.17 Box Girder Internal Diaphragm Not Attached to Flange

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

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

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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 the re-examination of the actual conditions can be made.

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

8.5.5 Advanced Inspection Techniques

In steel box girders as well as other steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the box girder, including:

- Magnetic induction
- Electrical resistivity
- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Topic 13.3 for Advanced Inspection Techniques.

8.5.6 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

Application of Condition State Assessment (Element Level)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the girder. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element Level Smart Flags are also used to describe the condition of the steel superstructure, both unpainted and painted.

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

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

The unit quantity for the girder is meters or feet and the total length of all box girders must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

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Topic 8.6 Trusses

8.6.1 Introduction

Metal truss bridges have been built since the early 1800's. They can be thought of as a deep girder with the web cut out. They are also the only bridge structure made up of triangles. The original metal trusses were made of wrought iron, then cast iron, then steel. When trusses were first being built of metal, material costs were very high and labor costs were low. Because trusses were made up of many short pieces, it was cost effective to build the members in the shop and assemble them at the site. Today the higher costs of labor and the lower costs of material have limited the use of trusses to major river crossings.

8.6.2 Design Characteristics

The superstructure of a truss bridge usually consists of two parallel trusses (see Figure 8.6.1). The trusses are the main load-carrying members on the bridge. There are three types of trusses, grouped according to their position relative to the bridge deck (see Figure 8.6.2):



Figure 8.6.1 Single Span Truss

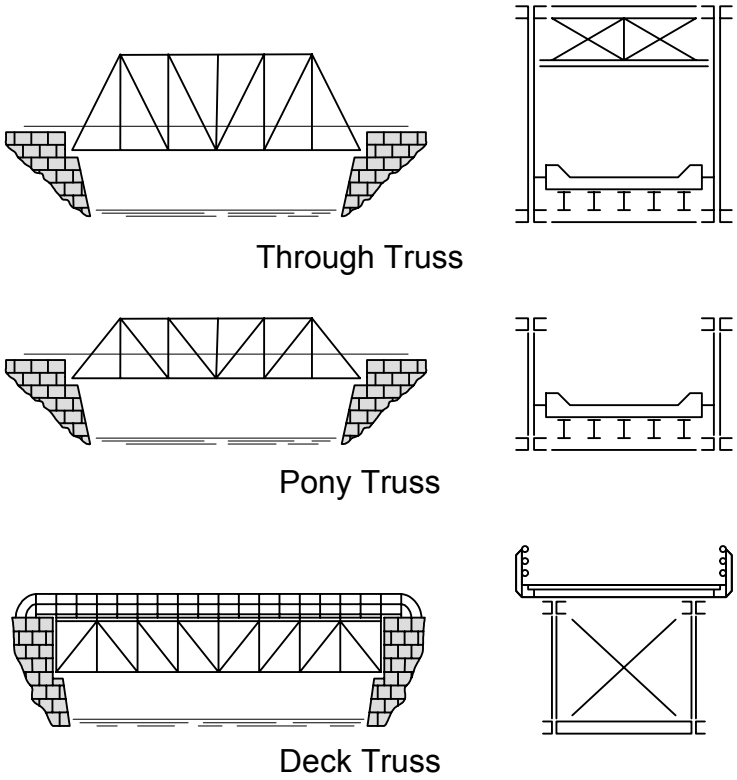


Figure 8.6.2 Through-Pony-Deck Truss Comparisons

Through Truss

On a through truss, the roadway is placed between the main members (see Figure 8.6.3). Through trusses are constructed when underclearance is limited.

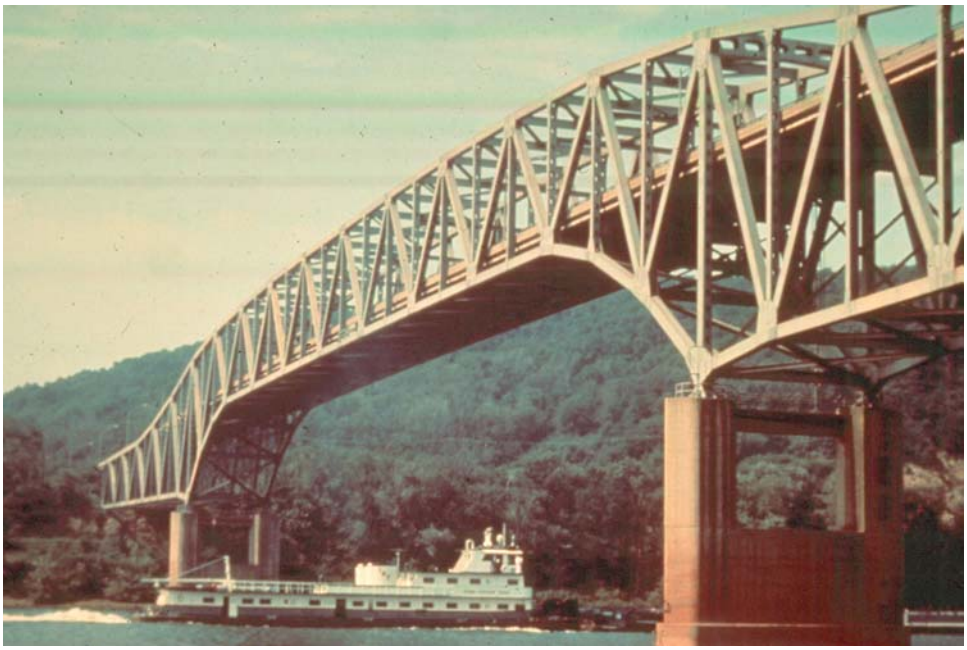


Figure 8.6.3 Through Truss

Pony Truss

A pony or "half-through" truss (see Figure 8.6.4) has no overhead bracing members connecting the two trusses. The vertical height of the pony truss is much less than the height of a through truss. Today, pony trusses are seldom built, having been replaced by the multi-beam bridge.



Figure 8.6.4 Pony Truss

Deck Truss

On a deck truss (see Figure 8.6.5), the roadway is placed on top of the main members. Deck trusses have unlimited vertical and horizontal clearances and can readily be widened. For these reasons, they are preferred over through trusses when under-clearance is not a concern.



Figure 8.6.5 Deck Truss

Other Applications

Trusses are generally considered to be main members. However, they are also used as floor systems in arches and as stiffening trusses in suspension bridges and arch bridges (see Figures 8.6.6 and 8.6.7). Trusses are also commonly used for movable bridge spans because they are lightweight and have higher overall stiffness (see Figure 8.6.8). Even towers are sometimes braced with web members, as a truss.



Figure 8.6.6 Suspension Bridge with Stiffening Truss



Figure 8.6.7 Arch Bridge with Stiffening Truss



Figure 8.6.8 Vertical Lift Bridge

Design Geometry

Bridge engineers have used a variety of arrangements in the design of trusses. Many of the designs were patented by and named after their inventor. One characteristic that all bridge trusses have in common is that the arrangement of the truss members forms triangles (see Figure 8.6.9).

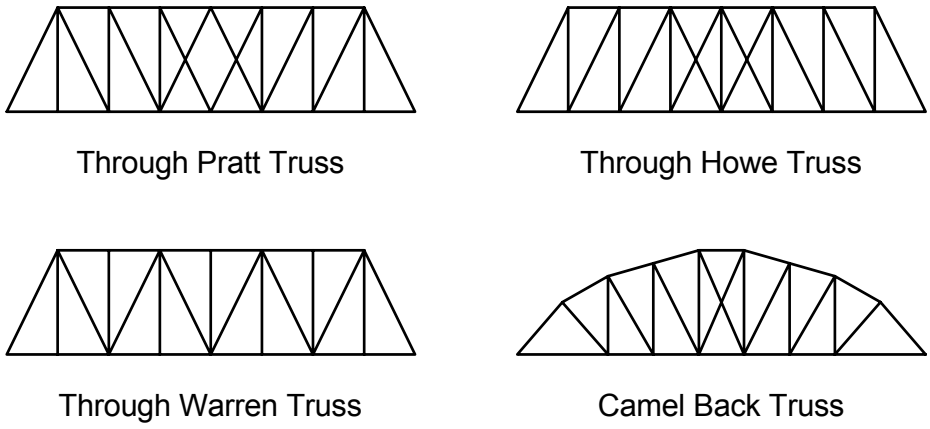


Figure 8.6.9 Various Truss Designs

Trusses have been constructed for short to very long spans, using simple, multiple, cantilever, and continuous designs (see Figure 8.6.10 to Figure 8.6.14). Cantilevered trusses often incorporate a "suspended" or "drop-in" span between two cantilever spans (see Figures 8.6.15, 8.6.16 and 8.6.17). The suspended span behaves as a simple span and is connected to cantilevered spans with pins or pin and hanger connections. The back span on a cantilever truss is called the anchor span.



Figure 8.6.10 Single Span Pony Truss



Figure 8.6.11 Single Span Through Truss



Figure 8.6.12 Multiple Span Pony Truss



Figure 8.6.13 Multiple Span Through Truss



Figure 8.6.14 Continuous Through Truss



Figure 8.6.15 Cantilever Through Truss



Figure 8.6.16 Continuous Deck Truss



Figure 8.6.17 Cantilever Through Truss

As stated earlier, a truss can be thought of as a very deep girder with portions of the web cut out. Truss members are divided into three groups: (see Figure 8.6.18)

- Top chord members
- Bottom chord members
- Web members (diagonals and verticals)

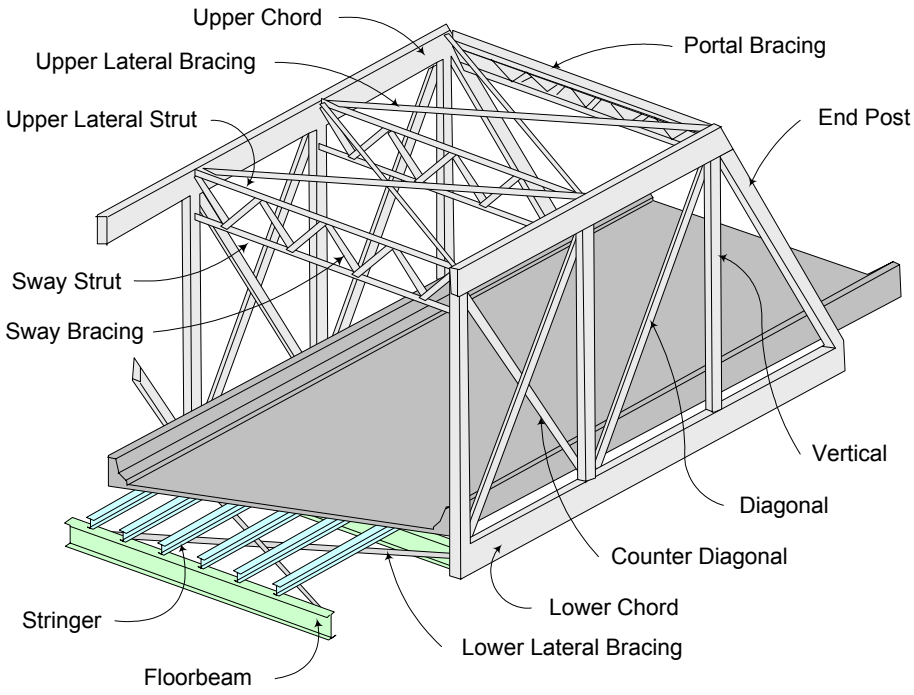


Figure 8.6.18 Truss Members and Elements

Truss members are fabricated from eyebars, rolled shapes, and built-up members (see Figure 8.6.19). Built-up sections are desirable for members that carry compression because they can be configured to resist buckling (see Figure 8.6.20). Structural tubing and fabricated box sections are popular for modern trusses because they provide a "clean" look and are easier to maintain.

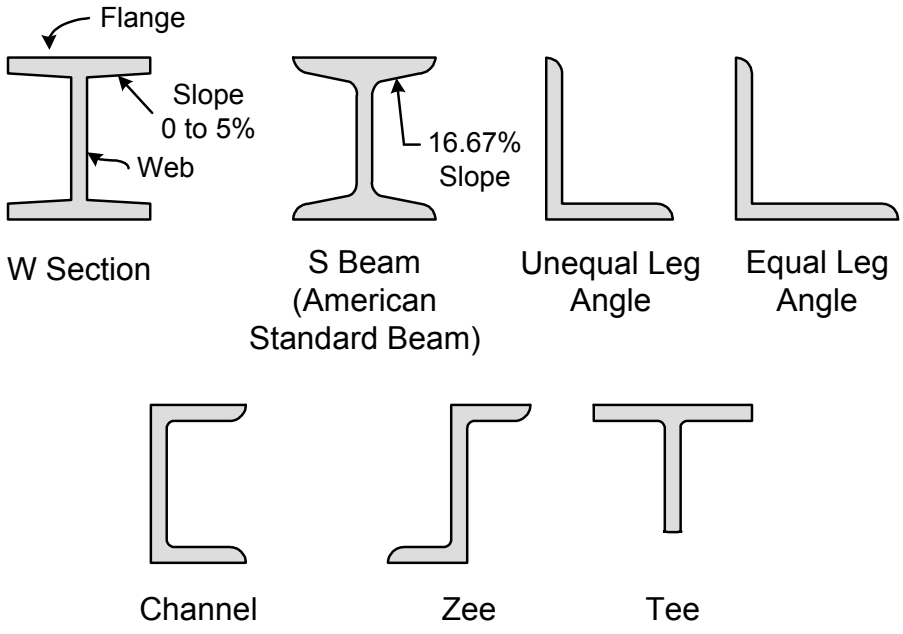


Figure 8.6.19 Rolled Steel Shapes

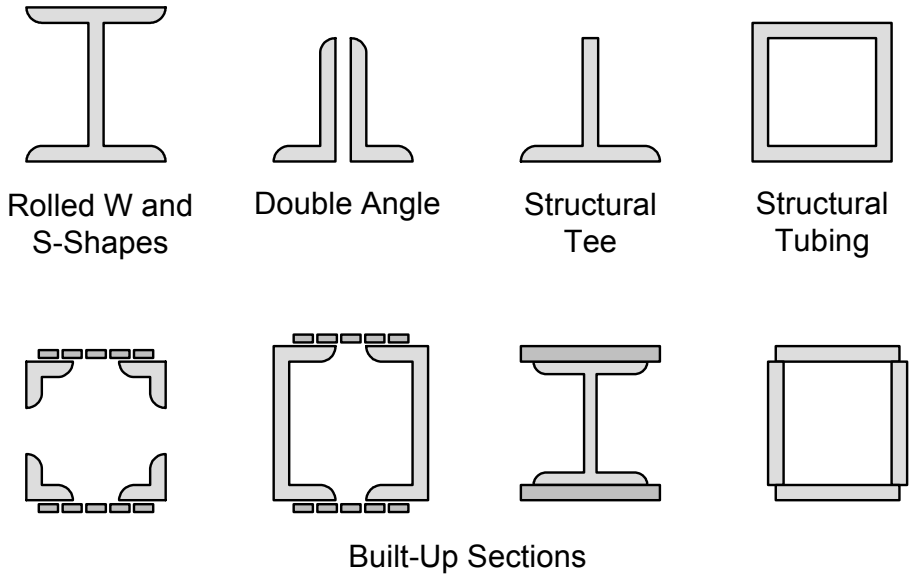


Figure 8.6.20 Typical Compression Members

Chord Members

Trusses, like beams and girders, support their loads by resisting bending. As the truss bends, the chord members behave like flanges of a beam and carry tension or compression forces (see Figure 8.6.21). On a simple span truss, the bottom chord is always in tension, while the top chord is always in compression. The diagonally sloped end post is a chord member. Top chords are also known as upper chords (U), and bottom chords are referred to as lower chords (L) (see Figure 8.6.22).

As truss bridge spans increase, cantilever and continuous designs are used, creating negative moment regions. Therefore, over an intermediate support, the top chord of a truss, like the top flange on a girder, is in tension (see Figure 8.6.21). The negative moment regions produce very large moments. It is common to find varying depth trusses on large complex structures, with the greatest depth at the supports where the moments are the largest (see Figure 8.6.23).

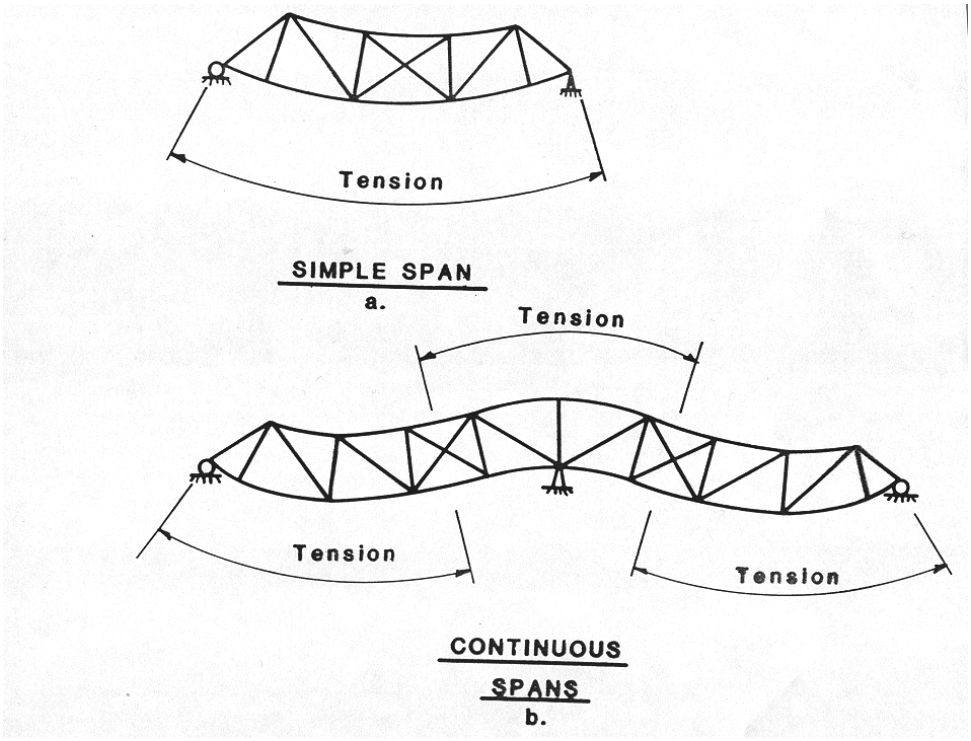


Figure 8.6.21 Axial Loads in Chord Members



Figure 8.6.22 Single Span Through Truss – General Elevation View



Figure 8.6.23 3 Span Continuous Through Truss

Web Members

The web members are typically connected to the top chord at one end and to the bottom chord at the other end. All trusses will have diagonal web members, and most trusses will also have vertical web members. Depending on the truss design, a web member may be in tension, compression, or may be subjected to force reversal and carry either type of stress for different loading conditions.

- Diagonals

For simple spans an easy method to determine when a truss diagonal is in tension or compression is to use the "imaginary cable - imaginary arch" rule (see Figure 8.6.24). Diagonals that are symmetrical about midspan and point upward toward midspan, like an arch, are in compression. Diagonals that are symmetrical about midspan and point downward toward midspan, like a cable, are in tension. This rule applies only to simple span trusses.

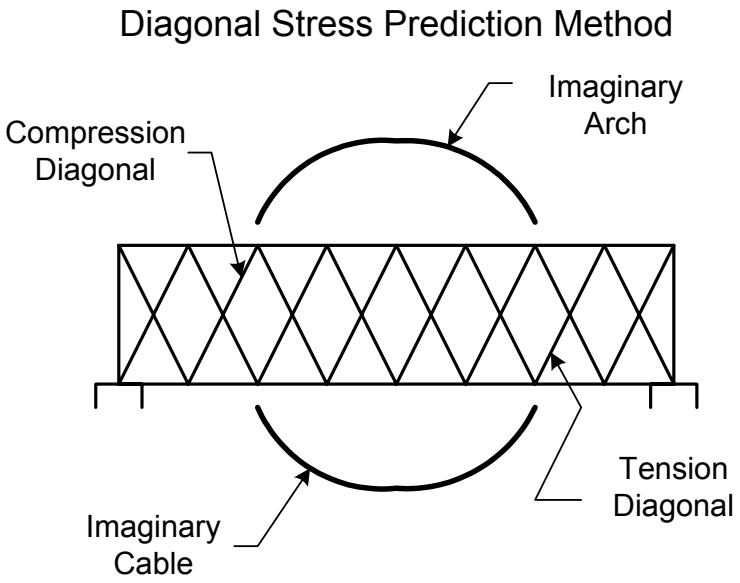


Figure 8.6.24 "Imaginary Cable – Imaginary Arch"

A tension diagonal can be referred to as a tie, while a compression diagonal is often called a strut or brace.

On older simple span trusses, it is easy to determine which members are in tension and which are in compression. The design of a 7.6 m (25-foot) tension member, subjected to a load, will require a much smaller member (cross section) than a 7.6 m (25-foot) compression member subjected to the same load. On older pin-connected trusses, compression members are always the larger built-up members as compared to the tension members, which were often eyebar members. The Pratt truss, with all its diagonals in tension, quickly replaced the Howe truss, whose diagonals are in compression. The Pratt truss was a lighter and therefore cheaper truss.

For older trusses, counters are tension-resisting diagonals installed in the same panel in which the force reversal occurs. They are oriented opposite from each other, creating an "X" pattern.

Counters are stressed only under live loads. On older bridges on which counters are bar shaped, they should be capable of being moved by hand during an inspection. Counters are found on most old trusses but rarely on new trusses.

With more complex truss designs (continuous and cantilever), the diagonal web members must be capable of withstanding both tension and compression. This is known as force reversal, and it is one of the reasons that, on many modern truss bridges, the appearance of the tension and compression diagonals is almost identical.

As trusses become longer and, more importantly, as live loads become larger, the forces in some diagonals on a bridge continually change from tension to compression and back again. This situation was more likely to occur near the center of the trusses. This is because the positive dead load shear decreases towards the center. At this location, the live load can cause a large enough

negative shear to overcome the positive dead load shear, resulting in compressive forces for the diagonal. Stress reversal of diagonals also occurs in the panels adjacent to the inflection points in continuous truss spans.

See Figure 8.6.18 of a sample truss schematic showing truss diagonals.

- Verticals

There is also an easy method to determine when a vertical member is in tension or compression for a simply supported truss. Verticals that have one diagonal at each end are opposite to the force of the diagonals (see Figure 8.6.25). Verticals that have two diagonals at the same end are similar to the force in the diagonal closest to midspan (see Figure 8.6.26). Verticals that have counters on both ends are in compression (see Figure 8.6.27).

A vertical compression member is commonly called a post or column, while a vertical tension member is sometimes called a hanger.

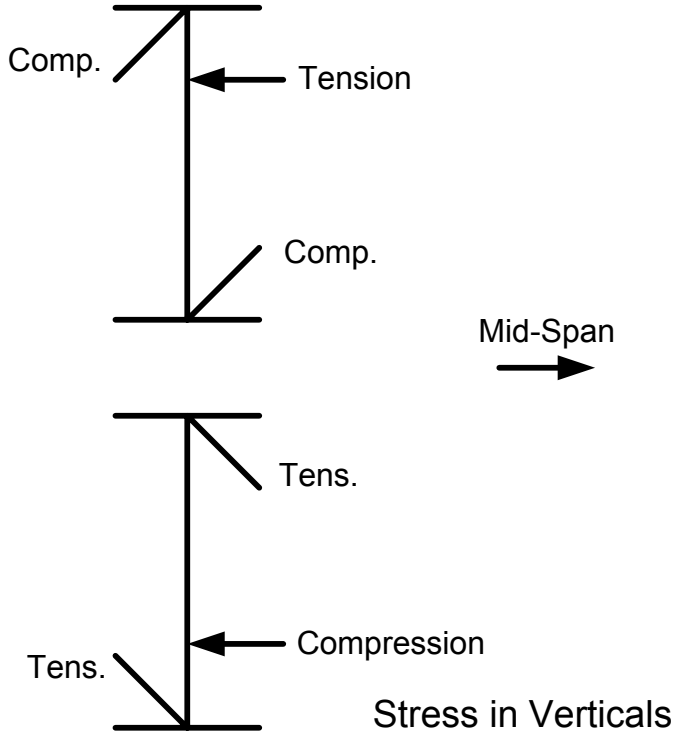


Figure 8.6.25 Vertical Stress Prediction Method

See Figure 8.6.18 of a sample truss schematic showing truss verticals.

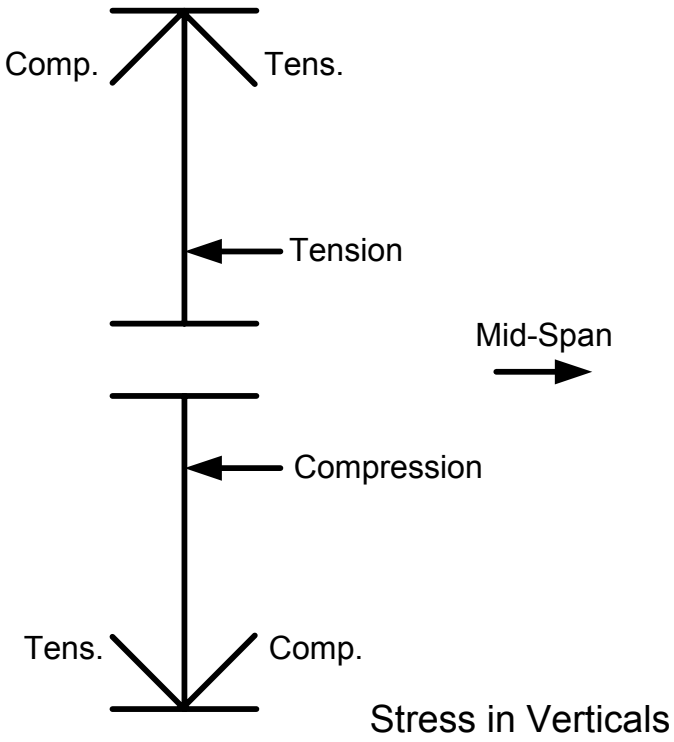


Figure 8.6.26 Vertical Stress Prediction Method

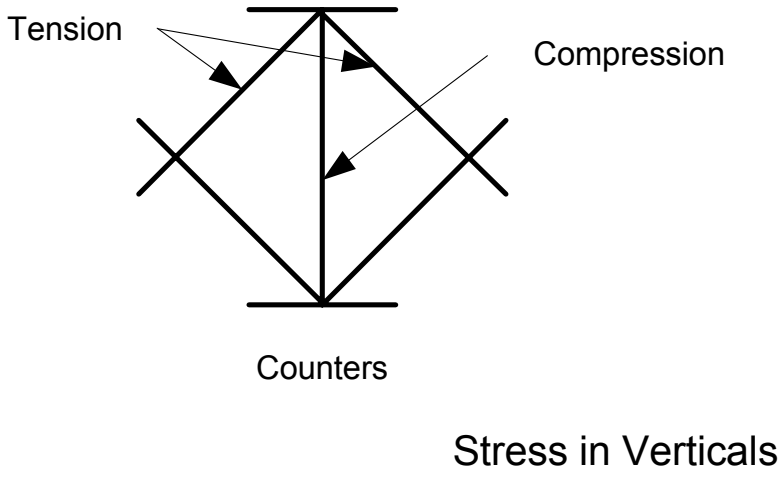


Figure 8.6.27 Vertical Stress Prediction Method

Panel Points and Panels

A panel point is the location where the truss members are connected together. Modern truss bridges are generally designed so that all members have approximately the same depth, thereby minimizing the need for shims and filler plates at the connections. This is often accomplished by varying the plate thicknesses of built-up members or using several grades of steel to meet varying stress conditions.

The connections are typically made using gusset plates and are made by riveting, bolting, welding or a combination of these methods. Connections using both rivets and bolts were popular on bridges constructed in the late 1950's and early 1960's, as high strength bolts began to replace rivets. Rivets were used during shop fabrication while bolts were used to complete the connection in the field (see Figure 8.6.28). Old trusses used pins at their connections (see Figure 8.6.29). Truss members may also be spliced, sometimes at locations other than the panel points.

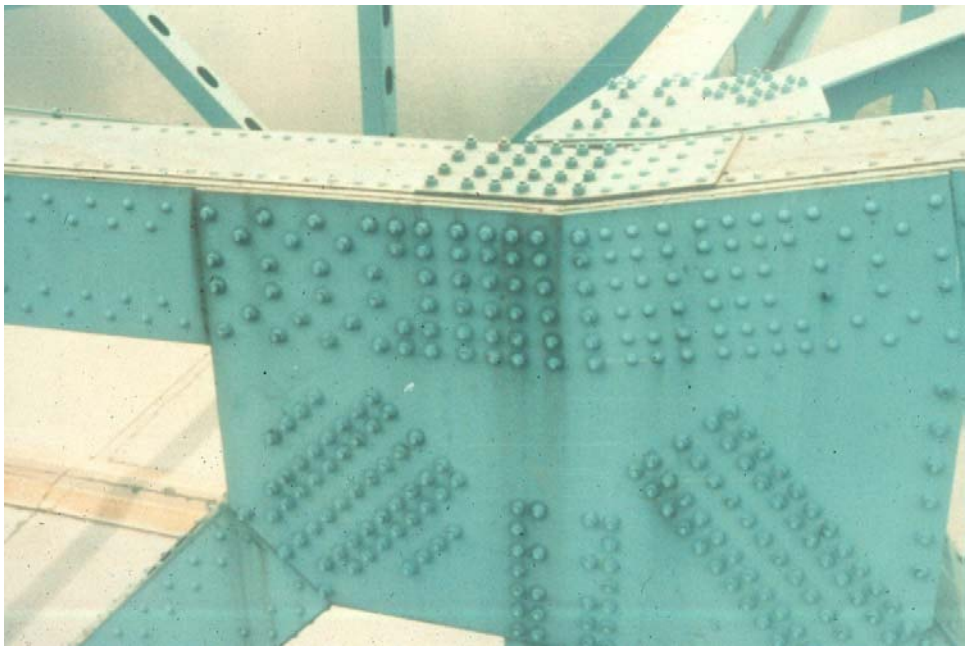


Figure 8.6.28 Truss Panel Point using Shop Rivets and Field Bolts

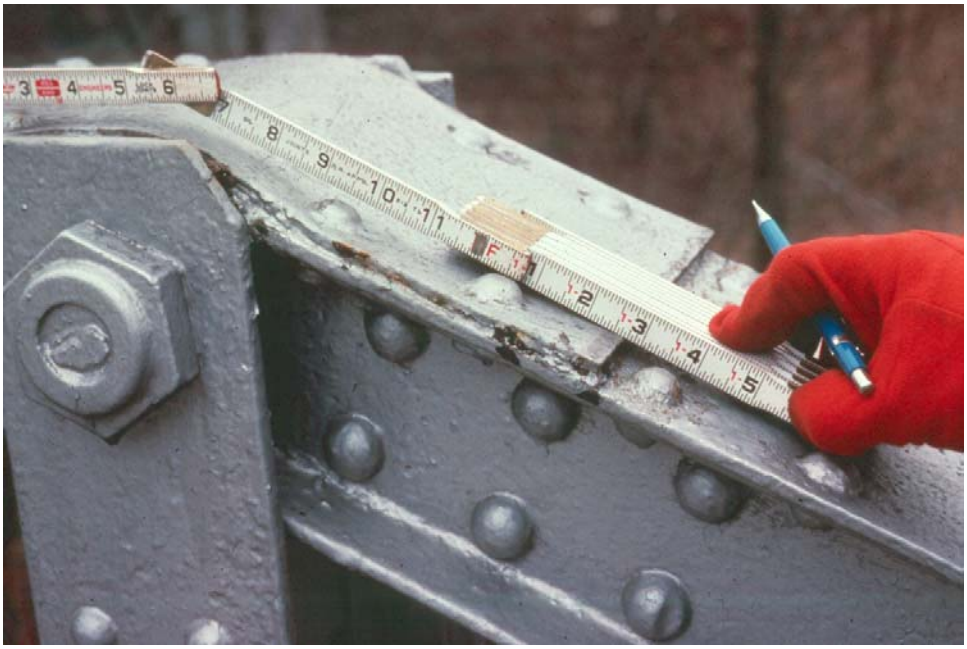


Figure 8.6.29 Pin Connected Truss

Either the letter U, for upper chord, or the letter L, for lower chord designates a panel point. Additionally, the panel points are numbered from bearing to bearing, beginning with 0 (zero). Most trusses begin with panel point L_0. Some deck trusses may begin with U_0. Upper and lower panel points of the same number are always in a vertical line with each other (e.g., U_7 is directly above L_7) (see Figures 8.6.30 and 8.6.31).

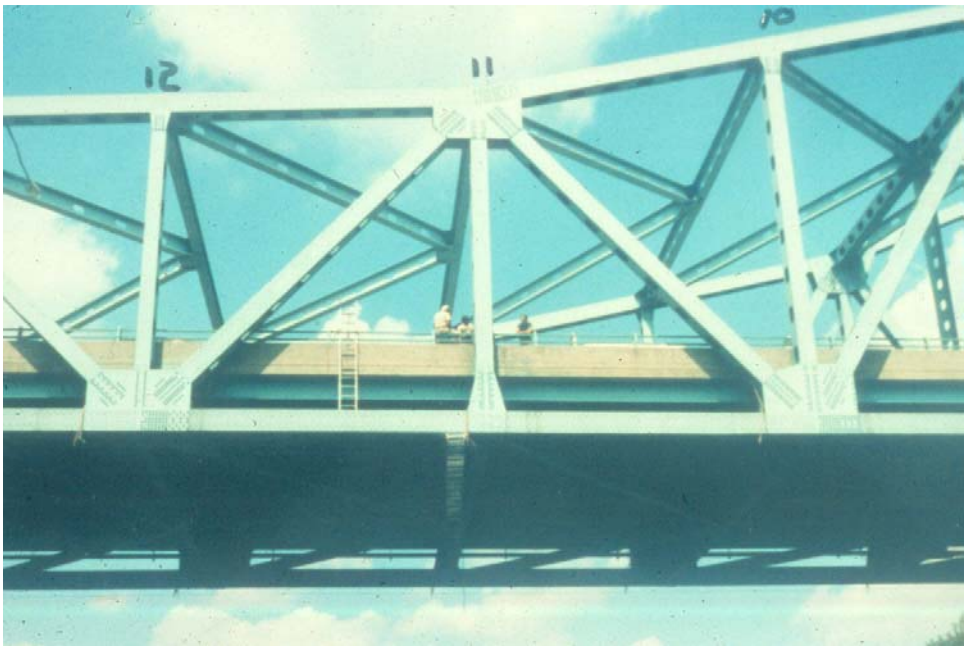


Figure 8.6.30 Truss Panel Point Numbering System



Figure 8.6.31 Deck Truss

A panel is the space, or distance, between panel points. Truss panels are typically 6 to 7.5m (20 to 25 feet) long but range from about 5 to 10 m (16 to 32 feet) deep. The panel length is a design compromise between cost and weight, with the longer panels requiring heavier floor systems.

As truss spans became longer, they also had to become deeper, increasing the distance between the upper and lower chords. They also required longer panels. As the panels became longer, the diagonals became even longer and the slope became flatter. The optimum angle between the diagonal and the horizontal is 45° to 55°.

To obtain a lighter floor system, designers subdivided the panel. The midpoint of each diagonal was braced with a downwardly inclined sub-diagonal in the opposite direction and with a sub-vertical down to the lower chord. Subpanel points are designated with the letter M and with the "half" number of the adjoining panels (e.g., $M_{7\ 1/2}$). The method of subdividing the truss created a secondary truss system within the main truss to support additional floorbeams. Baltimore and Pennsylvania trusses, patented in the 1870's, use this method. The K truss, a more recent design, accomplishes the same purpose (see Figure 8.6.32).



Figure 8.6.32 A Pennsylvania Truss, a Subdivided Pratt Truss with a Camel Back Top Chord

Floor System

Trusses have floorbeams at each panel and sub panel point along the truss. Floorbeams should be designated by their panel point number. Most trusses also have stringers in the floor system. The floor system of a truss bridge is therefore similar to the GFS floor system found on girder bridges. Floor systems (floorbeams and stringers) are subjected to bending and shear stresses. The difference is that there are two main trusses rather than two main girders (see Figure 8.6.33 and Figure 8.6.34).

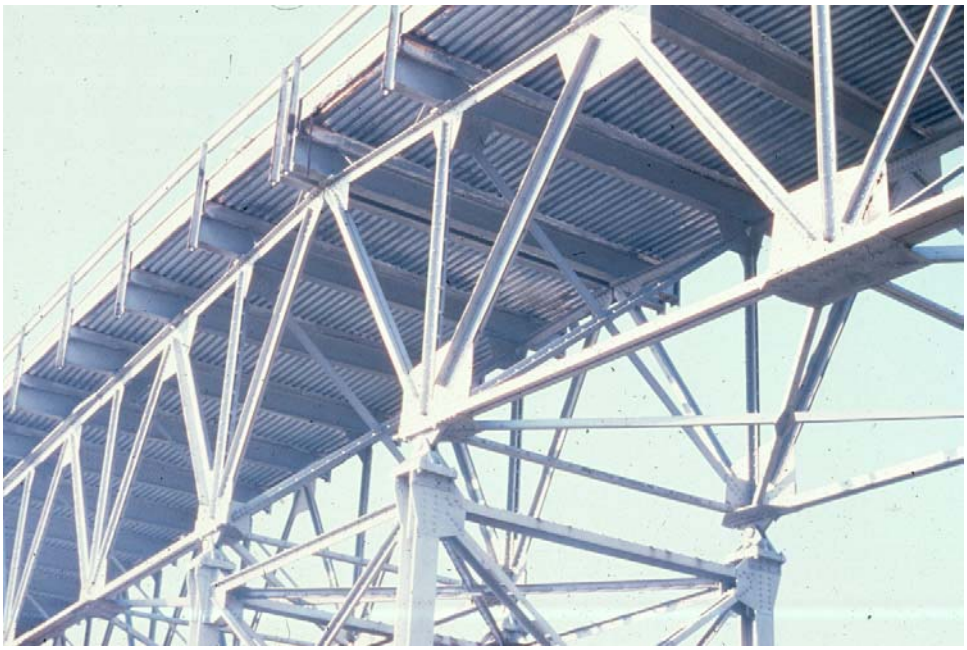


Figure 8.6.33 Deck Truss Floor System



Figure 8.6.34 Floorbeam Stringer Floor System

See Figure 8.6.18 of a sample truss schematic showing a truss floor system consisting of floorbeams and stringers.

Lateral Bracing

Upper/Lower Lateral Bracing

Upper/lower lateral bracing is in a horizontal plane and functions to keep the two trusses longitudinally in line with each other. Most trusses have upper and lower chord lateral bracing, with the exception of pony trusses, which do not have upper lateral bracing. The bracing is typically constructed from built-up and rolled shapes and is connected diagonally to the chords and floorbeams at each panel point using gusset plates (see Figure 8.6.18, Figure 8.6.35 and Figure 8.6.36).

The gusset plates are in a flat position and are therefore susceptible to a high degree of corrosion and deterioration (see Figure 8.6.37).

See Figure 8.6.18 of a sample truss schematic showing upper and lower lateral bracing.

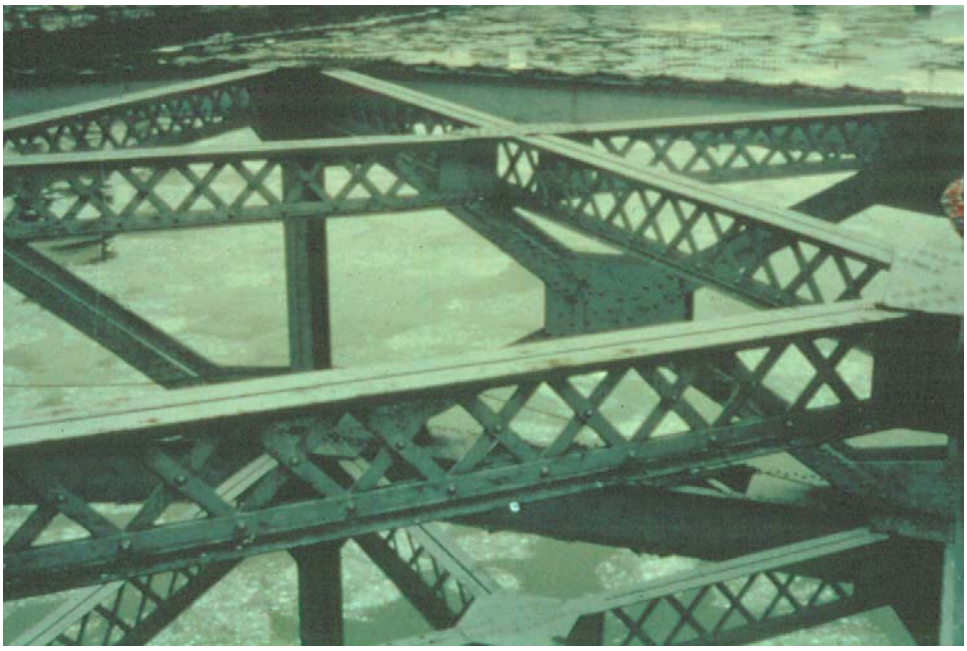


Figure 8.6.35 Upper Lateral Bracing



Figure 8.6.36 Lower Lateral Bracing



Figure 8.6.37 Corroded Lateral Bracing Gusset Plate

Sway/Portal Bracing

Sway bracing is in a vertical plane and functions to keep the two trusses parallel. The bracing is typically constructed from built-up or rolled shapes. The sway bracing at the end diagonal is called portal bracing and is much heavier than the other sway bracing. Sway bracing on old through trusses often limits the vertical clearance, and it therefore often suffers collision damage. Large pony trusses also have sway bracing in the form of a transverse diagonal brace from chord to chord. This member functions to keep the compression vertical from buckling by bracing the top chord (see Figures 8.6.18, 8.6.38, 8.6.39, 8.6.40 and 8.6.41).



Figure 8.6.38 Sway Bracing



Figure 8.6.39 Sway Bracing



Figure 8.6.40 Portal Bracing



Figure 8.6.41 Pony Truss "Sway Brace"

See Figure 8.6.18 of a sample truss schematic showing sway bracing.

Primary Members and Secondary Members

Primary members carry dead load and live load and consist of:

- Trusses (chords and web members)
- Floorbeams
- Stringers

Secondary members resist horizontal and longitudinal loads and consist of:

- Portal bracing
- Upper lateral bracing
- Lower lateral bracing
- Sway bracing

Secondary members do not contribute to the load-carrying capacity of the bridge. Rather, they function only to keep the primary members properly spaced. Only when that function is jeopardized will their condition influence the general condition of the bridge. It is common, however, for corrosion to occur on a primary member adjacent to the connection with a secondary member. It should be noted that built-up corrosion products might be hidden by paint.

The above primary and secondary members are shown on Figure 8.6.18.

8.6.3 Overview of Common Defects

Common defects that occur on steel truss bridges are:

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Tack welds in tension zones
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

8.6.4 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also

be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

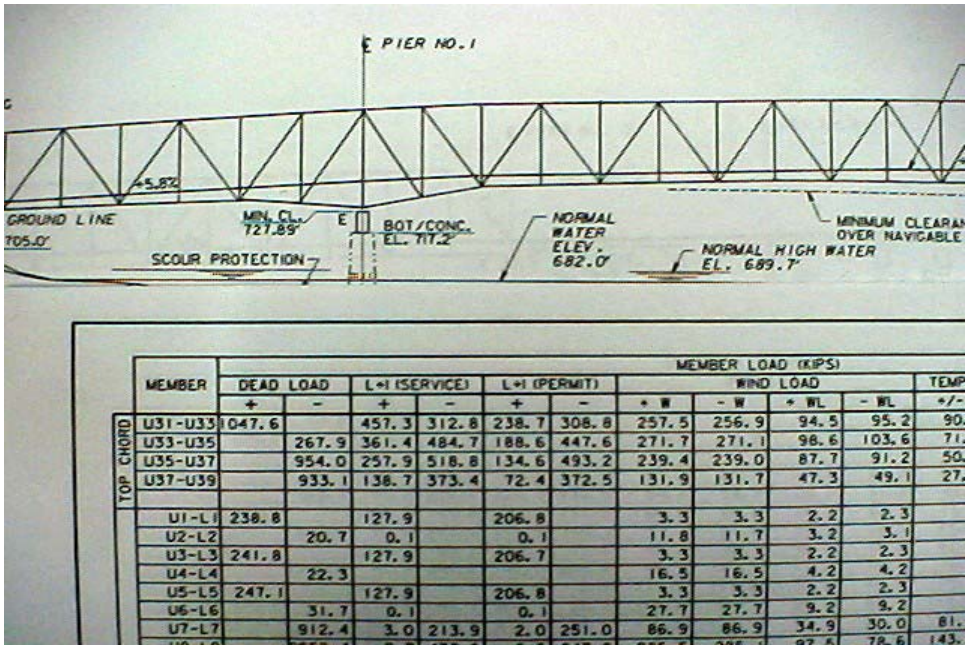
In steel trusses as well as other steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel truss members, including:

- Magnetic induction
- Electrical resistivity
- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Topic 13.3 for Advanced Inspection Techniques.

Locations

A truss consists of members, which are generally under axial loading only. Furthermore, many truss members are designed for force reversal. If a review of the bridge's design drawings indicates that a member is subjected to tension and compression, it should be inspected as a tension member (see Figure 8.6.42).



	MEMBER	MEMBER LOAD (KIPS) DEAD LOAD +	DEAD LOAD −	L+I (SERVICE) +	L+I (SERVICE) −	L+I (PERMIT) +	L+I (PERMIT) −	WIND LOAD + W	WIND LOAD − W	WIND LOAD + WL	WIND LOAD − WL	TEMP +/−
TOP CHORD	U31-U33	1047.6		457.3	312.8	238.7	308.8	257.5	256.9	94.5	95.2	90.
	U33-U35		267.9	361.4	484.7	188.6	447.6	271.7	271.1	98.6	103.6	71.
	U35-U37		954.0	257.9	518.8	134.6	493.2	239.4	239.0	87.7	91.2	50.
	U37-U39		933.1	138.7	373.4	72.4	372.5	131.9	131.7	47.3	49.1	27.
	U1-L1	238.8		127.9		206.8		3.3	3.3	2.2	2.3	
	U2-L2		20.7	0.1		0.1		11.8	11.7	3.2	3.1	
	U3-L3	241.8		127.9		206.7		3.3	3.3	2.2	2.3	
	U4-L4		22.3					16.5	16.5	4.2	4.2	
	U5-L5	247.1		127.9		206.8		3.3	3.3	2.2	2.3	
	U6-L6		31.7	0.1		0.1		27.7	27.7	9.2	9.2	
	U7-L7		912.4	3.0	213.9	2.0	251.0	86.9	86.9	34.9	30.0	81.

Figure 8.6.42 Truss Design Drawings

Bearing Areas

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

Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion.

Tension Members

For truss members subjected to tensile loads, special attention should be given to the following locations:

- Check for section loss (corrosion) and cracks (see Figure 8.6.43).
- For box-shaped chord members, check inside for debris and corrosion, cracks or section loss (see Figure 8.6.44).
- Examine eyebar heads for cracks in the eyes and in the forge zone (see Figure 8.6.45).
- Check loop rods for cracking where the loop is formed (see Figure 8.6.46).
- Where multiple eyebars make one member, check to see if the tension is evenly distributed - each eyebar element should be perfectly parallel to adjacent elements (see Figure 8.6.47).
- Check eyebars or loop rods where attachments are welded to them, especially if such attachments connect the eyebars together (see Figure 8.6.48).
- Determine whether the spacers on the pins are holding the eyebars and

loop rods in their proper positions.

- Look for repairs, especially welded repairs, if they have been applied to old, non-weldable steel tension members. Base metal cracks can easily develop at these locations (see Figure 8.6.48).
- Check the alignment of the members, make sure they are straight and not bowed - this could be a sign of permanent force reversal (see Figure 8.6.49).
- With a buckled lower tension chord, where has the load gone? Is something else overstressed as a result? (Possibly the adjacent tension diagonals) (see Figure 8.6.50)
- Observe the counters under live load for excessive wear and abnormal rubbing where the counters cross.

Check the condition of threaded members such as counters at turnbuckles. They should not be over-tightened or under-tightened. Pulling transversely by hand can check the relative tension. The counter should move slightly. If a problem is found, the inspector should not adjust the turnbuckle, but notify the bridge engineer.



Figure 8.6.43 Bottom Chord

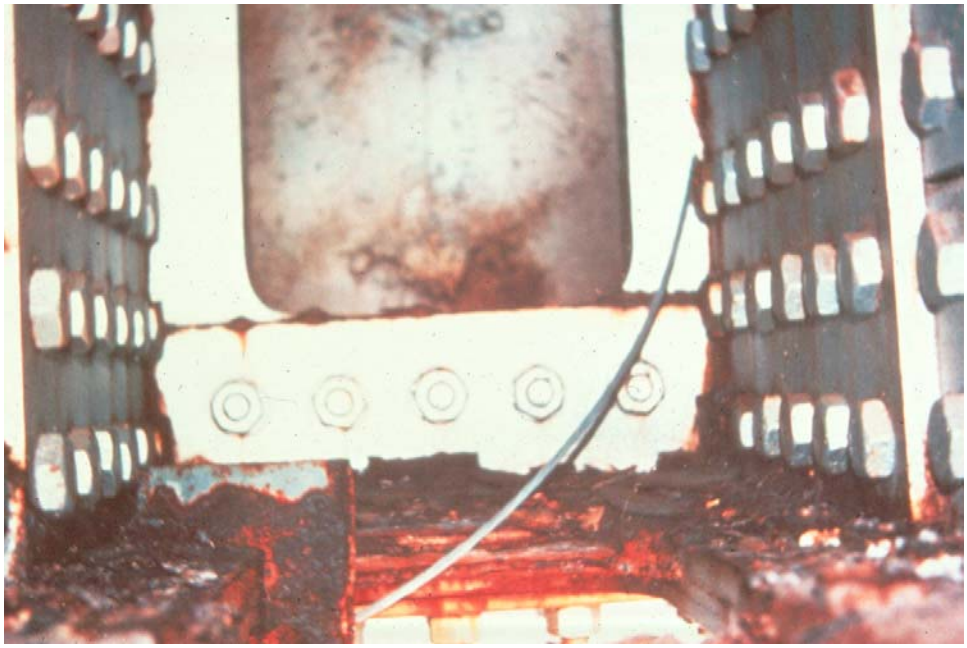


Figure 8.6.44 Inside of Box chord Member

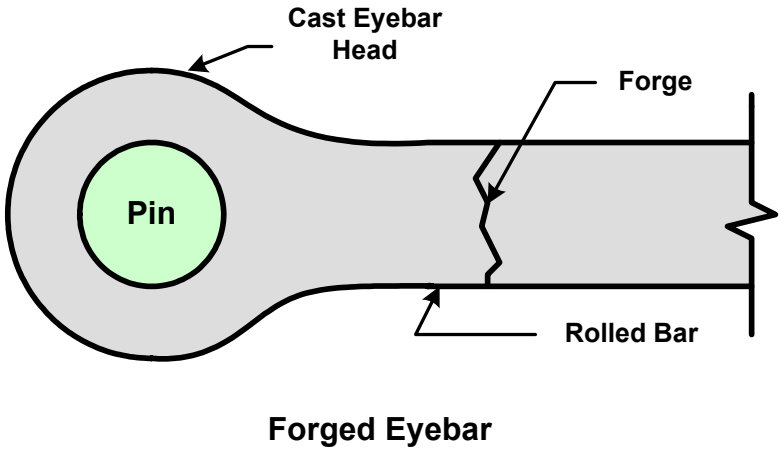


Figure 8.6.45 Forge Zone on an Eyebar

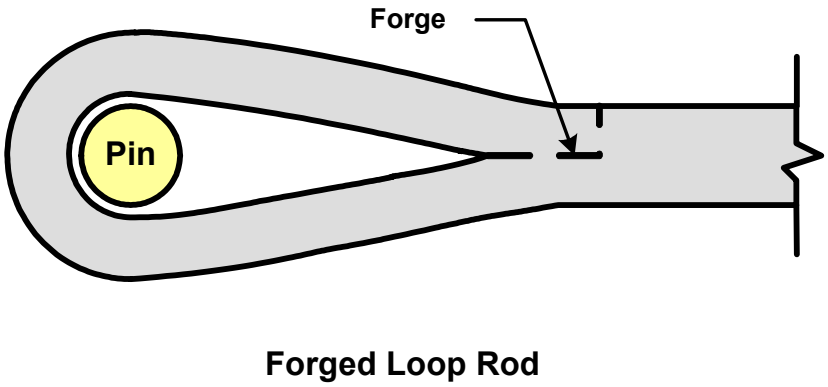


Figure 8.6.46 Forge Zone on a Loop Rod



Figure 8.6.47 Bottom Chord

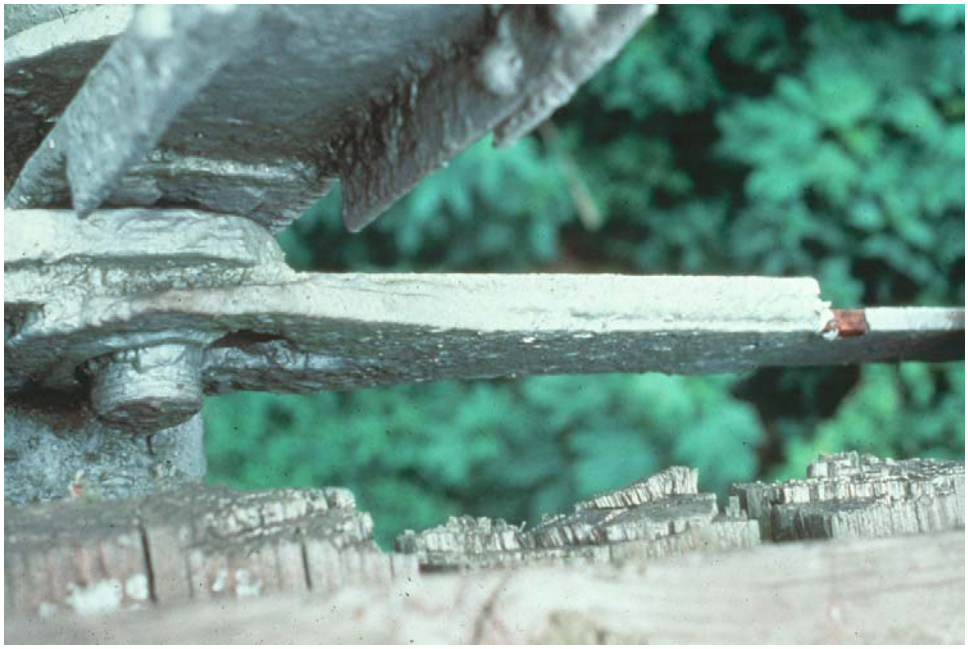


Figure 8.6.48 Welded Repair to Loop Rod



Figure 8.6.49 Bowed Bottom Chord Eyebar Member



Figure 8.6.50 Buckled Lowered Chord Member L0L1, due to Abutment Movement

On trusses with cantilevered and suspended spans the pin-connected joints that permit expansion are susceptible to freezing or fixity of the pinned joints. This can result in undesirable stresses in the structure - changing axial loaded members to bending members. Carefully inspect the pins at such connections for corrosion and fixity.

Compression Members

For truss members subjected to compressive loads, special attention should be given to the following locations:

- End posts and web members, which are vulnerable to collision damage from passing vehicles. Buckled, torn, or misaligned members may severely reduce the load carrying capacity of the member (see Figure 8.6.51).
- Check for local buckling, an indication of overstress (see Figure 8.6.52).
- Wrinkles or waves in the flanges, webs or cover plate are common forms of buckling.



Figure 8.6.51 Collision Damage to Vertical Member



Figure 8.6.52 Buckled End Post and Temporary Supports

Floor System

The floor system on a truss contains floorbeams and, most likely, stringers. These members function as beam members and are therefore subjected to bending stresses. The shear and flexure inspection locations and procedures discussed in previous topics also apply here. Distortion induced fatigue cracks have also developed in the webs of many floorbeams at connections to truss bridge lower chord panel points when the stringers are placed above the floorbeams. The webs of these floorbeams at the connections and adjacent to flanges and stiffeners need to be inspected routinely.

For steel truss floor systems, special attention should be given to the following locations:

- Check the end connections of floorbeams for corrosion as they are exposed to moisture and de-icing chemicals from the roadway (see Figure 8.6.53).
- Check the flanges of the floorbeams and stringers for corrosion, particularly under open grid decks (see Figure 8.6.54).
- Also check stringer webs for rust, and check the floorbeam and stringer connections for loose fasteners (see Figure 8.6.55).
- Listen for noises caused by moving members with the passage of traffic

(see Figure 8.6.56).

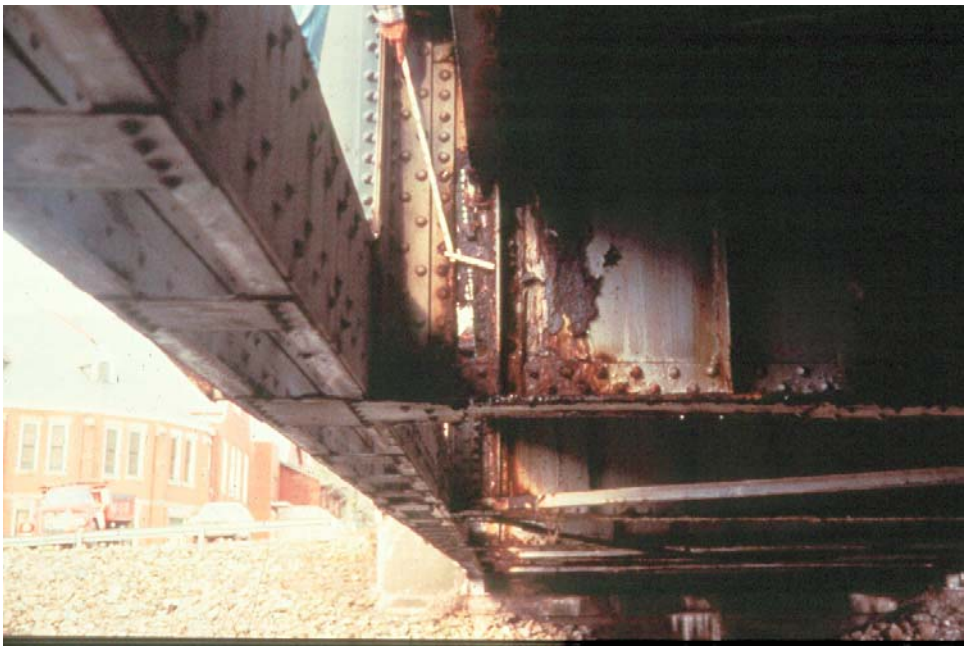


Figure 8.6.53 Deterioration of Floorbeam and Connection

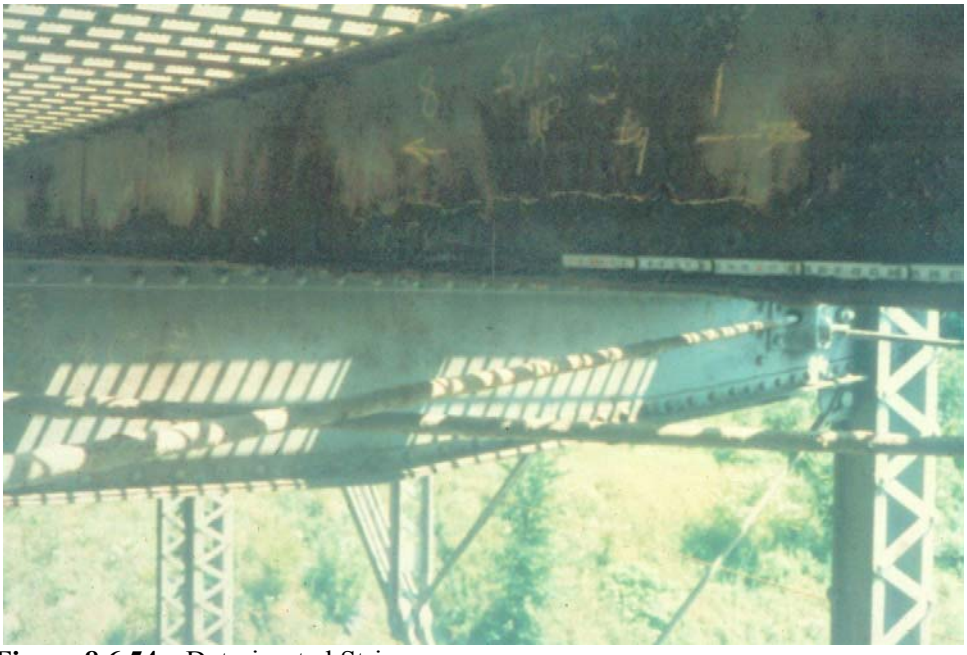


Figure 8.6.54 Deteriorated Stringer



Figure 8.6.55 Deteriorated Stringer Connection

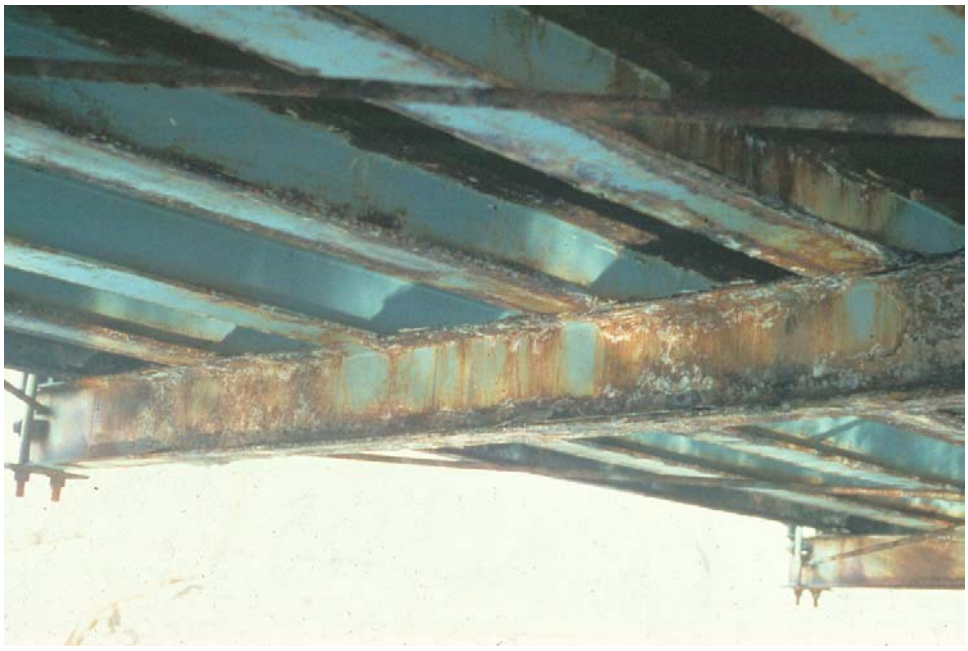


Figure 8.6.56 Deteriorated Floor System

Fatigue Prone Members and Details

For fatigue prone details, special attention should be given to the following locations:

- Check ends of welded cover plates of floorbeam and stringer flanges.
- Check the welded attachment of signs, railings, and utilities.
- Check the welds on any repair or reinforcement plate attached to the truss member (see Figure 8.6.57).

- Check for cracks at the copes and blocked flanges at ends of floorbeams and stringers (see Figure 8.6.58).
- Check the floorbeam and stringer connection angles for cracks (see Figure 8.6.59).
- Note: The floorbeam to truss is a very critical load path; these connections deserve very close scrutiny.
- Check the horizontal gusset plate connections of the lateral bracing to the floorbeam flanges or webs.
- Check the ends of the vertical truss members and the end gusset plates for cracks.
- Check the ends of the vertical and diagonal eyebar members for cracks.
- Check pins on suspended spans (see Figure 8.6.60).
- Check all tack welds, for example, between gusset plates and main members and between floorbeam and stringer connections. The existence of tack welds should be immediately brought to the attention of the bridge engineer.

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

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

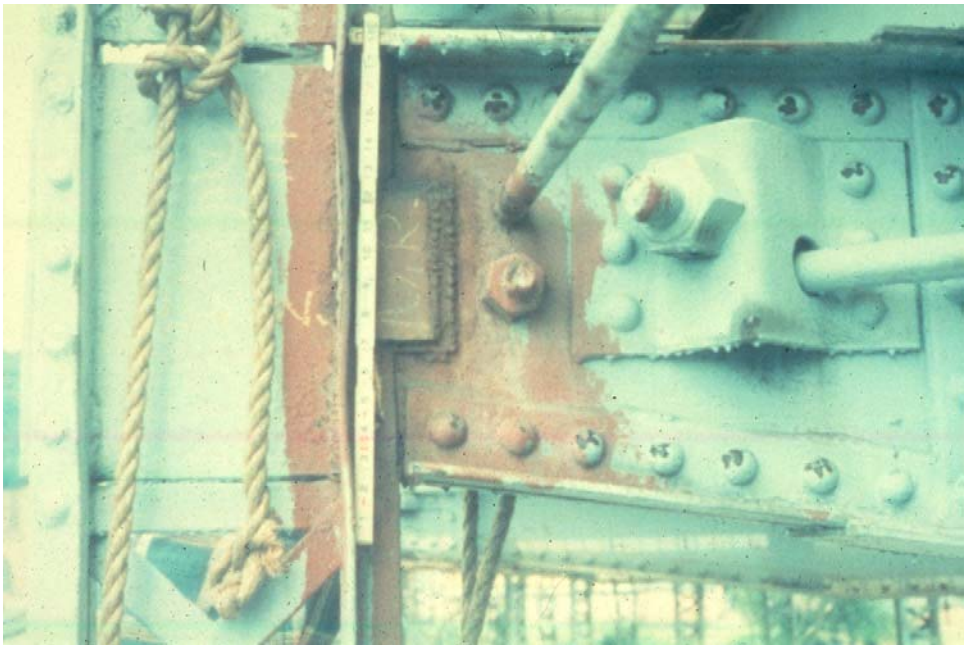


Figure 8.6.57 Welded Repair Plate



Figure 8.6.58 Coped Stringer



Figure 8.6.59 Clip Angles at Floorbeam and Stringer Connections



Figure 8.6.60 Suspended Span Pin

Refer to Topic 8.1 for fatigue prone details and photographs.

With two primary members in a truss, there is no load path redundancy. Therefore, trusses are non-redundant structures. Trusses are not, however, a single member. They are formed by the connection of many small members. Whether the failure of any given tension member would cause the truss to collapse is best determined by a detailed engineering analysis. The bridge inspector should assume that all tension members are fracture critical members until an analysis is performed and the FCM's have been identified.

Trusses by nature, can have some internal redundancy, depending upon the members involved. For example, the structure in Figure 8.6.61 has 119 members. The structural analysis indicated that there are 66 tension members, 30 of which are fracture critical (see Figure 8.6.62). The National Bridge Inspection Standards [650.03(e)(1)] require that fracture critical members be identified for individual bridges and the procedures for inspection listed.

Inspect pins for scoring and other signs of wear (see Figure 8.6.63). Be sure that spacers, nuts, retaining caps, and keys are in place.



Figure 8.6.61 Sewickley Bridge

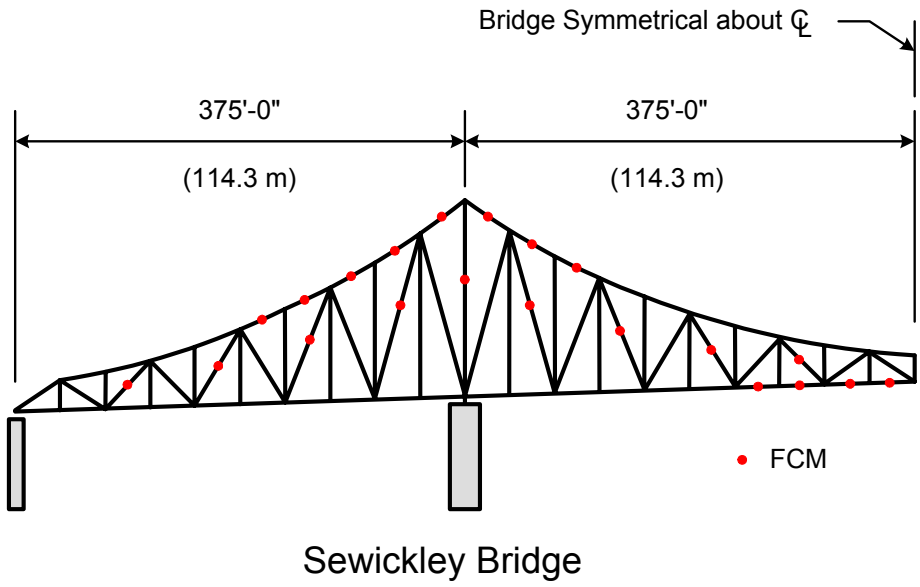


Figure 8.6.62 Through Truss with Fracture Critical Members Identified



Figure 8.6.63 Worn and Corroded Pin

Pin and hanger connections, when used in suspended span configurations in non-redundant truss systems, are fracture critical. This type of connection warrants particular attention. Corrosion can cause fixity in pin connections. This changes the structural behavior of the connection and is a source of cracking.

Usually the hanger plates are compact members similar to a vertical or diagonal. The hanger then slips between gusset plates at both the upper and lower chords. It is difficult to find a fixed reference point because the gusset plate dimensions are not usually given on design plans. However, two recommended options are the intersection of the upper or lower chord and nearest diagonal or the edge of the gusset plate along the axis of the hanger. Both of these points will provide readily identifiable reference points that can be re-created easily by the next inspection team. For this reason, measurements should be carefully documented along with the temperature and weather conditions. When inspecting a truss pin and hanger, locate the center of the pin, measure to a reference point to determine section loss, and compare the measurements to plans or previous inspection notes. Refer to Topic 8.4.3 for further details about the inspection of pin and hanger connections.

Secondary Members

For steel truss secondary members, special attention should be given to the following locations:

- Check for collision damage at the portals and at knee braces (see Figure 8.6.64).
- Check for loose fasteners at the bracing connections.
- Check the lateral bracing gusset plates for corrosion. These plates typically deteriorate more rapidly than other elements on a truss because they are exposed to, and retain, moisture and deicing salts (see Figure

8.6.37).

- Check the alignment of the lateral bracing members (see Figure 8.6.65).
- Check for corrosion and rust packing of bracing (see Figure 8.6.66).

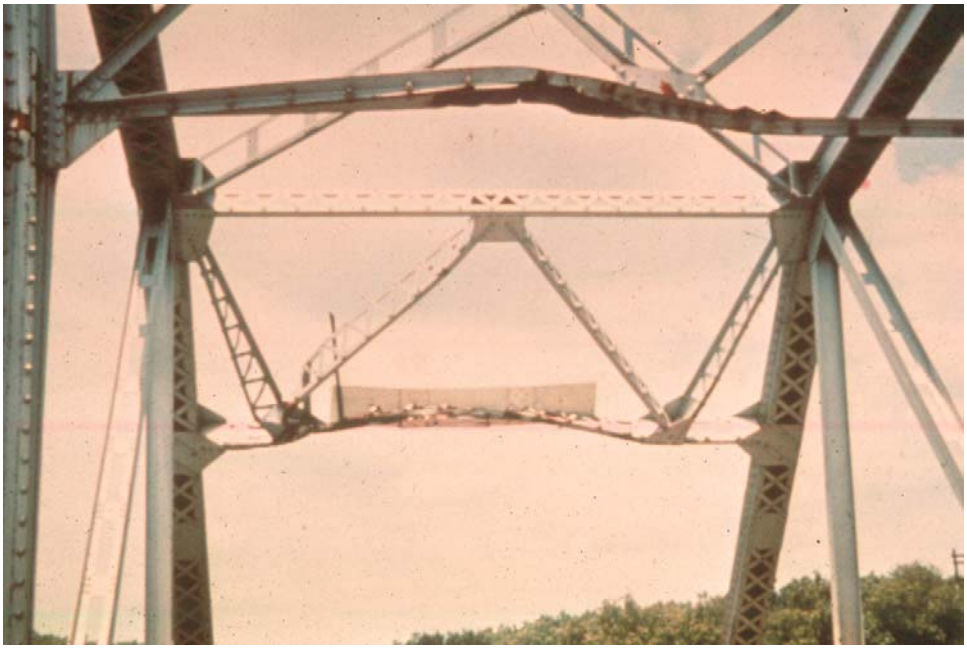


Figure 8.6.64 Collision Damage to Portal

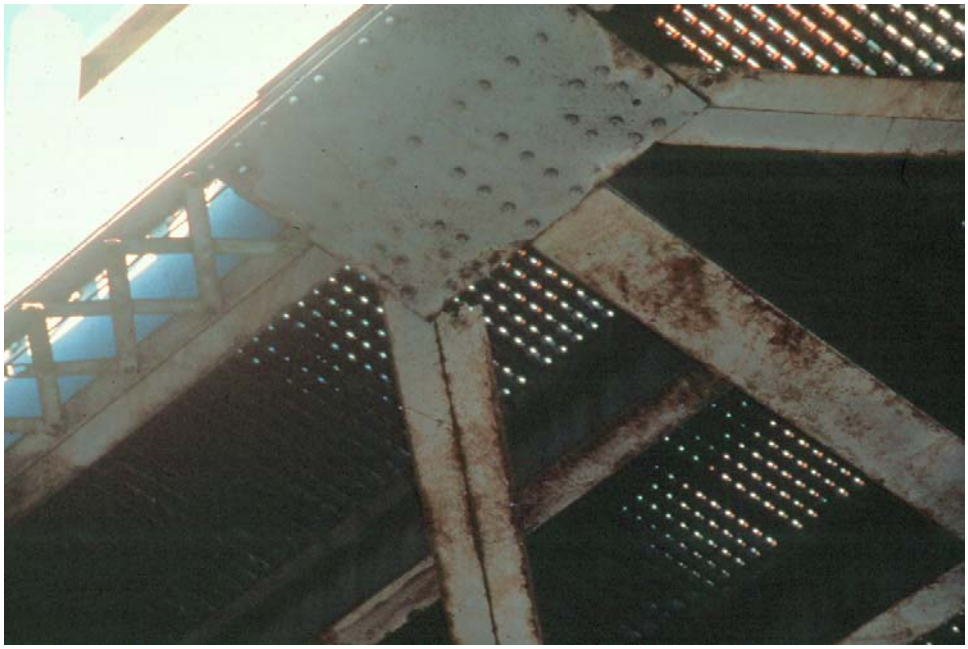


Figure 8.6.65 Lateral Bracing

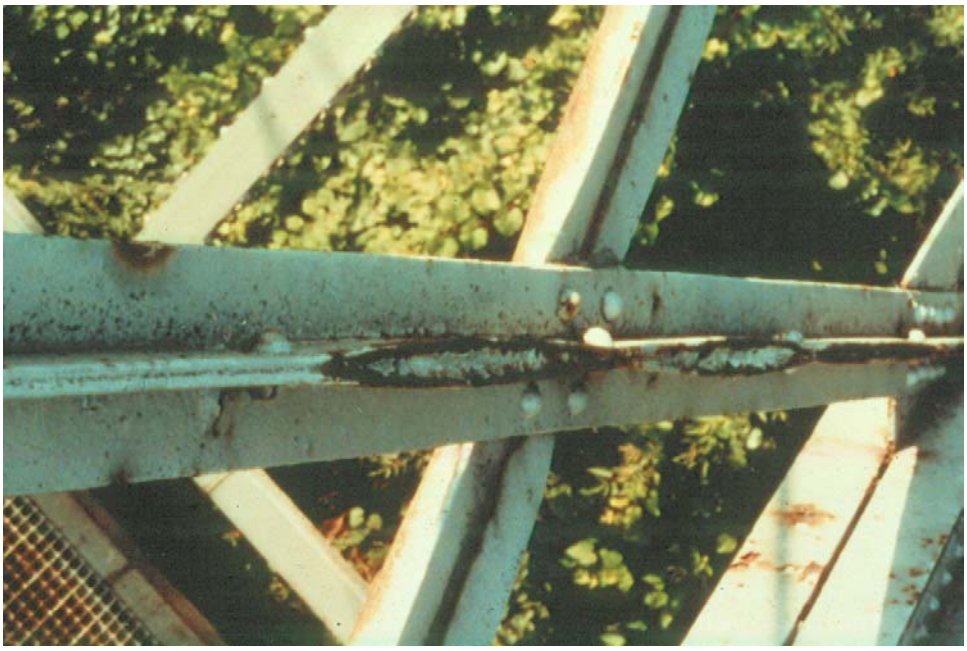


Figure 8.6.66 Corroded Sway Bracing

Areas that Trap Water and Debris

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

- Lateral bracing gusset plates
- Inside built-up chord members (horizontal surfaces)
- Areas exposed to drainage runoff
- Pockets created by floor system connections
- Tightly packed panel points
- Pin and Hanger assemblies

Non-critical Elements

Inspect chord members for corrosion, examining horizontal surfaces where moisture can collect. Check for corrosion and general deterioration of the lacing bars, stay plates, and batten plates.

8.6.5 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

The superstructure rating is only influenced by the condition of the main load carrying members.

Only corrosion, section loss, and fatigue cracks impact the superstructure rating. The location, dimensions and extent of the above mentioned discrepancies should be noted on inspection forms.

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the superstructure. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Pontis Smart Flags are also used to describe the condition of the superstructure.

In an element level condition state assessment of trusses, the AASHTO CoRe element is:

Element No.	Description
106	Open Girder/Beam – Unpainted Steel
107	Open Girder/Beam – Painted Steel
112	Stringer – Unpainted Steel
113	Stringer – Painted Steel
151	Floorbeam – Unpainted Steel
152	Floorbeam – Painted Steel
	Truss
120	Thru Truss (Bottom Chord) – Unpainted Steel
121	Thru Truss (Bottom Chord) – Painted Steel
125	Thru Truss (Excluding Bottom Chord) – Unpainted Steel
126	Thru Truss (Excluding Bottom Chord) – Painted Steel
130	Deck Truss – Unpainted Steel
131	Deck Truss – Painted Steel

The unit quantity for the truss is in meters (or feet). The total length must be placed in one of the four available condition states for unpainted steel superstructure, and one of the five available condition states for painted steel superstructure. Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For damage from steel fatigue, the “Steel Fatigue” Smart Flag, Element No. 356, can be used and one of the three condition states assigned. For signs of rust packing between steel plates, the “Pack Rust” Smart Flag, Element No. 357 can be used and one of the four condition states assigned. For damage from traffic impact, the “Traffic Impact” Smart Flag, Element No. 362, can be used and one of the three condition states assigned. For damage from section loss, the “Section Loss” Smart Flag, Element No. 363, can be used and one of the four condition states assigned.

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Topic 8.7 Steel Eyebars

8.7.1 Introduction

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



Figure 8.7.1 Typical Eyebar Tension Member on a Truss



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



Figure 8.7.3 Eyebar Chain Suspension Bridge



Figure 8.7.4 Anchorage Eyebar

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



Figure 8.7.5 Collapsed Silver Bridge

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



Figure 8.7.6 Inspection of Eyebars

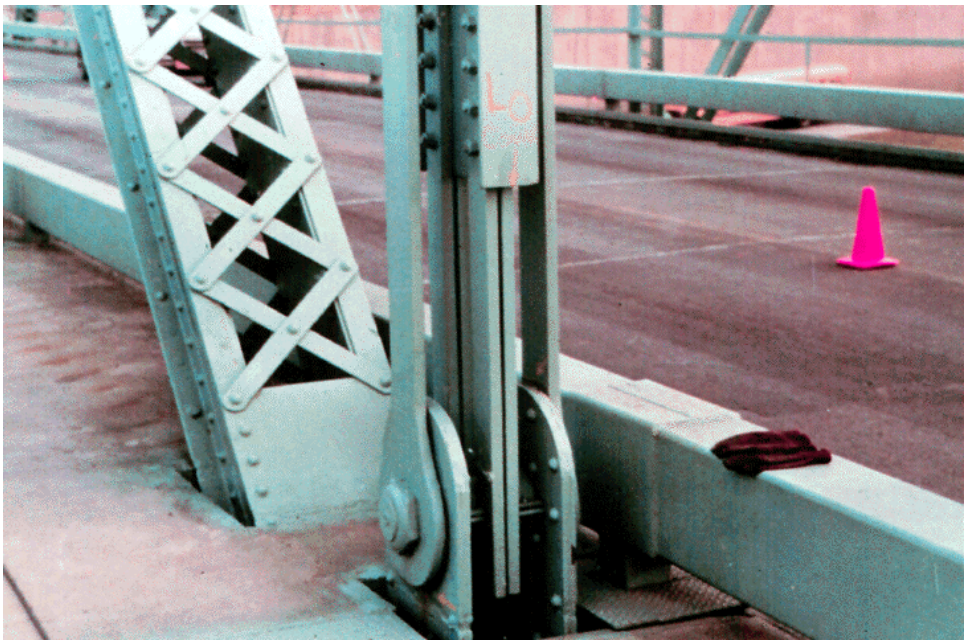


Figure 8.7.7 Retrofit of Eyebars to Add Redundancy

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

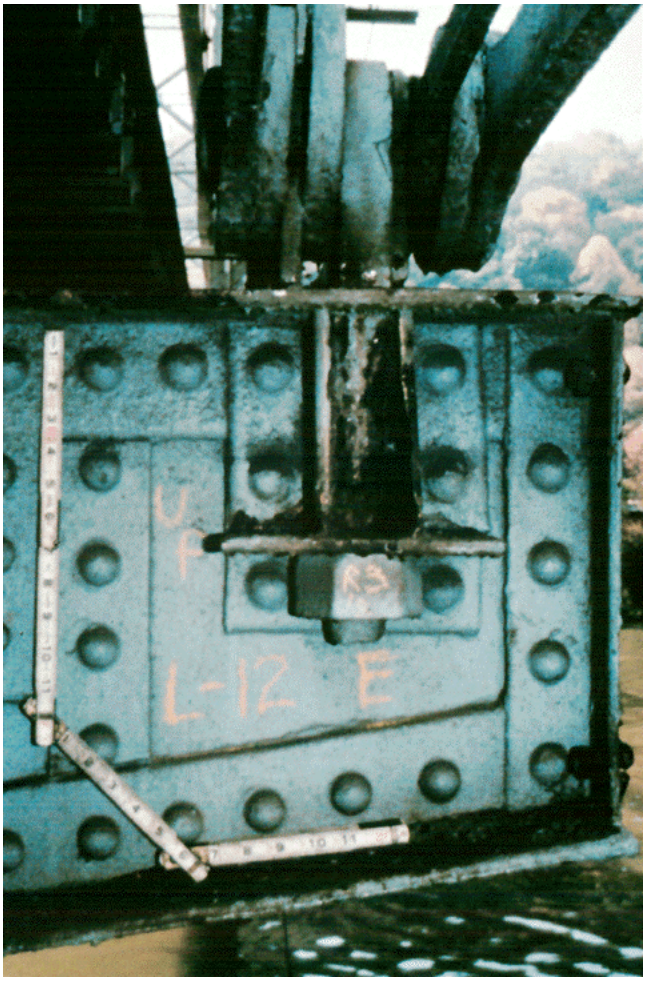


Figure 8.7.8 Close-up of Tightly Packed Eyebar Connection with Corrosion

8.7.2 Design Characteristics

Development of Steel Eyebars

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



Figure 8.7.9 Eads Bridge, St. Louis

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

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

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

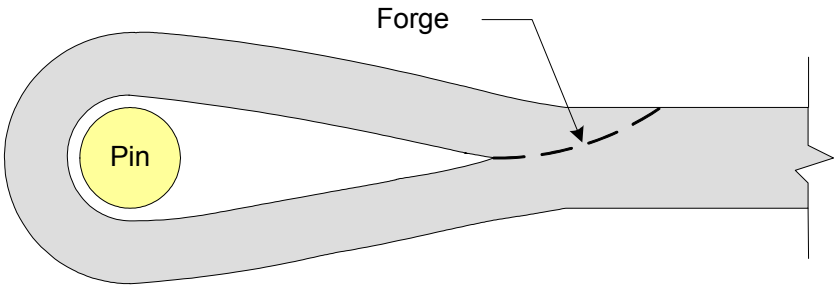
These heat treated alloy steels were extremely strong and contributed substantial economics, but they could not be welded.

Forging

Eyebars are formed by three methods of forging. Forging is a method of hot working to form steel by using hammering or pressing techniques. The three methods are:

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 Figure 8.7.10 and 8.7.11). Loop rods were first made of wrought iron, and later steel, by forging a heated bar around a pin, pounding the bar until a closed loop was formed.



Forged Loop Rod

Figure 8.7.10 Forging of a Loop Rod

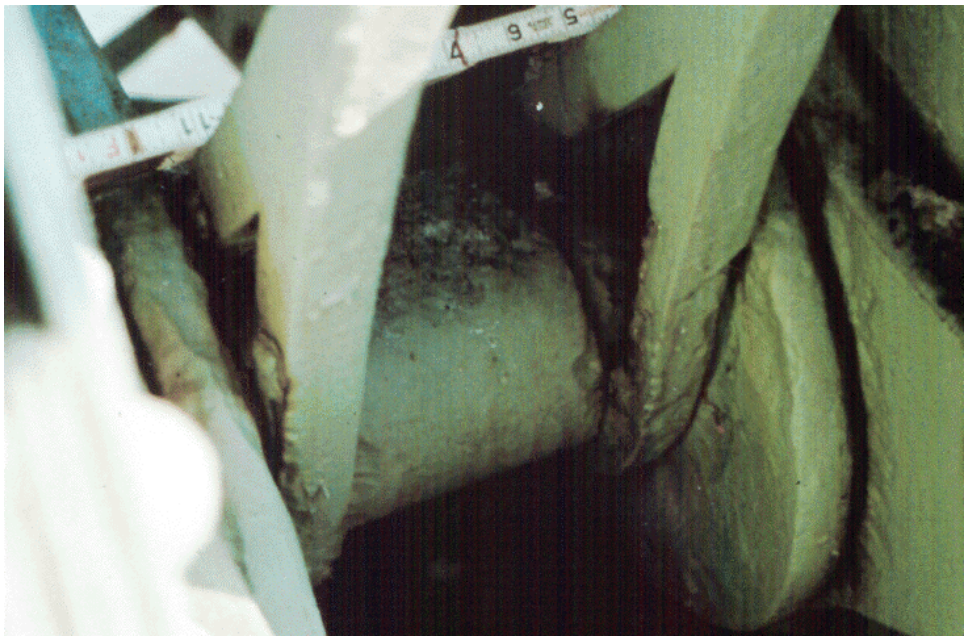


Figure 8.7.11 Close-up of the End of a Loop Rod

Pressing

Steel eyebars were also formed with a special type of mechanical forge press called an upsetting machine. The eyebar consists of the two heads (formed by

casting) joined to the ends of the shaft. The upsetting machine clamps the eyebar pieces between two dies with vertical faces. The eyebar is then forged and shaped by the horizontal action of a ram operated by a crankshaft (see Figure 8.7.12). Most other forging presses operate with vertical rams.

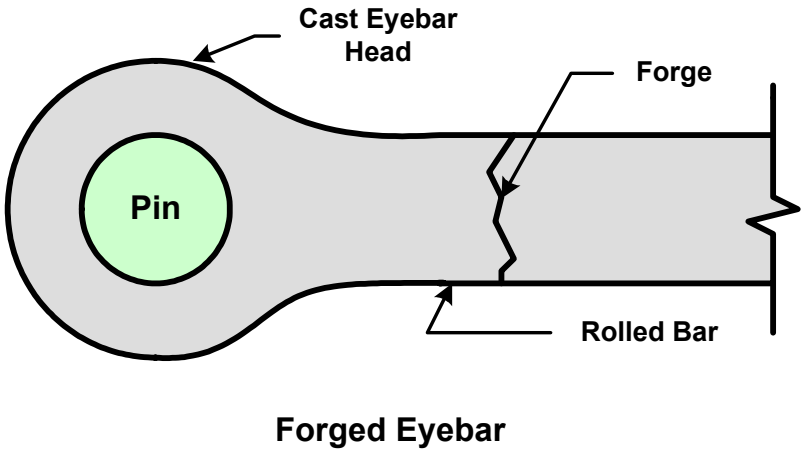


Figure 8.7.12 Forging of an Eyebar

The two other methods of hot working steel include rolling and extrusion. Rolling is performed in rolling mills, producing the standard structural shapes. Extrusion uses hydraulic presses to force the hot plastic steel through a die.

Pin Hole

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

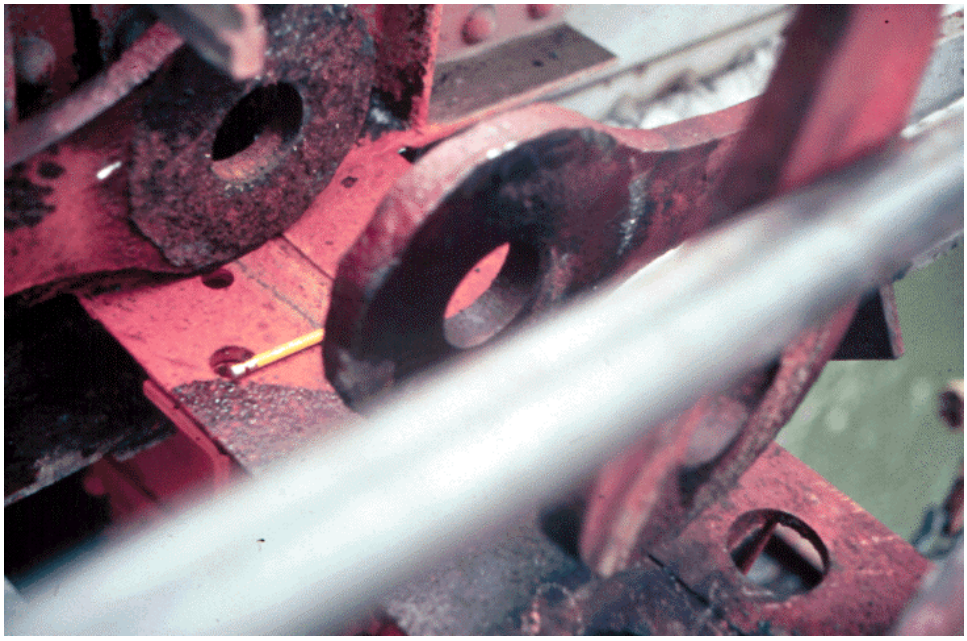


Figure 8.7.13 Close-up of an Eyebar Pin Hole (Disassembled Connection)

Heat Treating and Annealing

Heat treating of steel is an operation in which the steel is heated and cooled, under controlled conditions according to a predetermined schedule, for the purpose of obtaining certain desired properties.

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

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

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

More than one of these effects can often be obtained simultaneously. The inspector may find the terms “heat treated” and “annealed” on bridge plans to describe eyebars. This is very common.

Dimensions

The dimensions of a typical eyebar are as follows:

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

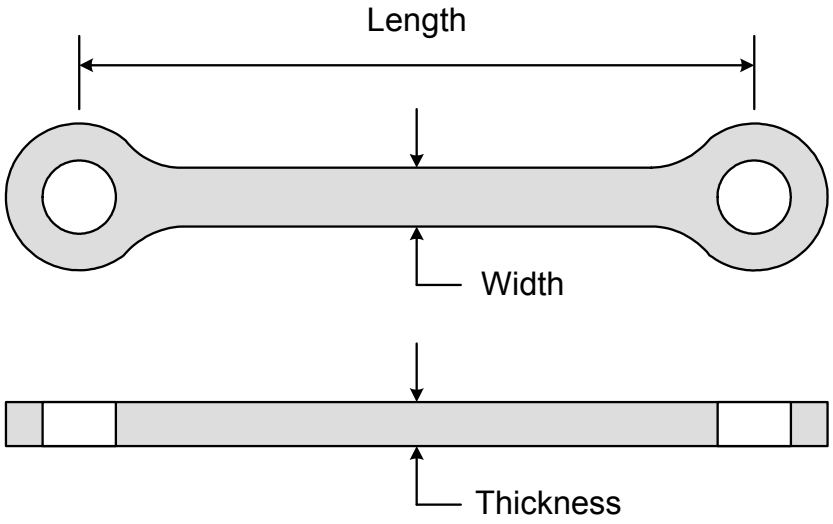


Figure 8.7.14 Eyebar Dimensions

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

Packing

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

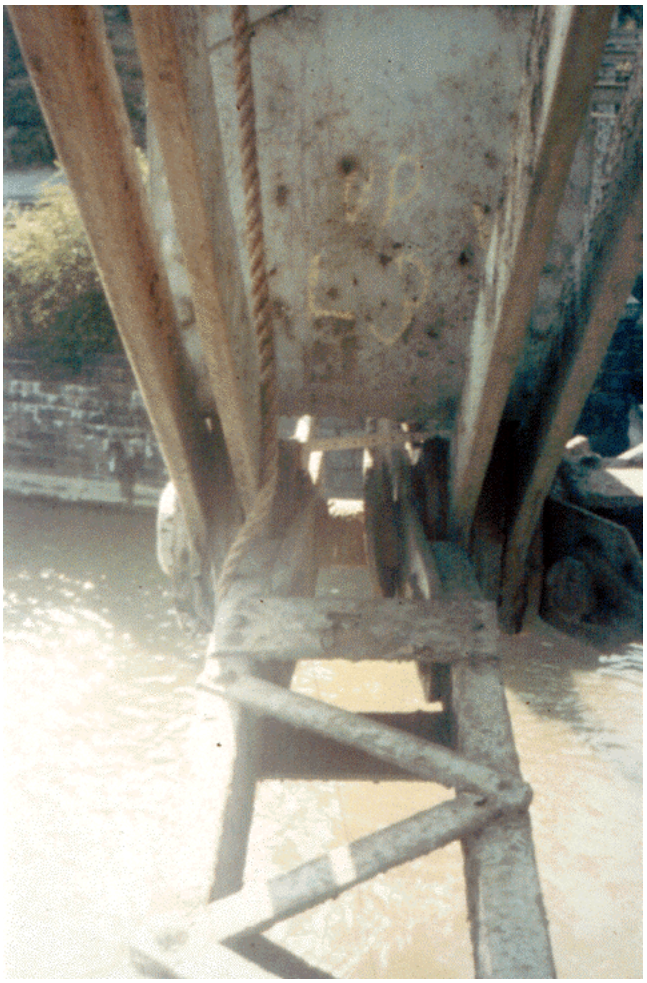


Figure 8.7.15 Loosely Packed Eyebar Connection

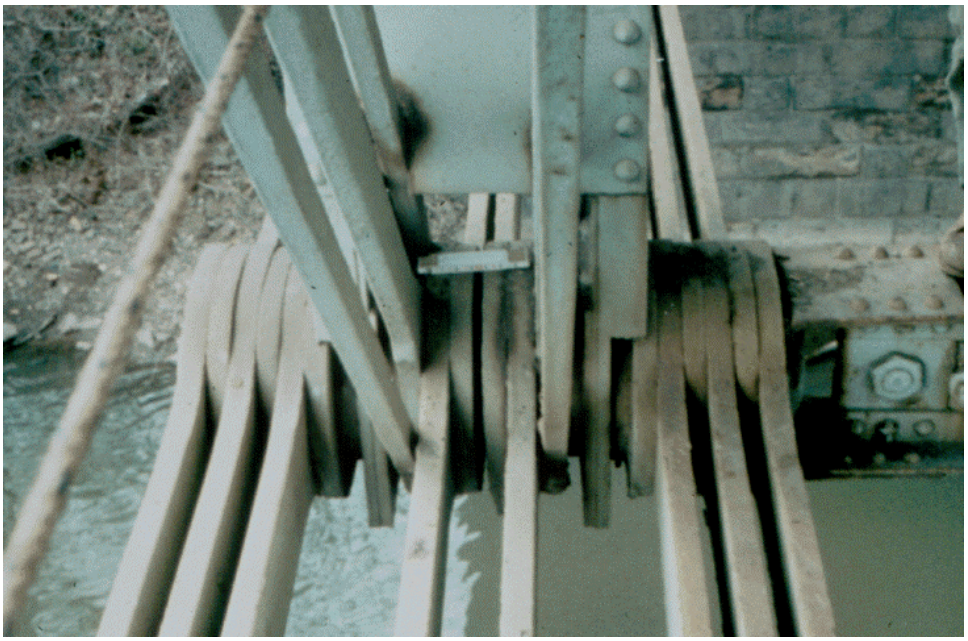


Figure 8.7.16 Tightly Packed Eyebar Connection

Spacers

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



Figure 8.7.17 Close-up of a Steel Filling Ring, or Pin Spacer

Redundancy

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

The collapse of the Silver Bridge is attributed to the failure of an eyebar within a non-redundant eyebar member. When the first eyebar failed, the second eyebar was unable to carry the load and resulted in the complete failure of that member. Redundancy is discussed in Topics P.1 and 8.1.



Figure 8.7.18 Non-redundant Eyebar Member

8.7.3 Overview of Common Defects

Common defects that occur on steel eyebar bridges include:

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Tack welds in tension zones
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

8.7.4 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

In steel eyebar bridges as well as other steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel arch, including:

- Acoustic Emissions Testing
- Computer Programs
- Computer Tomography
- Corrosion Sensors
- Dye Penetrant
- Magnetic Flux Leakage
- Radiographic Testing
- Robotic Inspection

- Ultrasonic Testing
- Eddy Current
- Brinell Hardness Test
- Charpy Impact Test
- Chemical Analysis
- Tensile Strength Test

See Topic 13.3 for Advanced Inspection Techniques.

Locations

Forge Zone

Inspect carefully the area around the eyebar head and the shank for cracks. This is where most failures occur in eyebars. Check the loop rods for cracks where the loop is formed (see Figure 8.7.19).



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

Tension Zone

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

Alignment

Check the alignment of the shank along the full length of the eyebar. The eyebar should be straight. A bowed eyebar indicates that a compressive force has been introduced (see Figure 8.7.20). The inclination of the eyebar to the plane of the truss should not exceed 5 millimeters per meter (1/16 inch per foot).



Figure 8.7.20 Bowed Eyebar Member

Misalignment can be caused by movement at the supports. The eyebars of the same member should be parallel (see Figure 8.7.21and 8.7.22).



Figure 8.7.21 Buckled Eyebar due to Abutment Movement



Figure 8.7.22 Non-parallel Eyebar Member

Areas That Trap Water and Debris

Areas that trap water and debris can result in active corrosion cells that can cause notches susceptible to fatigue or perforation and loss of section.

Packing

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

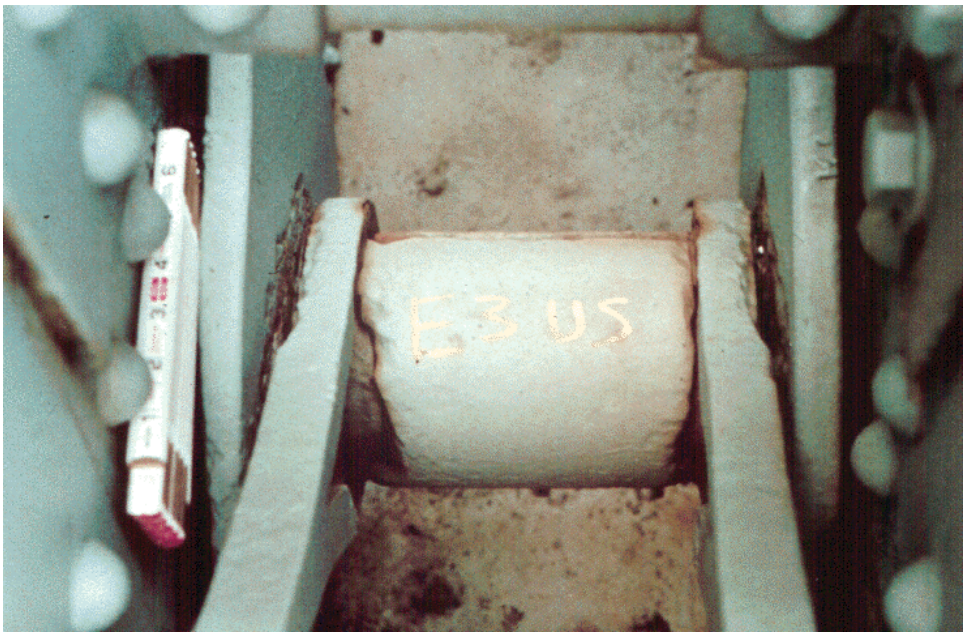


Figure 8.7.23 Corroded Spacer

- Corrosion - 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.

- Symmetry - The eyebars should be symmetrical about the central plane of the spacer (see Figure 8.7.24).



Figure 8.7.24 Symmetry at an Eyebar Connection

Areas Exposed to Traffic

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

Load Distribution

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



Figure 8.7.25 Eyebar Member with Unequal Load Distribution

Weldments

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

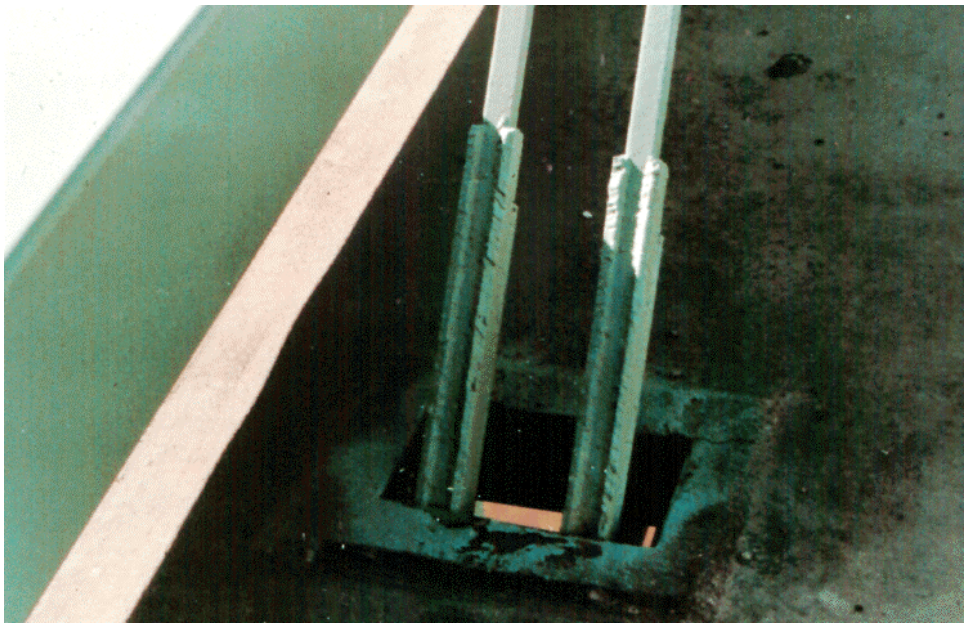


Figure 8.7.26 Welds on Loop Rods



Figure 8.7.27 Welded Repair to Loop Rods

Turnbuckles

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



Figure 8.7.28 Close-up of Turnbuckles



Figure 8.7.29 Welded Repair to Turnbuckles

Pins

Pins should be inspected for signs of wear and corrosion. Nondestructive methods such as ultrasonic inspection are recommended since visual inspection cannot reveal internal material flaws that may exist (see Figure 8.7.30). See Topic 13.3 for nondestructive testing methods.



Figure 8.7.30 Ultrasonic Inspection of Eyebar Pin

Fracture Critical Members

Members with only one or two eyebars should be considered fracture critical members. If members have three or more eyebars, they offer some degree of redundancy; however, a failure analysis should be performed to determine if they are also fracture critical (see Figure 8.7.31 and 8.7.32). See Topic 8.1 for a detailed discussion on fracture critical members.



Figure 8.7.31 Internally Non-redundant Eyebar Bottom Chord and Diagonal Truss Members



Figure 8.7.32 Internally Redundant Eyebar Top Chord Truss Member

8.7.5 Evaluation

State and federal rating guideline systems have been developed in order to aid in the inspection of steel eyebars. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) component rating method and the AASHTO element level condition state assessment method.

Application of the NBIS Under the NBIS rating guidelines, the steel eyebars are considered part of the

Rating Guidelines

superstructure and do not have an individual rating. The rating for the superstructure should take into account the condition of the steel eyebar assembly and may be lowered due to a deficiency in the steel eyebars. The superstructure is still rated as a whole unit but the steel eyebars may be the determining factor in the given rating.

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the steel bar assembly. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value.

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

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

The unit quantity for steel eyebars is meters (feet) and must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

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Topic 8.8 Steel Arches

8.8.1 Introduction

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

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



Figure 8.8.1 Deck Arch Bridge

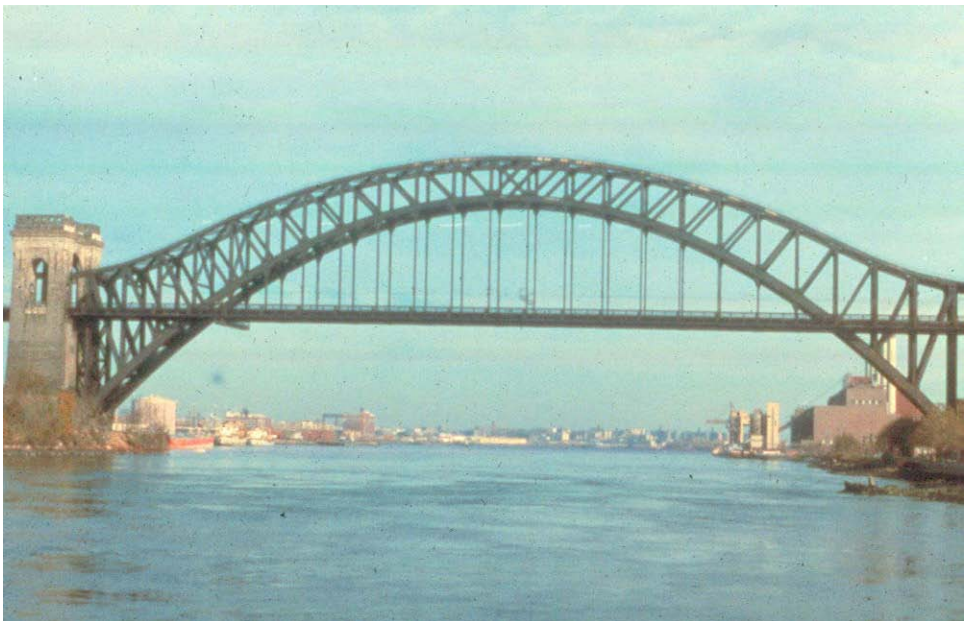


Figure 8.8.2 Through Arch Bridge

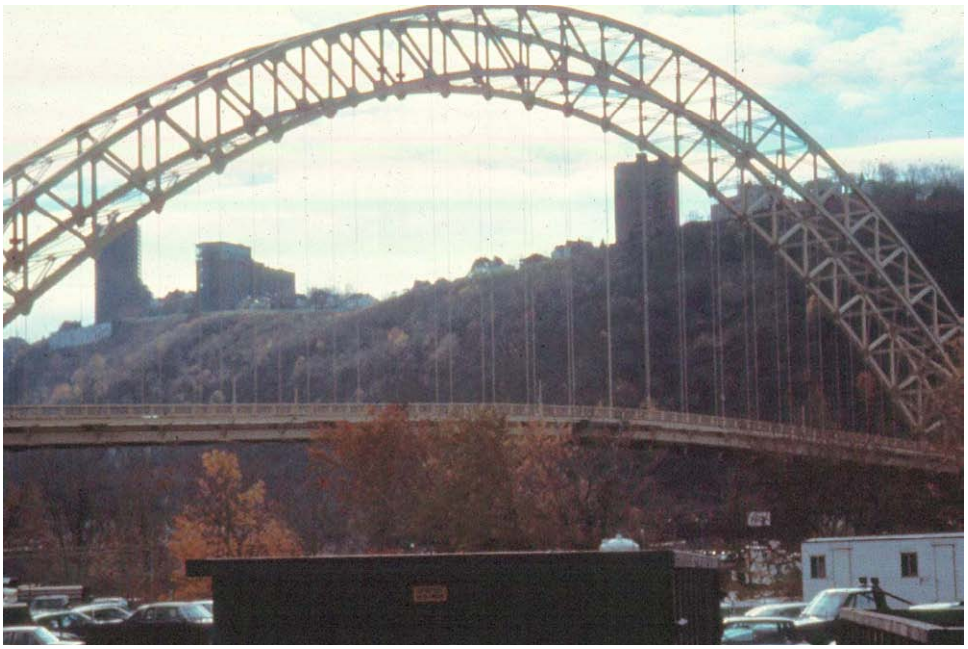


Figure 8.8.3 Tied Arch Bridge

8.8.2 Deck Arch Design Characteristics

General

Like its concrete counterpart, the steel open spandrel arch is designed to resist a load combination of axial compression and bending moment. The open spandrel steel arch is considered a deck arch since the roadway is above the arches (see Figure 8.8.4). The area between the arches and the roadway is called the spandrel. Arch bridges are considered simple spans because of the basic arch function. Steel deck arches can be used in very long spans, measuring up to 518 m (1700 ft).



Figure 8.8.4 Deck Arch

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

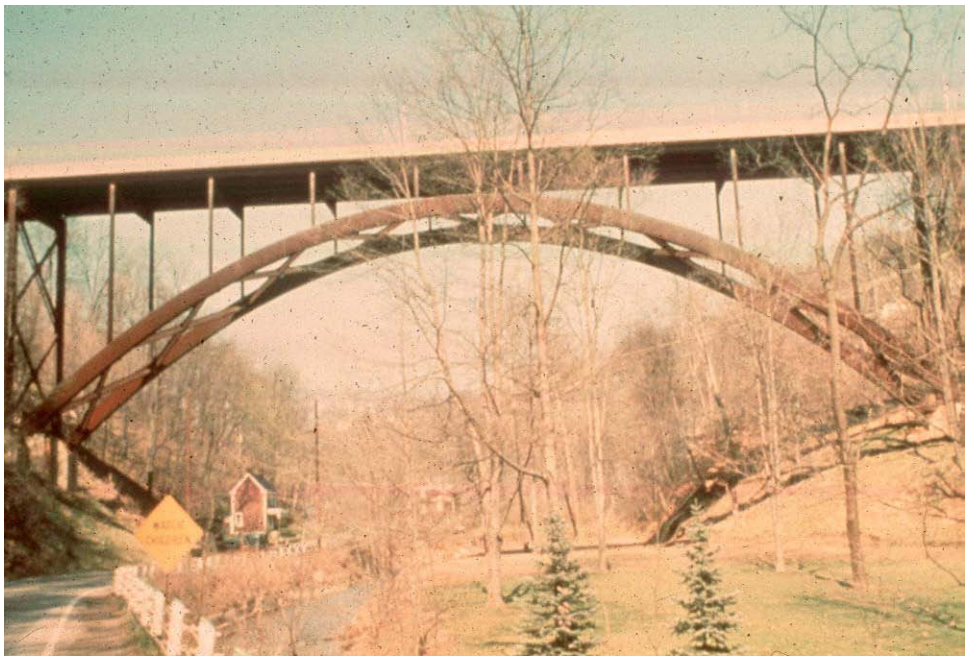


Figure 8.8.5 Solid Ribbed Deck Arch



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

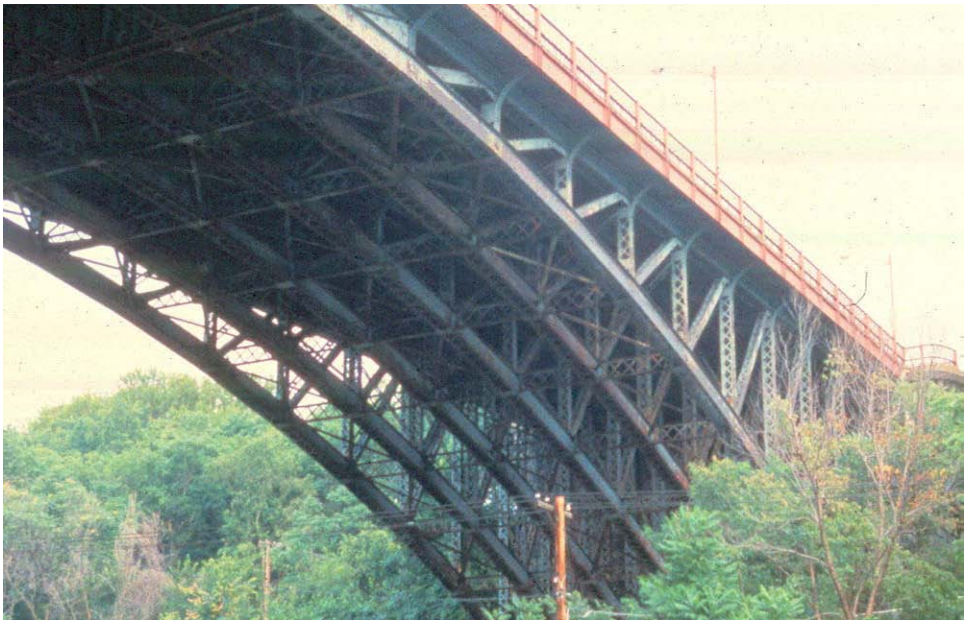


Figure 8.8.7 Spandrel Braced Deck Arch with Six Arch Ribs

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

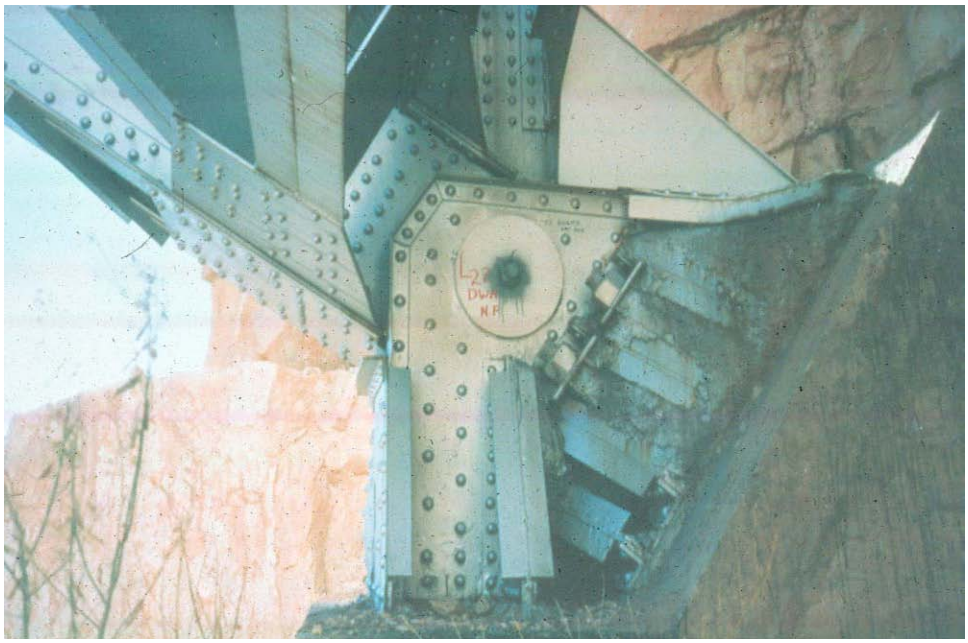


Figure 8.8.8 Hinge Pin at Skewback for Spandrel Braced Deck Arch (Navajo Bridge)

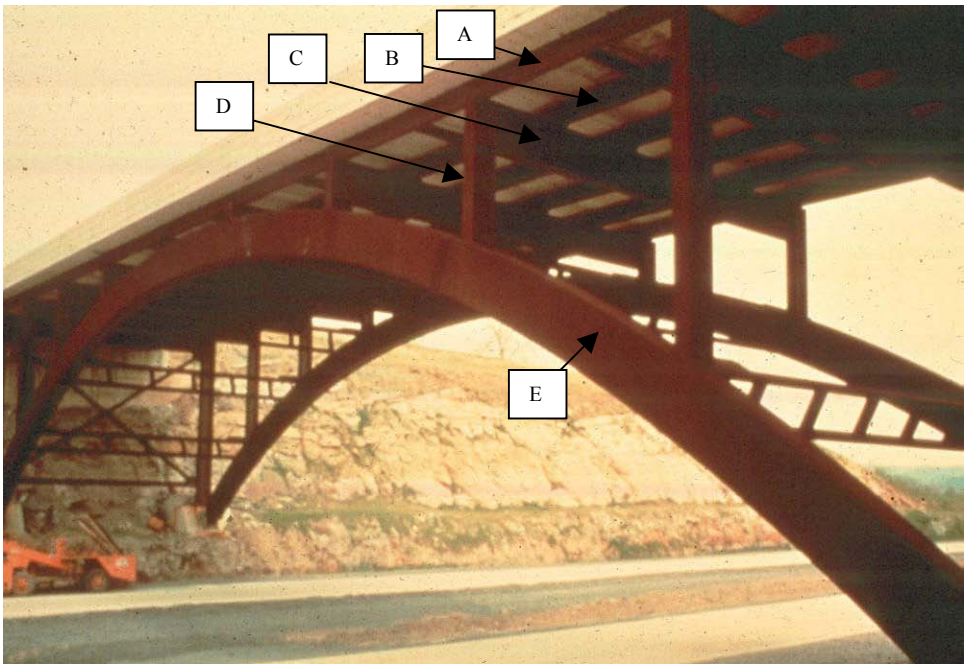
Primary and Secondary Members

The primary members of a deck arch bridge are (see Figure 8.8.9):

- The arches or ribs
- Spandrel columns
- Spandrel girders
- Floorbeams (if present)
- Stringers (if present)

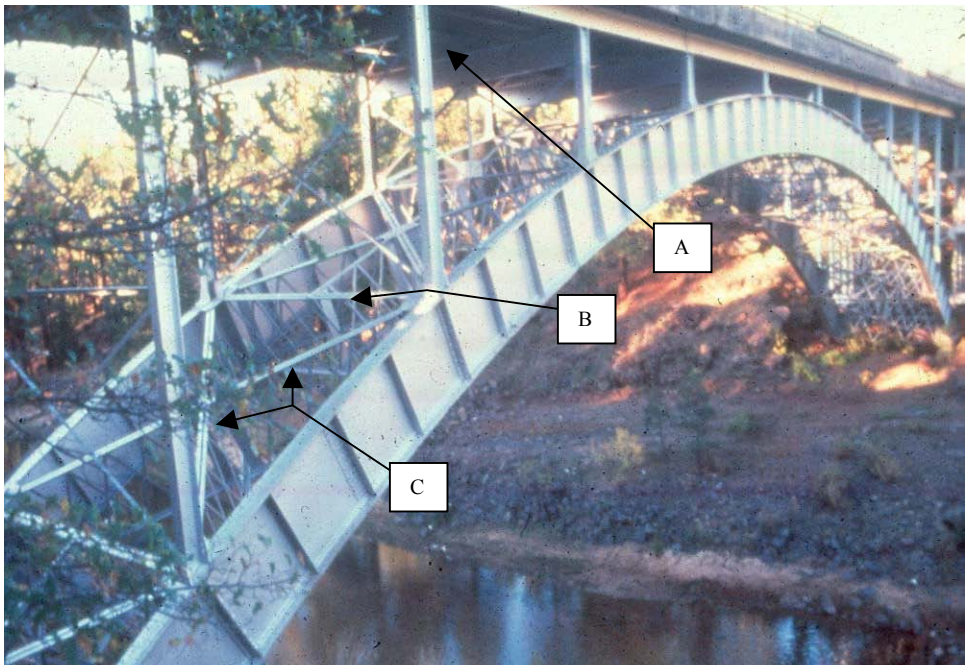
The secondary members of a deck arch bridge are (see Figure 8.8.10):

- Sway bracing
- Upper lateral bracing (bracing in the floor system)
- Lower lateral bracing (bracing in the arch rib)



Primary Members
A – Spandrel girders
B – Stringers
C – Floorbeam
D – Spandrel columns
E – Arch rib

Figure 8.8.9 Solid Ribbed Deck Arch Primary Members



Secondary Members
A – Upper lateral bracing
B – Sway bracing
C – Lower lateral bracing

Figure 8.8.10 Solid Ribbed Deck Arch Secondary Members

Load Transfer

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

Fracture Critical Members

The deck arch bridge has two or more main members. However, the arch is not a tension member and is therefore not considered fracture critical.

8.8.3 Through Arch Design Characteristics

General

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



Figure 8.8.11 Elevation View of a Braced Ribbed Through Arch

Arch bridges are considered simple spans because of the arch function, even though many bridges of this type consist of multiple arches. Through arches are typically two or three hinged.

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

The arch reactions, with their massive horizontal thrusts, are diagonally oriented and transmitted to the foundations at the base of the ribs.

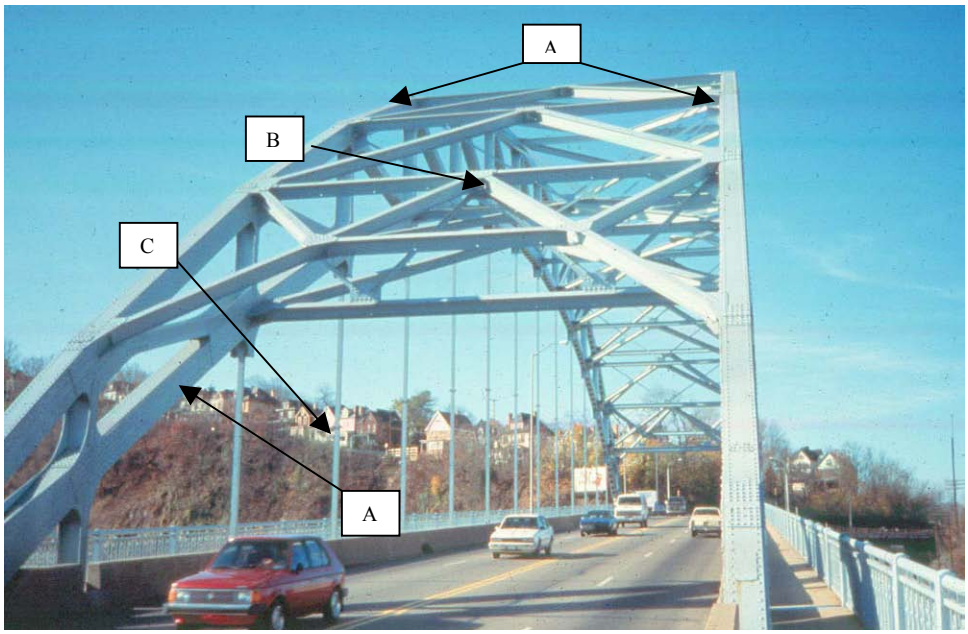
Primary and Secondary Members

The primary members of a through arch bridge are (see Figure 8.8.12):

- Arch ribs (consisting of top and bottom rib chords and rib chord bracing)
- Rib chord bracing
- Hangers
- Floor system

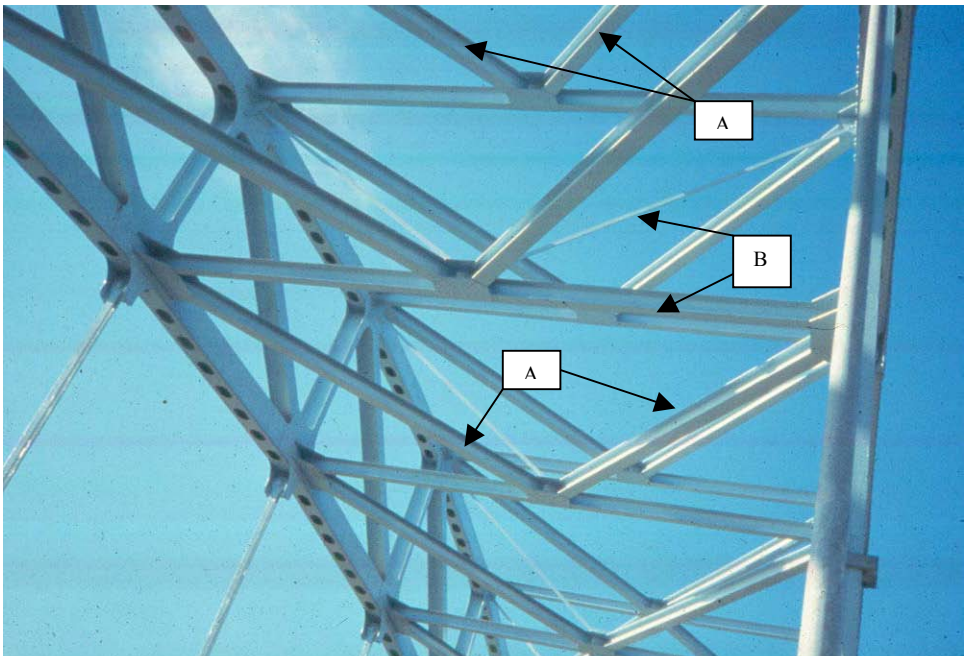
The secondary members of a through arch bridge are (see Figure 8.8.13):

- Sway bracing
- Lateral bracing (top and bottom rib chords)
- Lateral bracing (floor system)



Primary Members
A – Top and bottom rib chords
B - Rib chord bracing
C – Hangers
Floor system not shown

Figure 8.8.12 Through Arch Primary Members



Secondary Members
A – Lateral bracing (top and bottom rib chords)
B – Sway bracing
Lateral bracing in floor system not shown

Figure 8.8.13 Through Arch Bracing (Secondary Members)

Load Transfer

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

Fracture Critical Members

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

8.8.4 Tied Arch Design Characteristics

General

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

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



Figure 8.8.14 Tied Arch

The arch members are, again, called ribs and are usually fabricated box type members. Most of the tied arches found today are solid ribbed, although older tied arches incorporated the braced ribbed type.

The tie member is also typically a fabricated box type member. On large spans, the tie is large enough for the inspector to get inside the box. Some ties consist of truss members. The tie is also supported by hangers, which usually consist of wire rope cable, but can also be built-up members.

The longest steel tied arch in the United States is the Moundsville Bridge over the Ohio River. This structure has a main span of 278 m (912 feet).

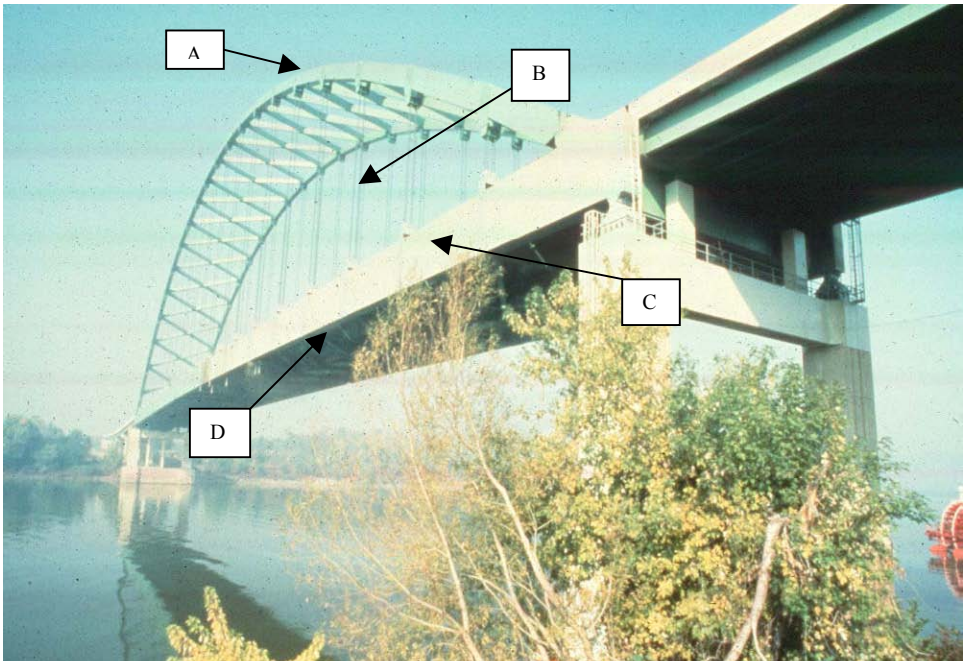
Primary and Secondary Members

The primary members of a tied arch bridge are (see Figure 8.8.15):

- Arch ribs
- Tie members
- Rib bracing truss (if present)
- Hangers
- Floor system

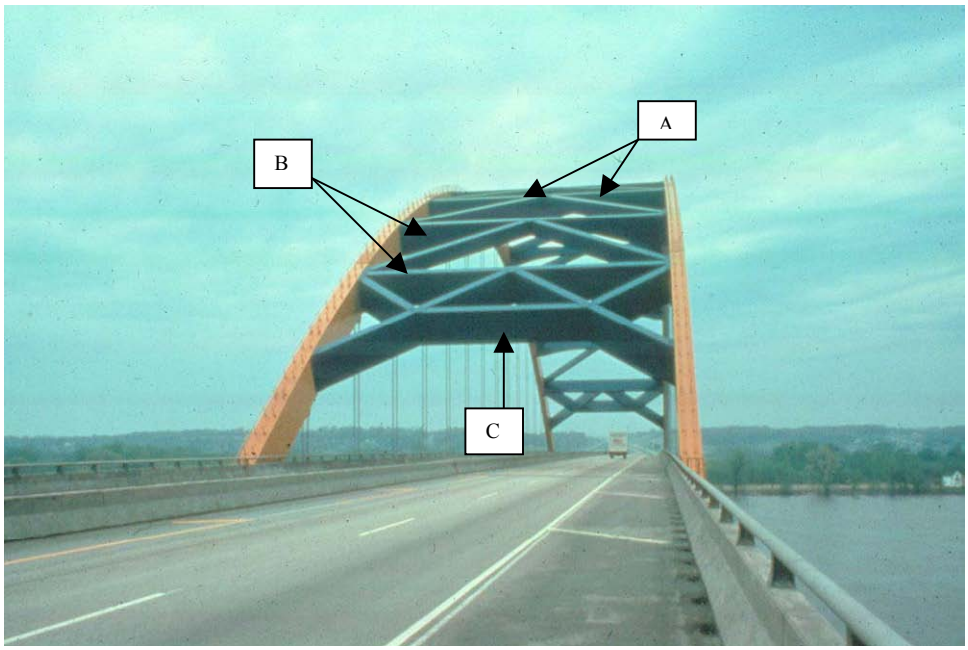
The secondary members of a tied arch bridge are (see Figure 8.8.16):

- Sway bracing
- Lateral bracing (arch rib)
- Lateral bracing (top chord)
- Lateral bracing (floor system)



Primary Members
A – Arch ribs
B – Hangers
C – Tie members
D – Floor system
Rib bracing truss not present.

Figure 8.8.15 Tied Arch Primary Members



Secondary Members
A – Lateral bracing (top)
B – Sway bracing
C – Portal bracing
Lateral bracing in the floor system not shown

Figure 8.8.16 Tied Arch Secondary Members

Load Transfer

Traffic loads are supported by a deck, which resists the loads in bending. The load from the deck is transmitted to the stringers (if present) and then the floorbeams. The stringer and floorbeams resist the traffic load in bending. The load is transferred to the hangers, which are in tension. The arch supports the hangers and transfers the compressive load to the ground at the supports. The horizontal component of the arch rib thrust is then transformed to the tie girders.

Fracture Critical Members

With only two load paths, the tied arch is a nonredundant structure. Like all arch ribs, tied arch ribs are compression members and are not fracture critical. The tie girders, on the other hand, are tension members and are considered fracture critical.

8.8.5 Overview of Common Defects

Common defects that occur on steel arch bridges include:

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Tack welds in tension zones
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

8.8.6 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

In steel arches as well as other steel bridges, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel arch, including:

- Magnetic induction
- Electrical resistivity
- Dye penetrant
- Ultrasonic testing
- Radiographic testing

- Acoustic emission
- Accelerometers
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Topic 13.3 for Advanced Inspection Techniques.

Locations

Arch Members

- Investigate for signs of buckling and crippling in the arch ribs
- Check for general corrosion and deterioration (see Figure 8.8.17)
- Examine the pins for corrosion and wear
- Inspect the alignment of the arch
- Check the arch rib splice plates
- Inspect steel arch girder type bridges as described in Topic 8.2
- Inspect steel arch box girder type bridges as described in Topic 8.5
- Inspect steel braced ribbed arch type bridges as described in Topic 8.6 (see Figure 8.8.18)

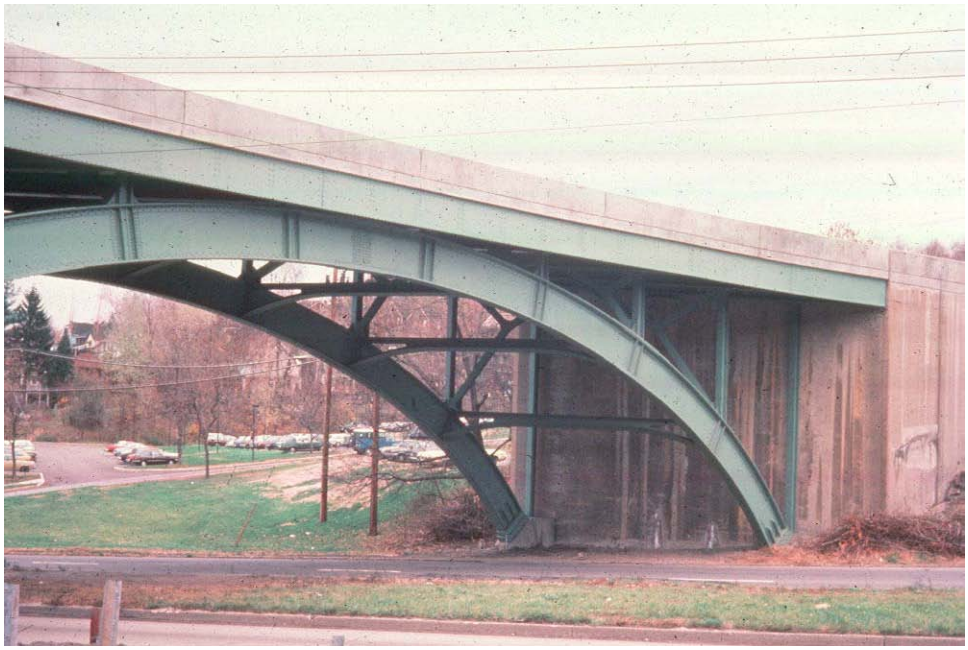


Figure 8.8.17 Solid Ribbed Deck Arch



Figure 8.8.18 Through Truss Arch Members

Bracing (Through and Tied Arches)

- Inspect the web members (posts and diagonals) in a manner similar to any other truss
- Depending on the truss design (e.g., Pratt or Warren), the web members will either be designed for tension, compression, or both
- Examine the top rib chord; this chord will be in compression

Spandrel Members (Deck Arch)

- Examine the end connections of the spandrel columns and spandrel girders for cracks and loose fasteners (see Figure 8.8.19)
- Check the spandrel girders/caps/columns for flexure, section loss, and buckling damage



Figure 8.8.19 Solid Ribbed Deck Arch Showing Spandrel Columns

Hangers (Through and Tied Arches)

- Check the connections at both ends of the hangers, and look for corrosion and cracks (see Figure 8.8.20 and 8.8.21)
- Examine the alignment of the hangers; the hangers may be near traffic, so inspect for collision damage
- Check the hangers for any welded attachment; examine the welds between the attachment and the hanger for cracks



Figure 8.8.20 Hanger Connection on a Through Arch

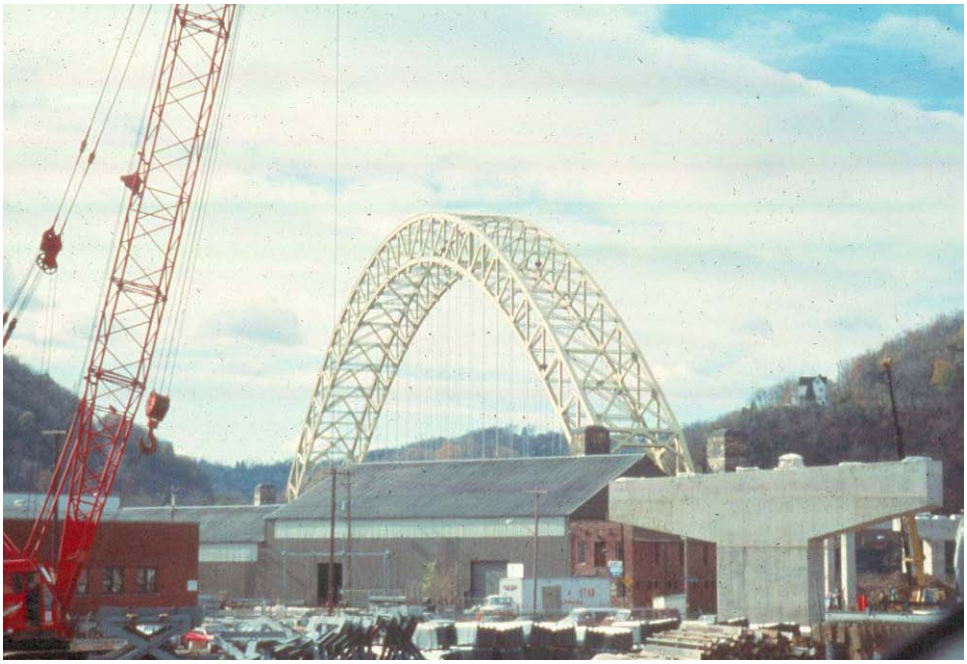


Figure 8.8.21 Exterior View of Hanger Connection on a Tied Arch

Floor System

The floor system should be inspected in the same manner as previously described in Topic 8.3, Steel Two-girder Systems & Steel Through Girder Systems. In addition:

Floorbeams/Stringers

- Inspect the end connections or bearing areas in the floorbeam to girder connections (see Figure 8.8.22).
- Check the stringer end connections.

Tied Arches

- Check floorbeam to tied member connection for distortion induced fatigue cracks in the webs of the floorbeams when the stringers are placed above the floorbeams. The webs of these floorbeams at the connections and adjacent to flanges and stiffeners need to be inspected routinely.



Figure 8.8.22 Floor System on a Through Arch

Tied Girder

- Determine if back-up bars were used to make corner welds in the tie box. If so, the back-up bars should be carefully examined to determine if they are continuous. Weld cracks can occur at points where the bars are discontinuous.
- Check all welds, and examine the ends of any cover plate.
- Especially check welds connecting internal diaphragms to the tie box. Inspect the floorbeam connections and the corner welds of the tie box (see Figure 8.8.23).

Fatigue Prone Details

- Check stiffener welds
- Look for cracks at the web gap details on the floorbeams
- Check the welds for cracks at the floorbeam and stringer connections
- Check for tack welds on the floor system and trussed brace
- Check any cover plate ends on the floor system members
- Check the welds and adjacent to tension webs and flanges for cracks
- Check back-up bar

- Check for cracks at any intersecting welds
- Check for discontinuous or intermittent welds
- Check diaphragm connection plate welds

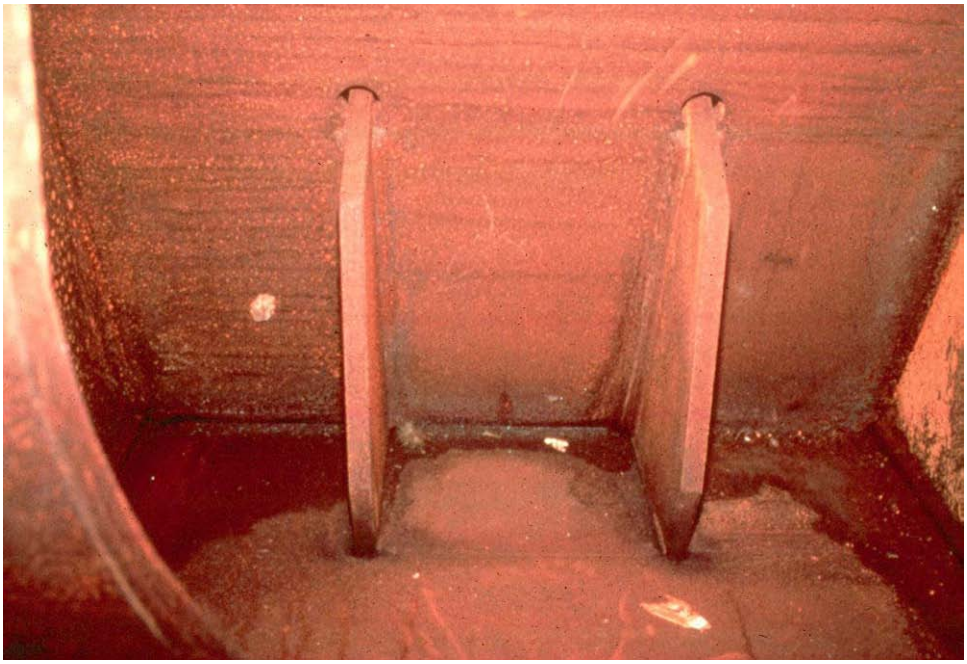


Figure 8.8.23 Tie Girder Interior

Out-of-plane Distortion

- Investigate the girders at the floorbeam connection for cracks in the webs due to out-of-plane distortion. For additional information on floor systems, see Topic 8.3, Steel Two-girder Systems & Steel Through Girder Systems.

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

Another region of web-diaphragm junction in box girders, which has incurred distortion induced fatigue cracks, is at partial depth diaphragms inside the tie girder of tied arch bridges (see Figure 8.8.24). The diaphragms are usually in alignment with the floorbeams, and the primary function is to resist deformation of the box section. However, the small, unstiffened regions of the webs do not have sufficient rigidity against lateral, out-of-plane distortion. The situation is analogous to that of webs at unattached ends of floorbeam or diaphragm connection plates. Fatigue cracks can develop at the ends of the diaphragm plates. The ends of all partial depth diaphragms in a box tie girder should be inspected.

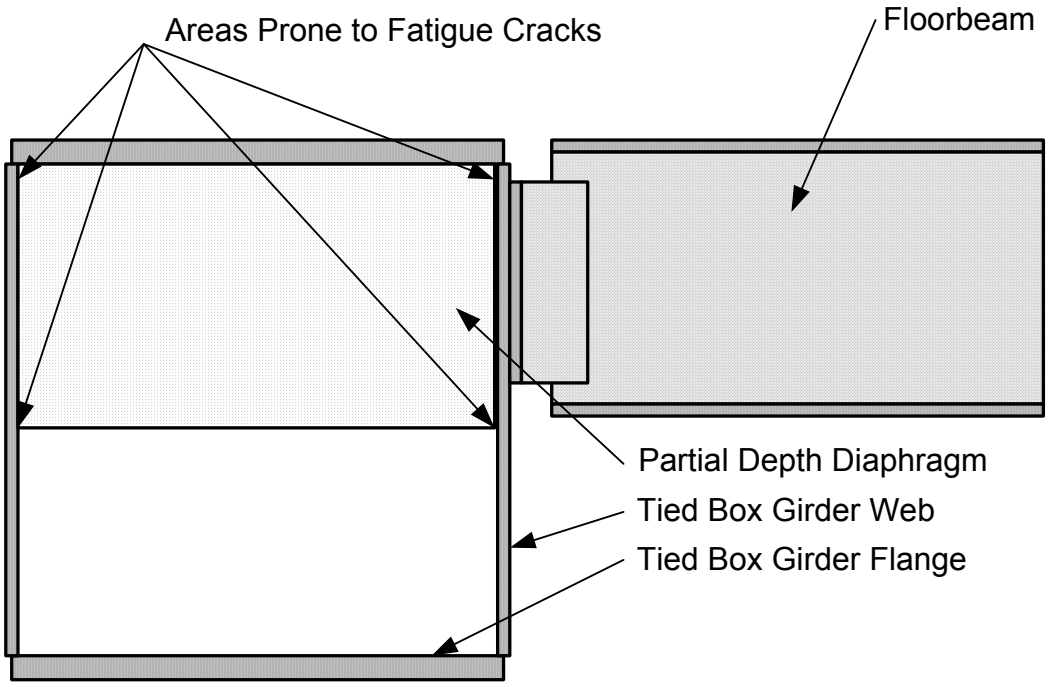


Figure 8.8.24 Partial Depth Diaphragm in a Tied Box Girder

Secondary Members

The secondary members include:

- Sway bracing
- Bracing in the floor system
- Bracing in the arch rib

The bracing systems should be inspected using methods similar to those previously described in Topics 8.3, 8.5, and 8.6 (see Figure 8.8.25). In addition:

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

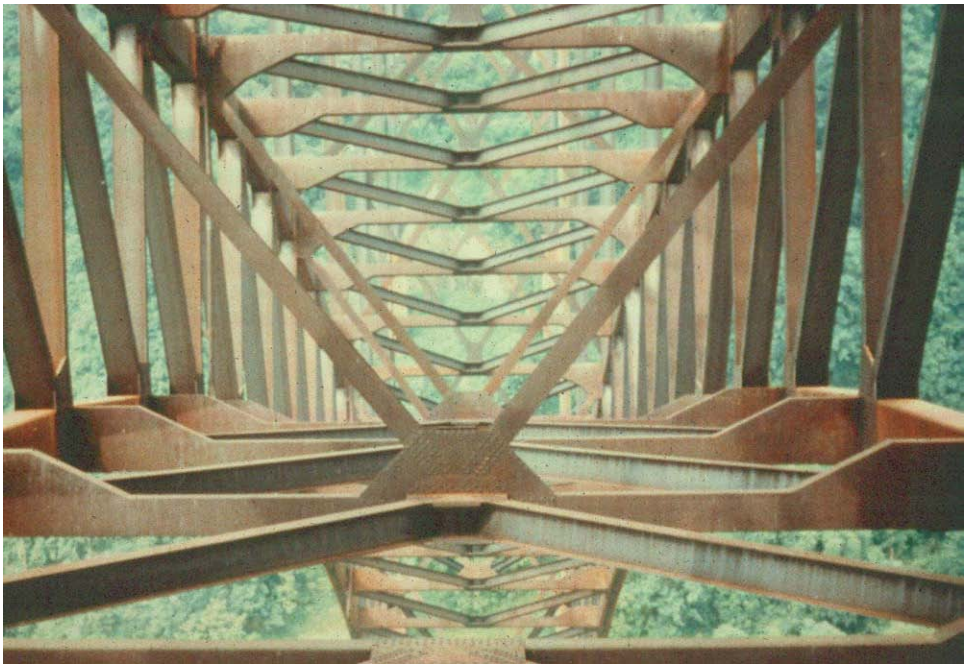


Figure 8.8.25 Bracing Members (New River Gorge Bridge)

Areas Exposed to Traffic

- Inspect any areas exposed to traffic for collision damage (see Figure 8.8.26). If collision damage is found, document the location and dimensions along with photos.



Figure 8.8.26 Areas Exposed to Traffic

Areas Exposed to Drainage

The areas that trap water and debris result in active corrosion cells, which cause loss of section and notches susceptible to fatigue. On arch bridges check:

- Lateral bracing gusset plates
- Pockets created by floor system connections
- Areas exposed to drainage runoff

Bearing Area

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

Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. (see Topic 9.1).

8.8.7 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the element level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the arch. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element level Smart Flags are also used to describe the condition of the steel superstructure, both unpainted and painted.

In an element level condition state assessment of a steel arch bridge, the AASHTO CoRe elements may be:

Element No.	Description
101	Unpainted Steel Closed Web/Box Girder
102	Painted Steel Web/Box Girder
106	Unpainted Steel Open Girder/Beam
107	Painted Steel Open Girder/Beam
112	Unpainted Steel Stringer (Stringer Floorbeam System)
113	Painted Steel Stringer (Stringer Floorbeam System)
140	Unpainted Steel Arch
141	Painted Steel Arch
151	Unpainted Steel Floorbeam
152	Painted Steel Floorbeam
160	Unpainted Steel Pin and Hanger Assembly

161	Painted Steel Pin and Hanger Assembly
201	Unpainted Steel Columns
202	Painted Steel Columns

The unit quantity for the arch is meters or feet and the total length of the arch ribs must be placed in one of the four available condition states for unpainted and five available condition states for painted. The unit quantity for the floor system is meters or feet and the total length of floor beams and stringers must be placed in the 4 or 5 available condition states (see Topic 8.3). The unit quantity for columns is each and the total quantity must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

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Topic 8.9 Steel Rigid Frames

8.9.1 Introduction

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



Figure 8.9.1 Typical Rigid K-frame Constructed of Two Frames

8.9.2 Design Characteristics

General

Because of their unique shape and characteristics, frames are not referred to as having a single, simple, multiple, or continuous span. Also, steel rigid frame structures are used only in straight horizontal applications.

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

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



Figure 8.9.2 Typical Rigid Frame Constructed of Multiple Frames

Rigid frames can have features similar to fabricated girder bridges such as:

- A floor system of floorbeams and stringers
- Web stiffeners
- Interior diaphragms

K - Frames

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



Figure 8.9.3 Typical K-frame

It is possible to think that the legs of the K-frame look very much like piers and consider them part of the substructure. This is not the case for the following reasons:

- A bearing separates the superstructure and substructure. There is no bearing between the legs and the girder portion of the frame (see Figure 8.9.3).

- Bending forces are transferred between the girder portion and the legs. No properly functioning bearing, if one existed, would permit such a transferring of moment (see Figure 8.9.4).



Figure 8.9.4 Connection Between Legs and Girder Portion of a K-frame

Delta Frames

In some designs, a triangular frame configuration can be used. For very long spans, two K-frames can be connected together end-to-end (see Figure 8.9.5). Instead of one of the end spans bearing on an abutment, it is connected to the end span of another K-frame. The bottoms of the legs are also connected together and share the same bearing (see Figure 8.9.6). This type of configuration is known as a delta frame. The end span leg connections form an inverted triangle: the Greek letter Delta (∇).



Figure 8.9.5 Typical Delta Frame



Figure 8.9.6 Leg Connections and Bearing Area on a Delta Frame

Bearings

Regardless of the frame configuration, the entire portion of the bridge, which constitutes the frame, is considered the superstructure. The legs of rigid frames are supported by relatively small concrete footings and bearings which are essentially hinges (see Figure 8.9.7).



Figure 8.9.7 Hinge Bearing

Primary Members

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

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

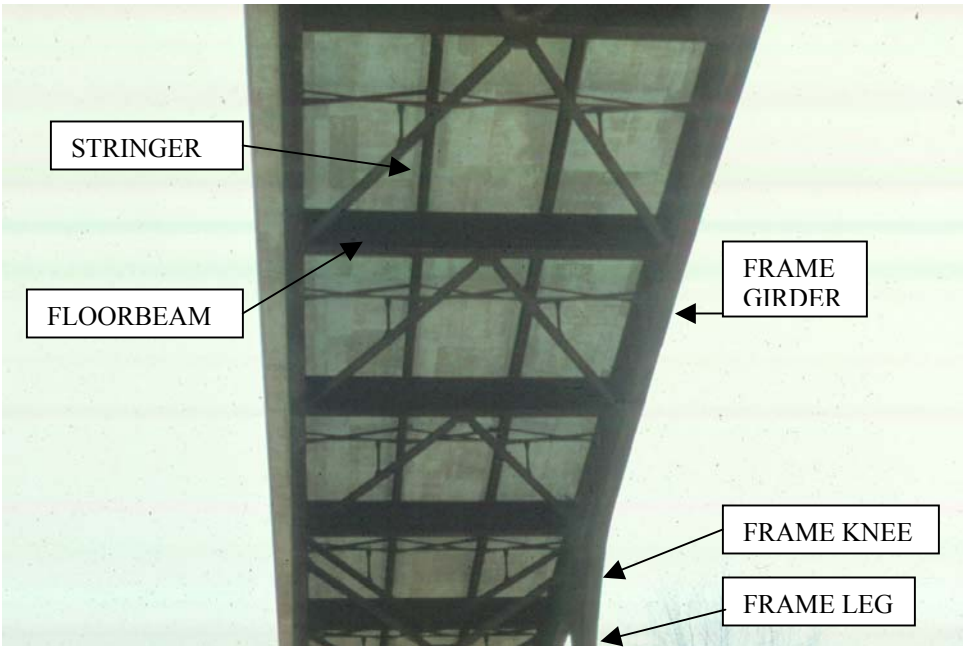


Figure 8.9.8 Frame Members, Floorbeams, and Stringers

Stiffeners

Steel rigid frames may have up to three different types of stiffeners (see Figure 8.9.9).

Transverse Stiffeners

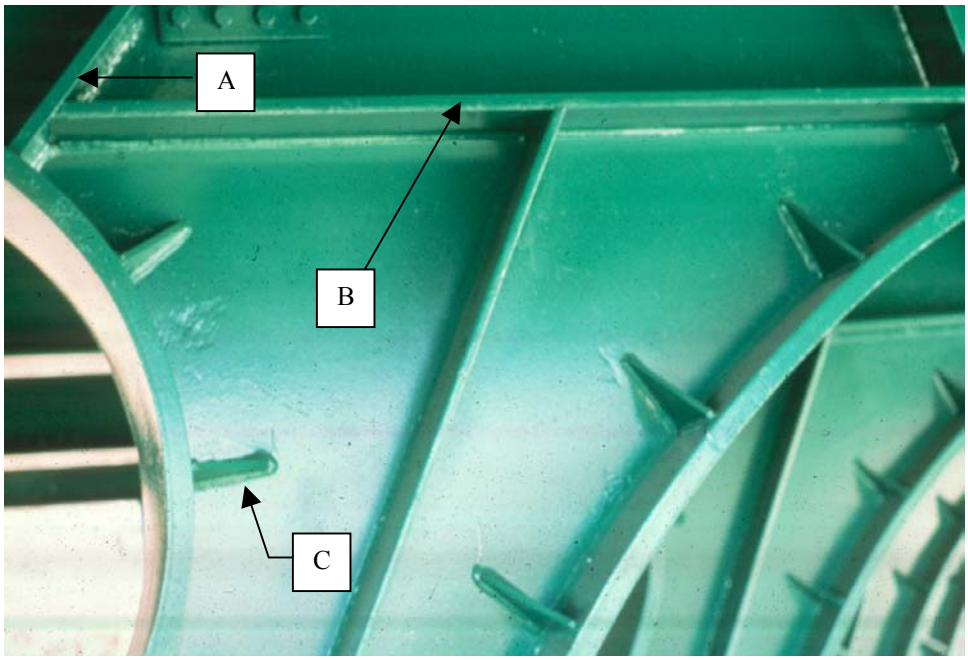
Transverse stiffeners are placed approximately perpendicular to the flanges and welded to the web and flanges of the frame at spacings required by design. Transverse stiffeners are used to prevent buckling in high shear regions.

Longitudinal Stiffeners

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

Radial Stiffeners

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



Stiffeners
A – Transverse
B – Longitudinal
C – Radial

Figure 8.9.9 Transverse, Longitudinal, and Radial Stiffeners on a Frame Knee

Floor System Arrangements

A rigid frame will have one of three floor systems:

Multiple Frame System

For a multiple frame system, the deck is supported only by the frames (see Figure 8.9.10).

Frame-Floorbeam System

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

Frame-Floorbeam-Stringer System

This system consists of floorbeams connected to the girder portion of the two frames. Longitudinal stringers, parallel to the frames, are connected to the floorbeams (see Figure 8.9.11). The stringers are typically rolled sections and are connected to the web of the floorbeams. This arrangement typically has a wider rigid frame spacing than the frame-floorbeam system.



Figure 8.9.10 Deck Underside on a Multiple Frame Rigid Frame



Figure 8.9.11 Two Frame Bridge with Floorbeam-Stringer Floor System

Secondary Members

Lateral Bracing

In a two or three frame system, members are placed diagonally between the frames. This bracing restricts any differential and transverse movements between the frames. This bracing is in the plane of the bottom flange of the girder portion of the frame or between the legs of the frame (see Figure 8.9.12).

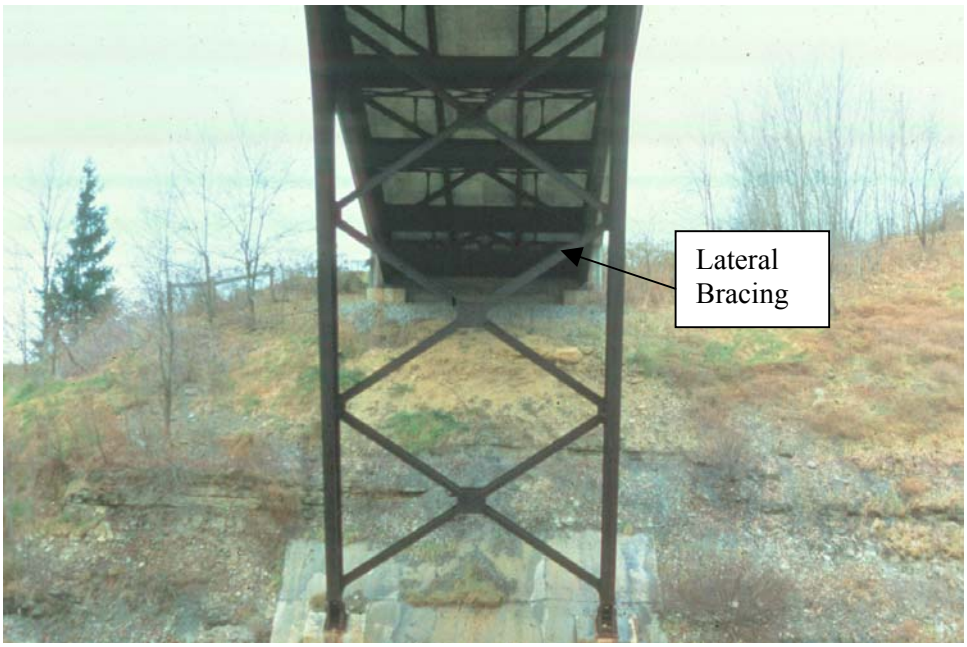


Figure 8.9.12 Lateral Bracing

Diaphragms

In a multiple frame system diaphragms are placed perpendicular between the frames. This bracing minimizes any transverse movements between the frames (see Figure 8.9.13).

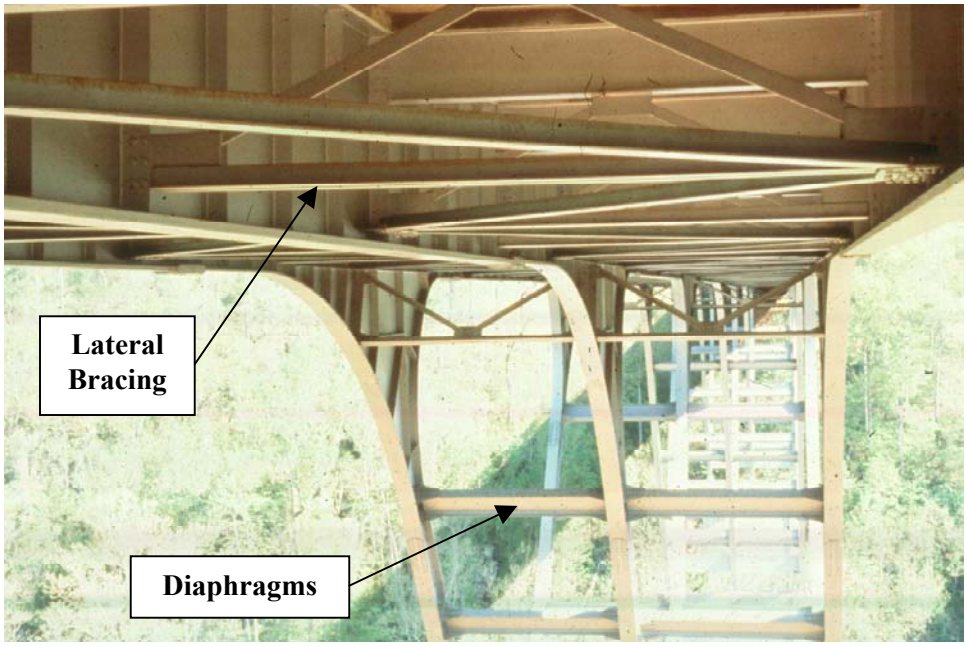


Figure 8.9.13 View of Lateral Bracing and Diaphragms

Stress Zones

Each element of the frame resists various levels of stress due to moment and shear. Tension zones are similar to those for concrete rigid frames (see Figure 8.9.14).

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

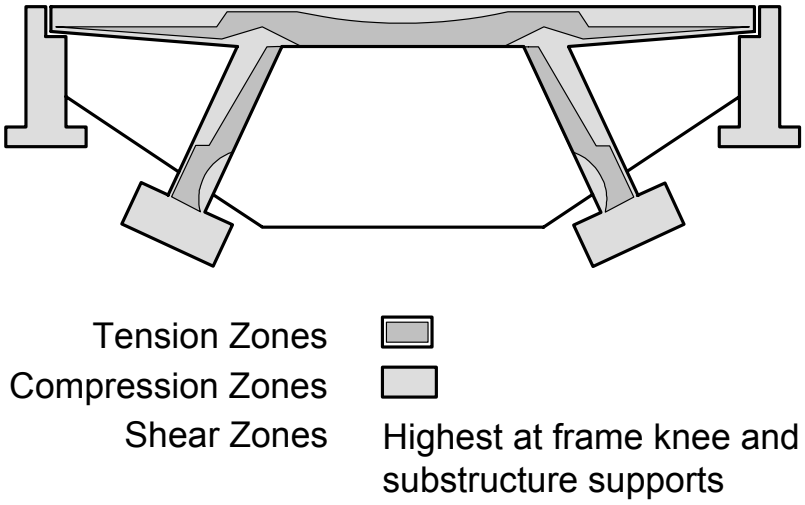


Figure 8.9.14 Stress Zones in a Frame

Fracture Critical Members

A rigid frame consisting of two frames does not have load path redundancy and is therefore non-redundant. This means that a two frame steel rigid frame is a fracture critical bridge type (see Figure 8.9.15).



Figure 8.9.15 Dual Frame Rigid Frame – A Fracture Critical Structure

A rigid frame bridge consisting of three or more frames has load path redundancy and is not fracture critical (see Figure 8.9.16).

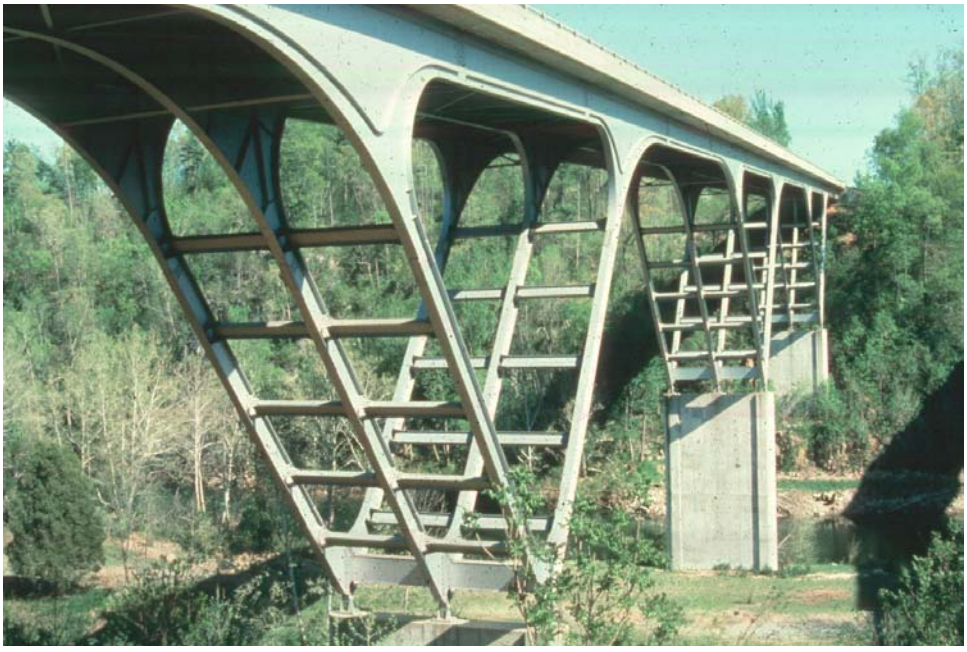


Figure 8.9.16 Multiple Frame Rigid Frame – Not a Fracture Critical Structure

8.9.3 Overview of Common Defects

Common defects that can occur on steel rigid frame bridges include:

- Failure of the paint system
- Pitting
- Surface rust
- Section loss
- Fatigue cracking
- Collision damage
- Overload damage
- Heat damage
- Buckling

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

8.9.4 Inspection Procedures and Locations

Procedures

Visual

Most cracks 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. Once a crack is found, other nondestructive inspection methods, such as dye penetrant, magnetic particle, ultrasonic testing, eddy current, or radiography, are used to further clarify the extent of the crack (refer to Topic 13.3). More exact visual observations can also be employed using a magnifying unit after cleaning the paint from the suspect area.

Physical

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

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

Advanced Inspection Techniques

In steel rigid frames, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel rigid frame, including:

- Acoustic Emissions Testing
- Computer Programs
- Computer Tomography
- Corrosion Sensors
- Dye Penetrant
- Magnetic Flux Leakage
- Radiographic Testing
- Robotic Inspection
- Ultrasonic Testing
- Eddy Current
- Brinell Hardness Test
- Charpy Impact Test
- Chemical Analysis
- Tensile Strength Test

See Topic 13.3 for Advanced Inspection Techniques.

Locations

Bearing Areas

- Check the bearing areas of the frame, including bearing stiffeners, for corrosion, section loss, buckling, or malfunctions of the bending device (see Figure 8.9.17).



Figure 8.9.17 Bearing Area of a Two Frame Bridge

Shear Zones

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

Flexure Zones

- Check the tension and compression flanges for corrosion, section loss, cracks or buckling (see Figure 8.9.18).
- Special attention should be given to the flanges at the connection between the legs and girder portion of the beam. Bending moment is at its highest in this area.



Figure 8.9.18 General Elevation View



Figure 8.9.19 Diaphragm Connection

Fatigue Prone Details

- Check any attachment welds located in the tension areas of the frame.
- Inspect frame leg hinges for frozen or worn pins.
- Check web stiffener welds
- Check welded flange splices, particularly where changes in thickness and/or width occur (see Figure 8.9.20)
- Check intersecting welds
- Pay close attention to the knee area. Vertical, longitudinal, and radial stiffeners may all be present.

Areas Exposed to Traffic

- Inspect frame girder sections over roadways for collision damage
- Check for collision damage if the bridge crosses over a highway, railway, or navigable channel; document any cracks, section loss, or distortion found

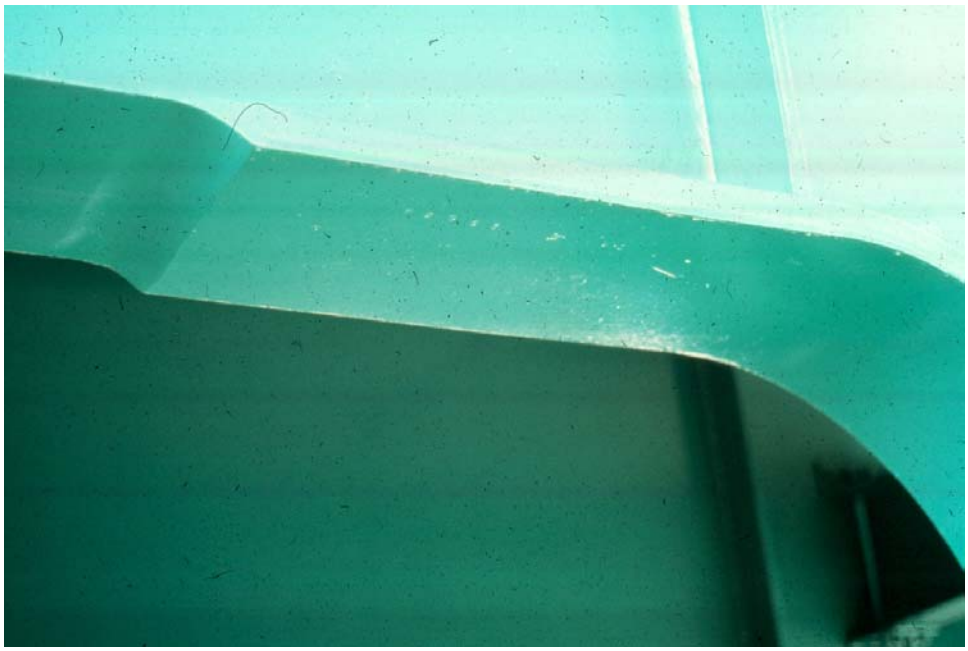


Figure 8.9.20 Welded Flange Splice on a Delta Frame

Areas Exposed to Drainage

The areas that trap water and debris result in active corrosion cells and can cause in notches susceptible to fatigue or perforation and loss of section. On rigid frame bridges check:

- lateral bracing gusset plates
- pockets created by floor system connections

Secondary Members

- Check horizontal connection plates which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Investigate the areas beneath drainpipes and deck joints for corrosion from exposure to roadway drainage.
- Check the connection areas of the lateral bracing or diaphragms for cracked welds, fatigue cracks, and loose fasteners (see Figure 8.9.19).
- Check for distortion in the bracing members.

8.9.5 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the rigid frame. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element Level Smart Flags are also used to describe the condition of the steel superstructure, both unpainted and painted.

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

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

For the AASHTO CoRe elements, the unit quantity for the rigid frame is meters or feet and the total length of the frames must be placed in one of the four available condition states for unpainted and five available condition states for painted. The unit quantity for columns is each and the total quantity must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For states that create their own non-CoRe elements, the bridge inspector must use that particular state's Bridge Inspection Manual to figure out the appropriate non-CoRe element(s) as well as the correct condition state(s).

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

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Section 9
Inspection and Evaluation of Bridge Bearings

Topic 9.1 Bridge Bearings

9.1.1 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 permit longitudinal movement of the superstructure due to thermal expansion and contraction (expansion bearings only)
- To allow rotation caused by dead load and live load deflection.

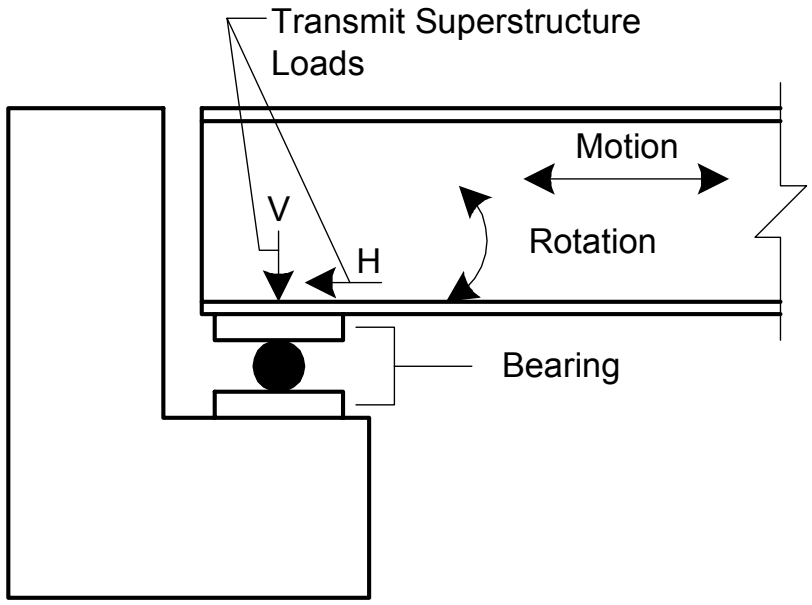


Figure 9.1.1 Functions of a Bearing

Fixed and Expansion Bearings

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 expansion bearings. Both fixed and expansion bearings permit rotation that occurs as loads are applied or removed from the bridge.



Figure 9.1.2 Fixed and Expansion Bearings

This chapter identifies the various types of bridge bearings, their elements, and how they function in relation to bridge structures. It also describes the inspection and evaluation of the various types of bearings.

9.1.2 Four Basic Elements of a Bearing

A bridge bearing consists of four basic elements:

- Sole plate
- Bearing or bearing surface
- Masonry plate
- Anchorage

Sole Plate

The sole plate is a steel plate that is attached to the bottom flange of girders or beams or to the bottom chord of trusses. A sole plate may also be embedded into the bottom flange of a prestressed girder. With concrete beams, girders, or slabs, the lower flange or bottom of the section may function as a sole plate.

Bearing or Bearing Surface

The bearing or bearing surface 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 above to the substructure unit.

Anchorage

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 expansion bearing element to provide restraint against transverse movement. The local or governing agency requirements should be checked to determine the minimum bolt diameter and the minimum embedded length.

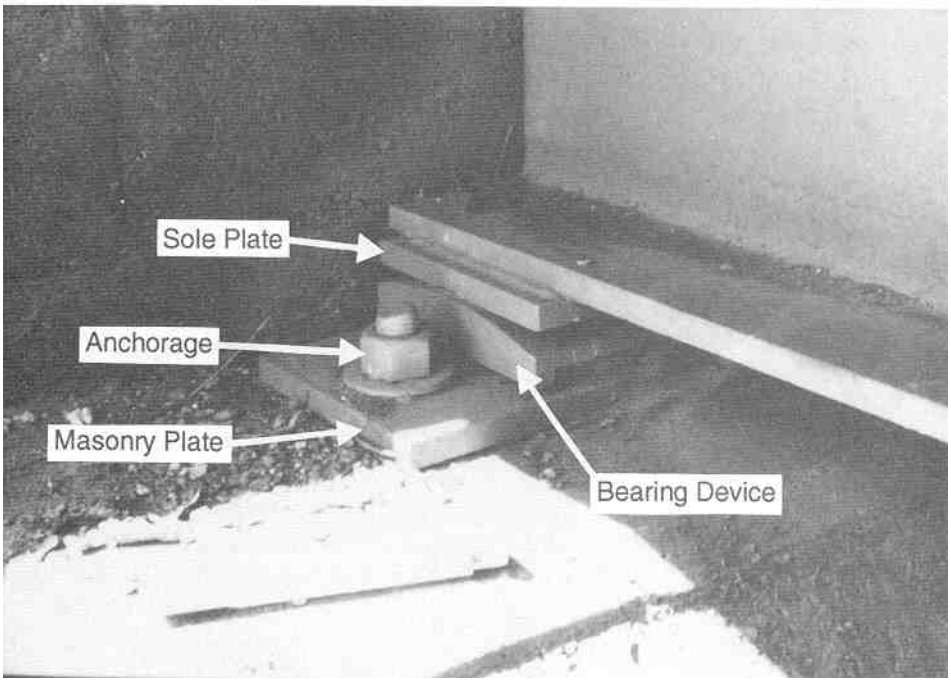


Figure 9.1.3 Elements of a Typical Bridge Bearing

Not all bearings have all four of these distinct elements. All bearings do, however, have at least one of them: the bearing surface.

9.1.3 Bearing Types and Functionality

Various expansion bearing types have evolved out of the need to accommodate superstructure movement. These bearings include:

- Sliding plate bearings
- Roller bearings
- Rocker bearings
- Pin and link bearings
- Elastomeric bearings
- Seismic Bearings
- Pot bearings
- Restraining 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 of less than 12.2 m (40 feet). Longitudinal movement is provided by one plate sliding upon another. The basic difference in the various types 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 surfaces planed smooth. Lubrication between the plates consisted of grease, graphite, and tallow. Unfortunately, the lubricant tended to hold dirt which absorbed moisture, eventually corroding and freezing the bearing. ("Freezing," as used in describing bearings, indicates that the bearing has become inoperable due to corrosion, mechanical binding, dirt buildup, or other interference. The bearing can not move as intended.)

The next generation of sliding 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 would overcome the freezing forces. However, the smaller plate actually wore a groove in the larger one, eventually freezing the bearing anyway.

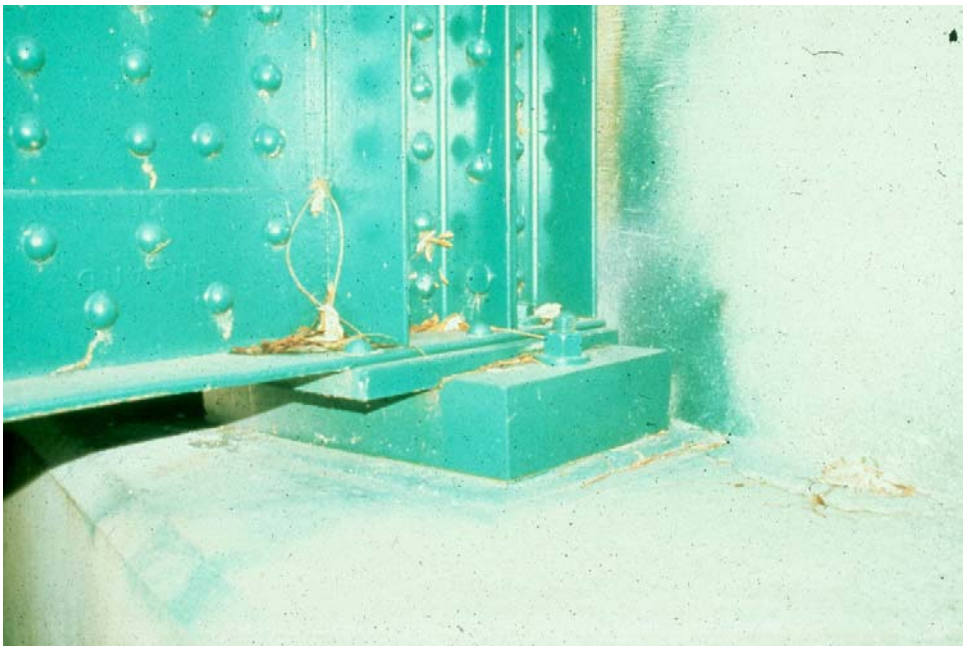


Figure 9.1.4 Lubricated Steel Plate Bearing

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. Since it does not corrode, bronze 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 mechanical locking of the plates may take place.



Figure 9.1.5 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 of less than 12.2 m (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. If the bearings are kept free from dirt and abrasive dusts, this can be true.

These bearings are widely available in many different forms, including plates, plates with one side cut to a radius, and half cylinders. The flat side provides translation movement, and rotational movement is provided by the radius side. (See Figure 9.1.6).

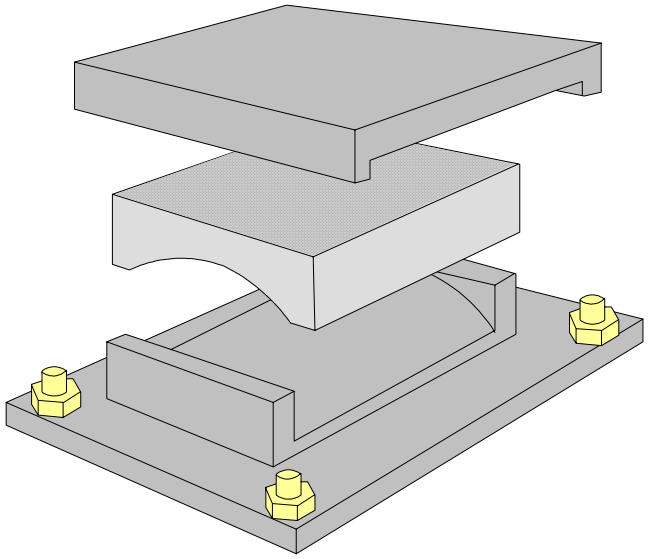


Figure 9.1.6 Self-lubricating Bronze Sliding Plate Bearing

Roofing Felt / Tar Paper

Another type of bearing consists of oil-soaked felt or tar paper that has been dusted with graphite. Several layers are placed on the abutment 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 movement.

PTFE on Stainless Steel Plates

A compound known as "polytetrafluoroethylene" (PTFE or TFE) 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 must be combined with suitable fillers. These fillers are typically glass fiber and bronze and, while giving strength to the PTFE, they do not change its low coefficient of friction.

Roller Bearings

A roller bearing consists of a cylinder, which “rolls” between the sole plate and masonry plate as the superstructure expands, and contracts. 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 bearings. Rollers can vary in size, with specified diameters ranging from about 150 to 380 mm (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, especially those supporting concrete superstructures.

An unrestrained roller will gradually work itself out from underneath a bridge. 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 9.1.7 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 38 to 50 mm (1½ 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 and have usually met with little success. (See Figure 9.1.8).

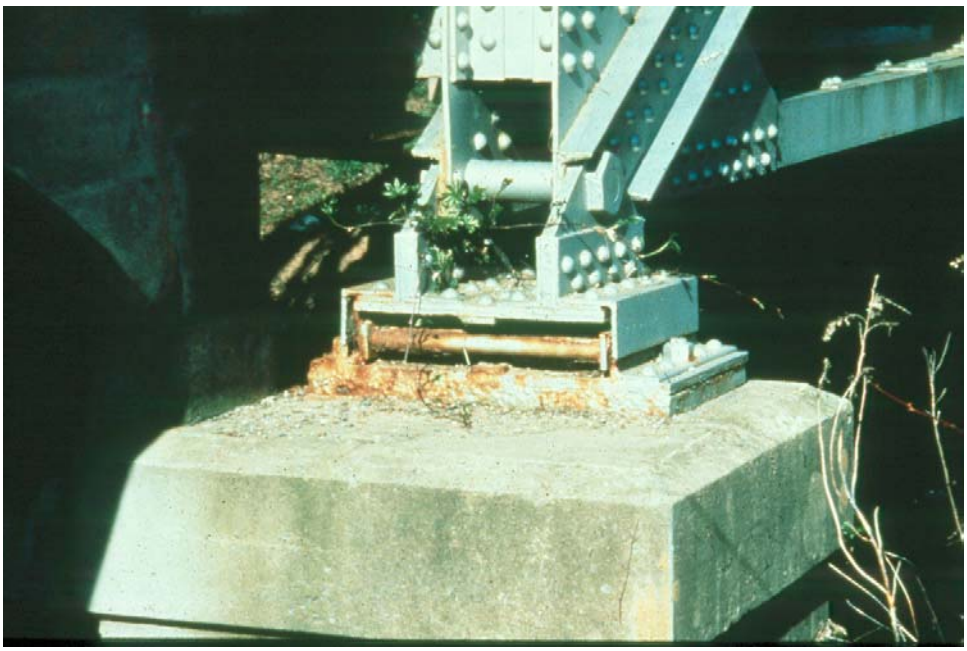


Figure 9.1.8 Roller Nest Bearing

Rocker Bearings

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

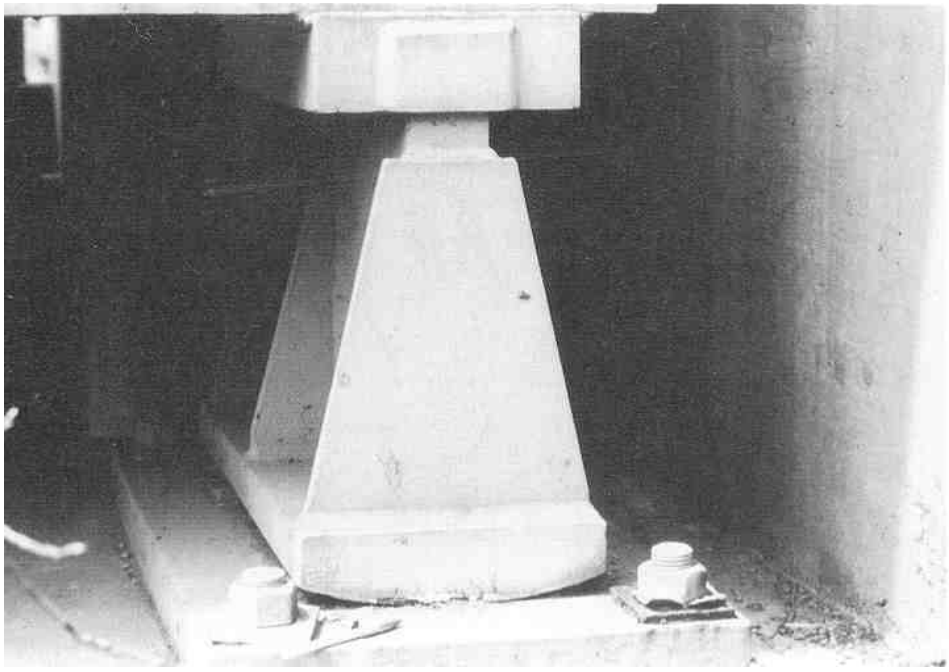


Figure 9.1.9 Rocker Bearing

Segmental Rocker Bearings

Segmental rocker bearings were the next major development in bearing devices. These rockers evolved out of the use of large rollers. When the rollers get up to 510 mm (20 inches) in diameter, they become very heavy and difficult to handle. Since only a small portion of the roller is actually used, the unused portion can be cut away and a substantial weight savings can be realized.

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



Figure 9.1.10 Segmental Rocker Bearing

Rocker Nest Bearings

A group of several rockers forms a rocker nest. Rocker nests provide many small areas for dirt and moisture to collect, similar to roller nest bearings.



Figure 9.1.11 Segmental Rocker Nest Bearing

Pinned Rocker Bearings

The pinned rocker is the most popular rocker bearing design today. The top is basically a large pin and tends to keep the bearing aligned correctly. Longitudinal movement is provided by the rotation afforded by the pin and the rolling provided by the rocker. 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.

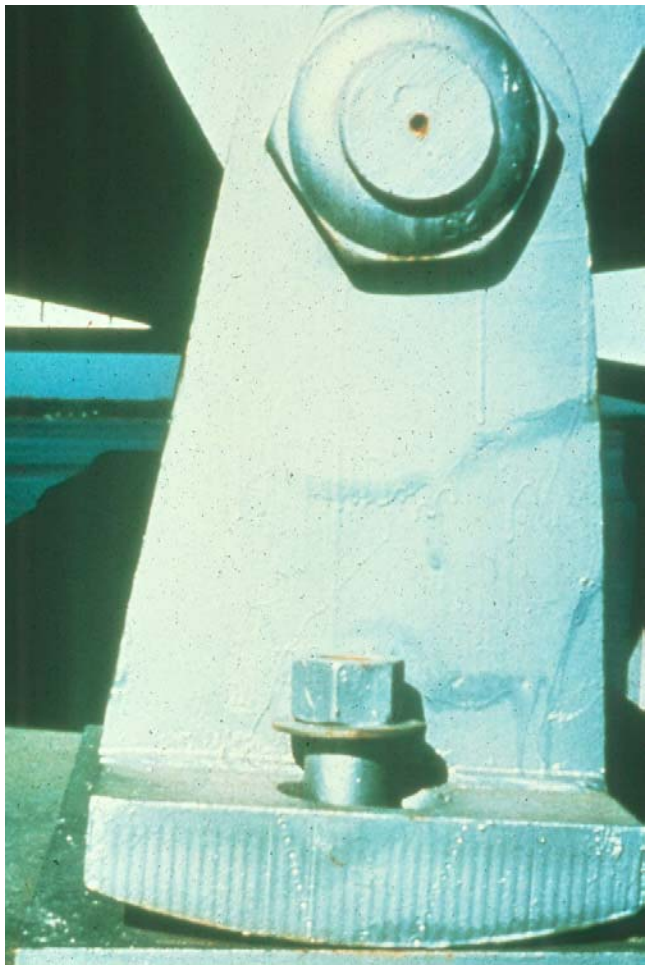


Figure 9.1.12 Pinned Rocker Bearing

Pin and Link 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 section. It consists of two vertically oriented steel plates pinned at the top and bottom to allow longitudinal movement. 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 9.1.13 Pin and Link Bearing

Elastomeric Bearings

Elastomeric bearings include both plain and laminated neoprene pads.

Plain Neoprene Pads

A plain neoprene bearing consists of a rectangular pad of pure neoprene and is used primarily on short span, prestressed concrete structures. Neoprene bearings are popular for steel beam bridges as well. Expansion and contraction are achieved through a shearing deformation of the neoprene.

Typically these bearings are of uniform thickness and are rectangular with parallel sides, but round, disc-shaped pads have also been used.

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 9.1.14 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. 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 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 would dramatically decrease its useful life. The laminated bearing eliminates this excess bulging and allows the expansion and contraction without excessive wear.

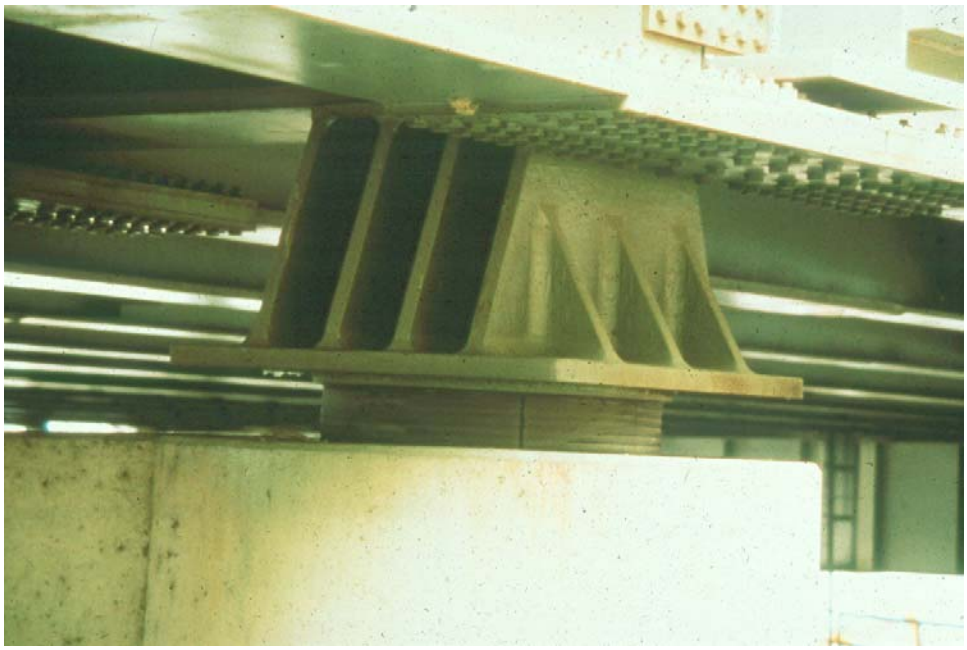


Figure 9.1.15 Laminated Neoprene Bearing Pad

Seismic Bearings

Isolation Bearings

The isolation bearing was developed to protect structures against earthquake damage. It is similar to the laminated bearing in that it is a sandwich of neoprene and steel plates. It also contains a lead core that is used primarily for seismic loads. A cover of neoprene protects the steel plates but the top of the lead core remains exposed.

The isolation bearing behaves like a laminated bearing when exposed to normal bridge loading. The lead core stiffens the bearing and helps it to resist these loads. However, under 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.

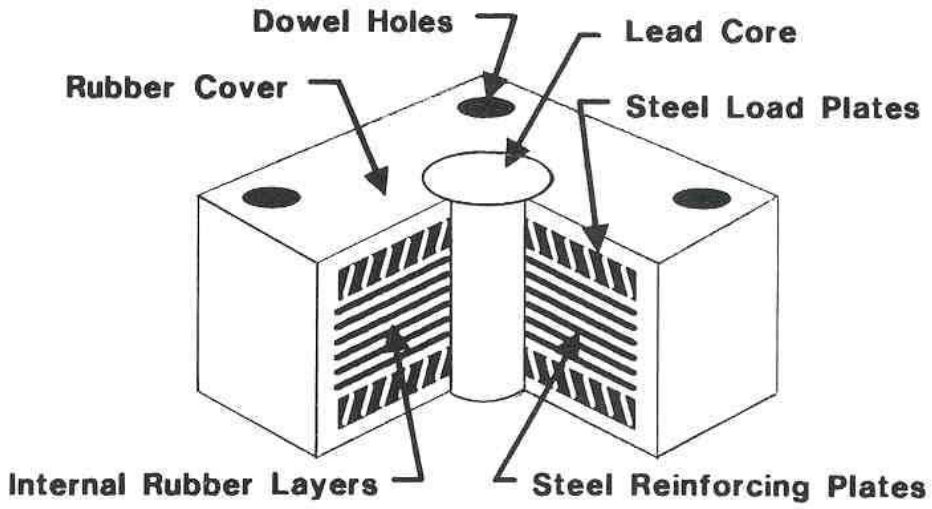


Figure 9.1.16 Lead Core Isolation Bearing



Figure 9.1.17 Lead Core Isolation Bearing

Friction Pendulum Bearings

Another bearing type designed to protect against earthquake damage is friction pendulum bearings. The bearings are designed to reduce lateral loads and shaking movements transmitted to the structure. They can protect structures and their contents during strong, high magnitude earthquakes and can accommodate 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. The period of the bearing is selected by choosing the radius of curvature of the concave surface. It is independent of the mass of the supported structure. Torsion motions of the structure are minimized because the center of stiffness of the bearings automatically coincides with the center of mass of the supported structure.

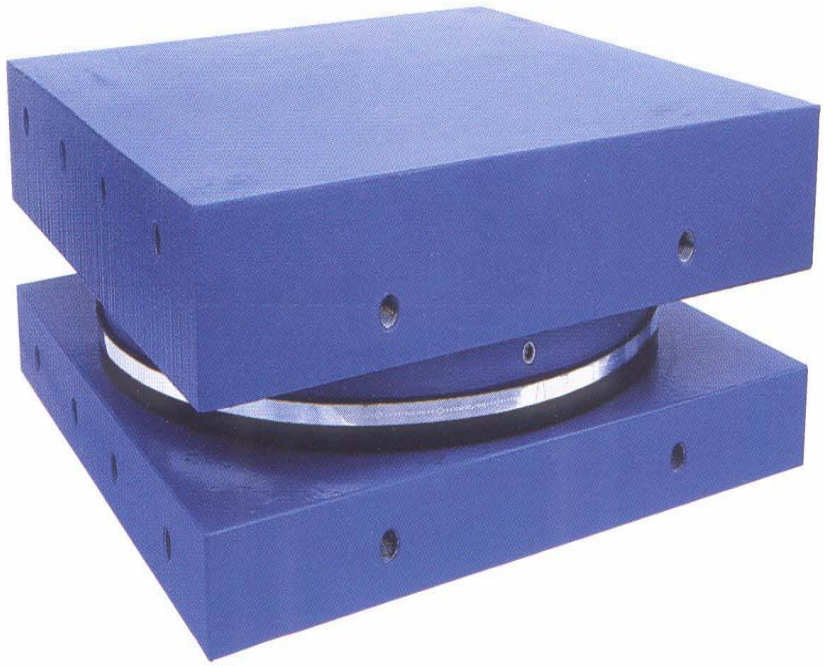


Figure 9.1.18 Friction Pendulum Bearing

High Dampening Rubber Bearings

High dampening 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.

Pot Bearings

Pot bearings allow for the multi-dimensional rotations of a structure. There are two different pot bearing configurations: neoprene and spherical.

Neoprene Pot Bearings

A neoprene pot bearing has a plate of stainless steel that is attached to the sole plate. This plate slides on a disc of PTFE. The PTFE disc 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 substructure. This cylinder is referred to as the pot. Guide bars in the expansion pot bearing restrict movement. A fixed bearing version of this configuration does not possess the stainless steel plate or the PTFE disc.

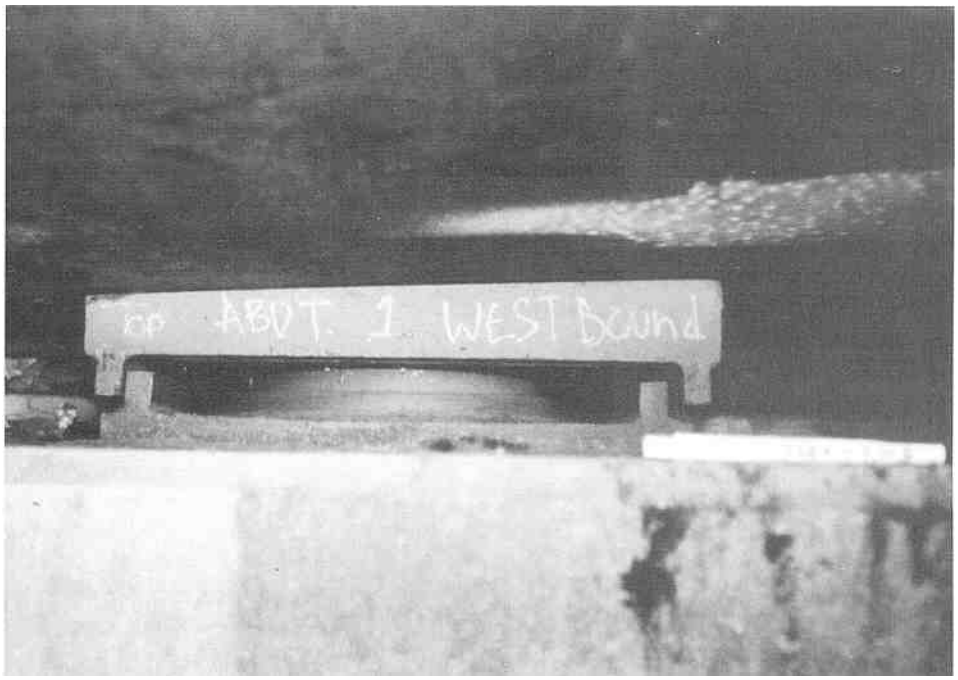


Figure 9.1.19 Pot Bearing

Spherical Pot Bearings

A spherical pot bearing has a plate of stainless steel that is attached to the sole plate. This plate slides on a disc of PTFE that is bonded to an aluminum alloy casting. The casting has a flat top and a spherical bottom. The bottom of the casting fits into another PTFE-coated aluminum alloy casting. The spherical shaped castings allow for the rotation of the structure. A fixed bearing version of this configuration has the upper aluminum casting attached to the sole plate. There is no stainless steel plate sliding on the PTFE disc.

Restraining 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. 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.

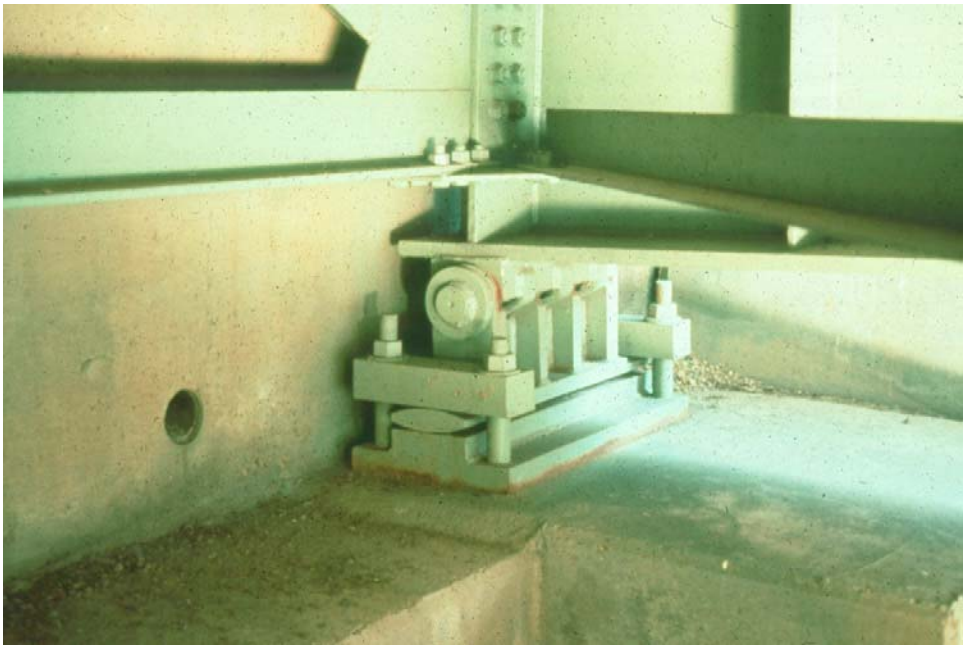


Figure 9.1.20 Restraining Bearing

9.1.4 Inspection Procedures

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

All bearings must have a suitable support. A distance of several inches should exist between the edge of the masonry plate and the edge of the supporting member, abutment, or pier. Note any loss of section to the supporting member near the bearing (e.g., spalling of a concrete bridge seat).

Although there are many different types of bearings, bearing inspection can be broken down into two major categories based upon the basic materials from which they are made:

- Metal bearings
- Elastomeric 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 the most susceptible to deterioration, while the others mentioned are either non-corrosive or corrosion-resistant. Consequently, the following discussions will concentrate on the most common materials: steel bearings and elastomeric bearings.

Inspection of Steel Bearings

Some inspection items are common to all steel bearings. For example, steel bearings are subject to the same corrosive forces as steel beams or girders. Therefore, check all bearing elements for any significant section loss.

Frozen bearings can occur when deterioration and debris buildup cause the bearing to bind up, thereby preventing free movement. Evidence of a frozen bearing includes bending, buckling, improper alignment of members, or cracks in the bearing seat.

Bearings should be properly aligned, and bearing surfaces should be 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 sole plate rests on only a portion of the masonry plate. The full load of the superstructure is therefore being applied to a smaller area on the substructure. This 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.

The bearing should not be loose. Looseness can be identified by noise at the bearing or by visually detectable movement in the bearing when the bridge is subjected to live loads. Loosening can be caused by any of the following:

- 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 or missing fasteners that are used to attach the bearing to either the superstructure or the support
- Worn bearing elements
- Uplift in curved bridge superstructures
- Pavement pressure which drives the backwall into the beams

Examine for broken or cracked welds and missing or sheared fasteners.

Bearings and lateral shear keys on skewed bridges should be 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).

Also, record the temperature during the inspection.

The following figures show examples of some common deficiencies.



Figure 9.1.21 Heavy Corrosion on Steel Rocker Bearing



Figure 9.1.22 Frozen Rocker Bearing



Figure 9.1.23 Spalling of Bridge Seat Due to High Edge Stress

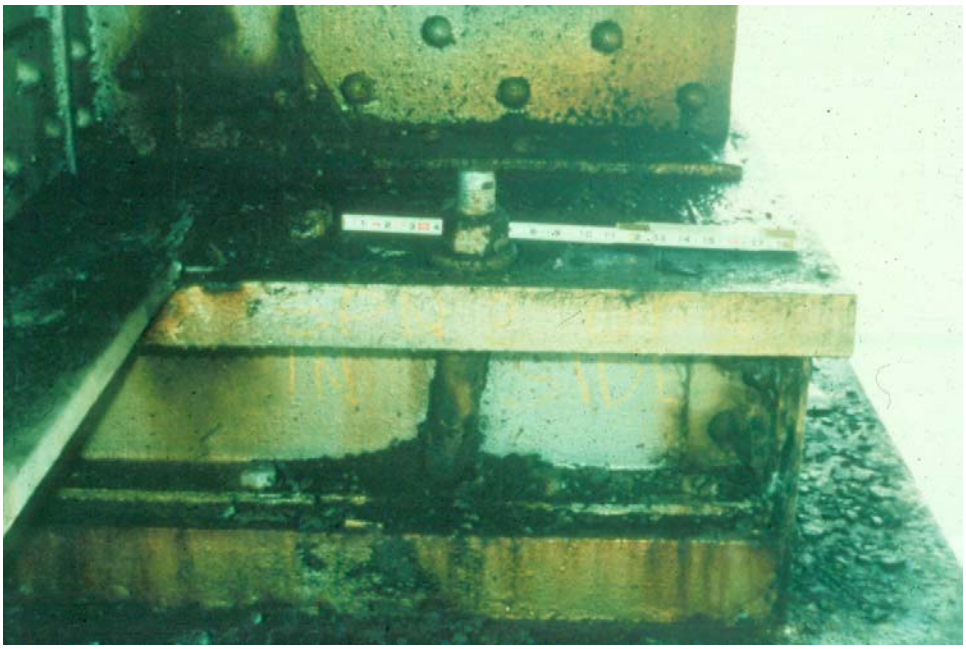


Figure 9.1.24 Bent Anchor Bolt

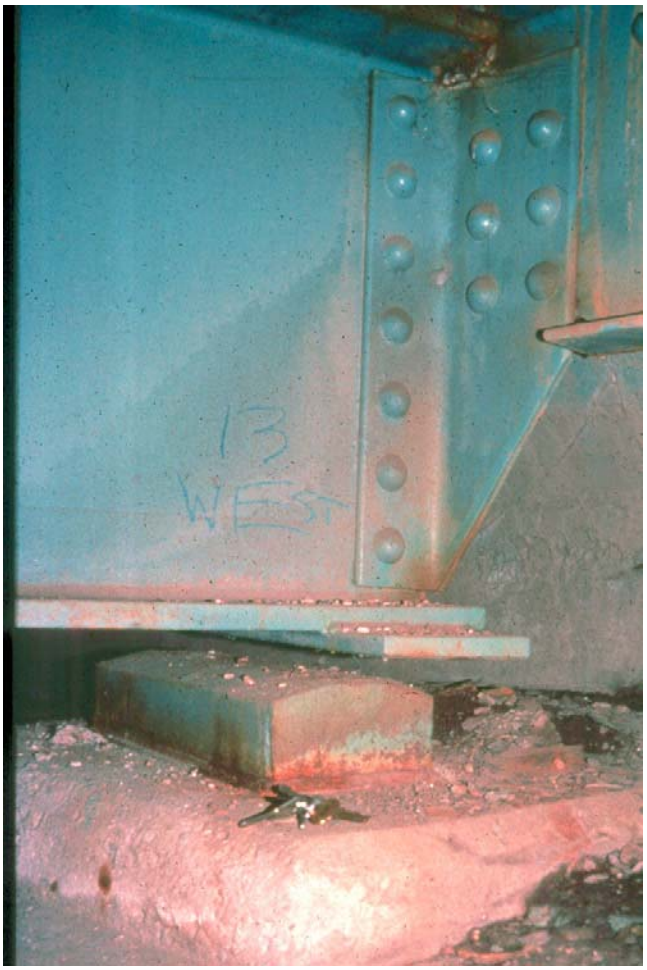


Figure 9.1.25 Uplift at Bridge Bearing

Specific inspection items for the various types of steel bearings are discussed in the following paragraphs.

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°F. 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 9.1.27). 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 should be measured to the nearest 3 m (1/8 inch), and the temperature at the time of inspection should be recorded.

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

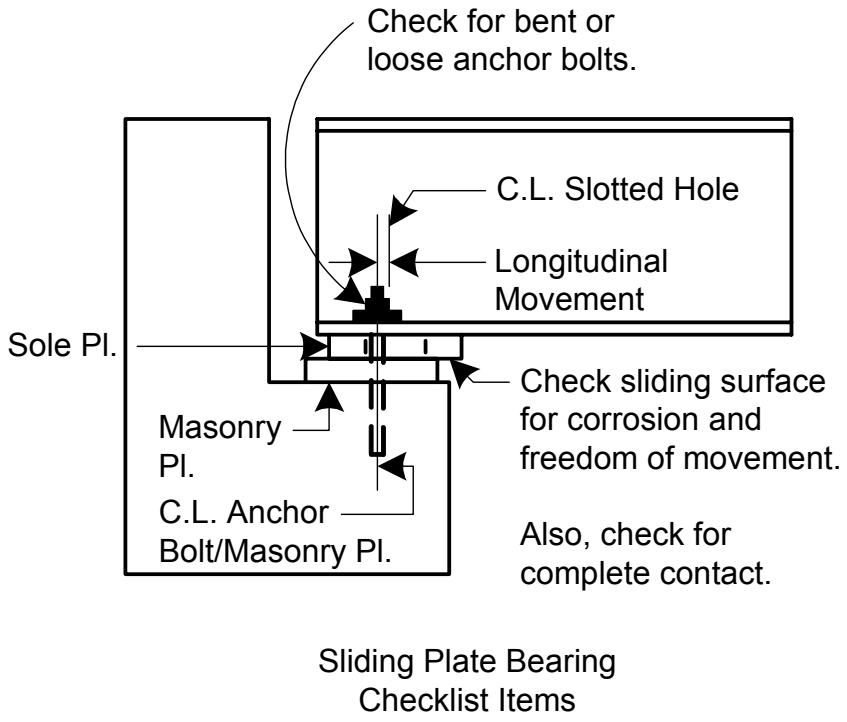


Figure 9.1.26 Sliding Plate Bearing Inspection Checklist Items



Figure 9.1.27 Longitudinal Misalignment in Bronze Sliding Plate Bearing

Roller Bearings

Roller bearings are similar to sliding plate bearings in that the roller unit should 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 (see Figure 9.1.28). 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 should be recorded.

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

The position of the roller should also 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.



Figure 9.1.28 Damaged Roller Nest Bearing

Rocker Bearings

Some rocker bearings have markings on the rocker and masonry plates. With no expansion or contraction, these marks should 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.

Measurements should be to the nearest 3 m (1/8 inch), and the inspection temperature should be recorded.

Rockers should be inspected for proper tilt. In warmer temperatures (above 68ºF), the rockers should be tilted towards the backwall in the expanded direction; in colder temperatures, the rockers should be tilted backward in the contracted position away from the backwall. Also check for exposure of the pintles if any are known to be present.

Pins and other contact surfaces should be examined for wear and freedom of movement.

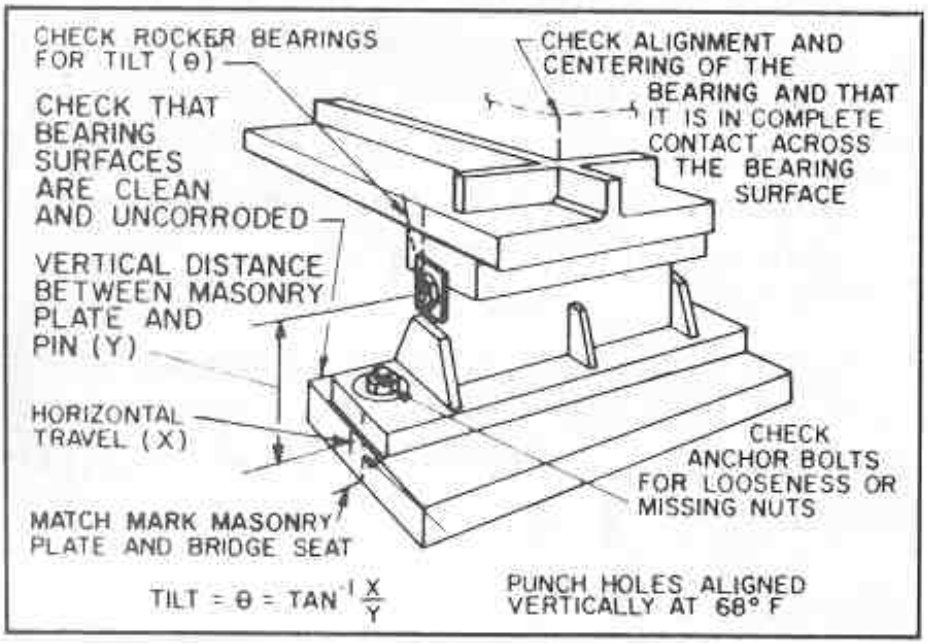


Figure 9.1.29 Rocker Bearing Inspection Checklist Items

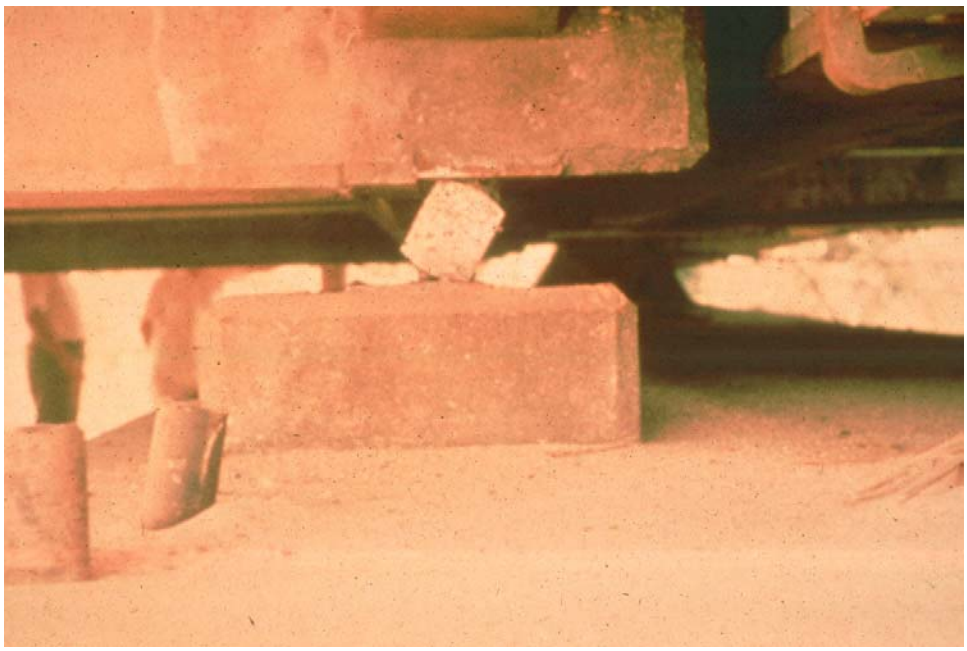


Figure 9.1.30 Excessive Tilt in a Segmental Rocker

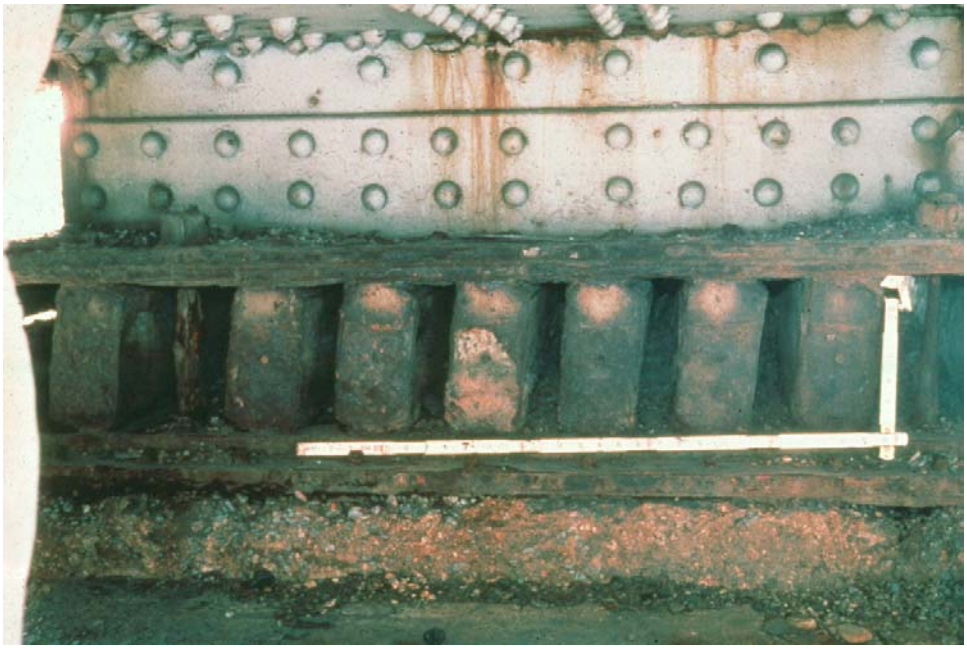


Figure 9.1.31 Frozen Rocker Nest

Pin and Link Bearings

Inspection of pin and link bearings is essentially the same as that described for pins and hangers in Topic 8.4. 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 13.3. Since disassembly is impractical during normal periodic bridge inspections, the inspector must closely examine all exposed portions of the pin and link for signs of corrosion, wear, stress, cracks, bending, and misalignment. If warranted, the inspector should 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.

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 should investigate lateral movement as well. The inspection temperature at which the measurements are taken should also be recorded.

Although not normally required, pot bearing rotation should also 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:

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

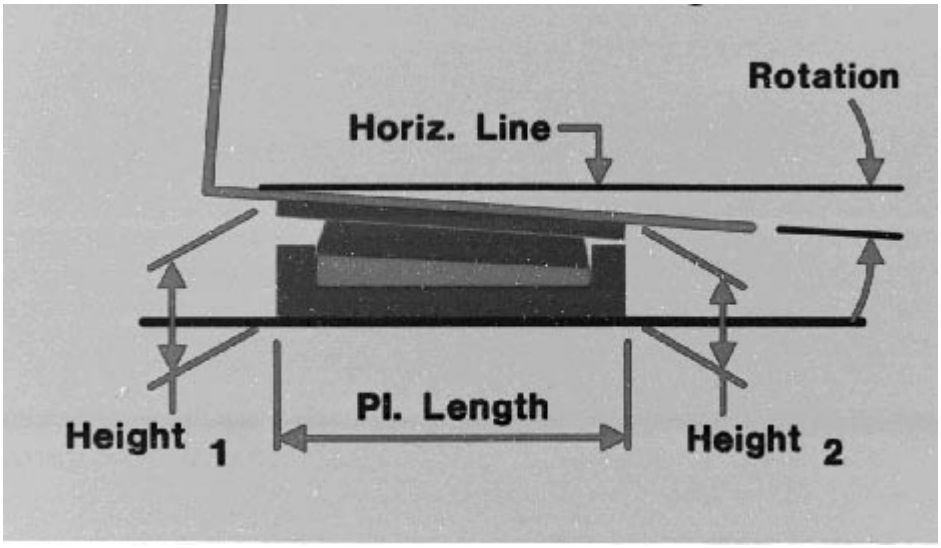


Figure 9.1.32 Angle of Rotation for Pot Bearing

Since the pot bearing allows multidirectional rotation, the inspector should 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, 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 would affect the smooth operation of the bearing.

Restraining Bearings

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/bolts) and pins is the main concern. Where these elements encompass a normal bridge bearing, the inspection of the bearing assembly itself follows the procedures normally used for that particular type of bearing.

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

testing to determine their condition.

Inspection of Elastomeric Bearings

Inspection of elastomeric bearings is somewhat simpler than the steel bearings previously discussed 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 will be discussed here.

Neoprene Bearings

Neoprene bearing pads should be inspected for excessive bulging (approximately greater than 15% of thickness). 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/thickness of the pad, then it should be noted. As expansion and contraction of the structure takes place, the bulge will tend to roll on the beam or bridge seat.

The bearing pad should be inspected for any splitting or tearing. Close attention should 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 should also be inspected for variable thickness other than that attributable to normal rotation of the bearing.

A plain (unlaminated) pad should 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 will eventually experience a shearing failure. Pad growth is not usually a problem with laminated bearings.

Close attention should 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 should 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 must be known in order to determine the bearing rotation.

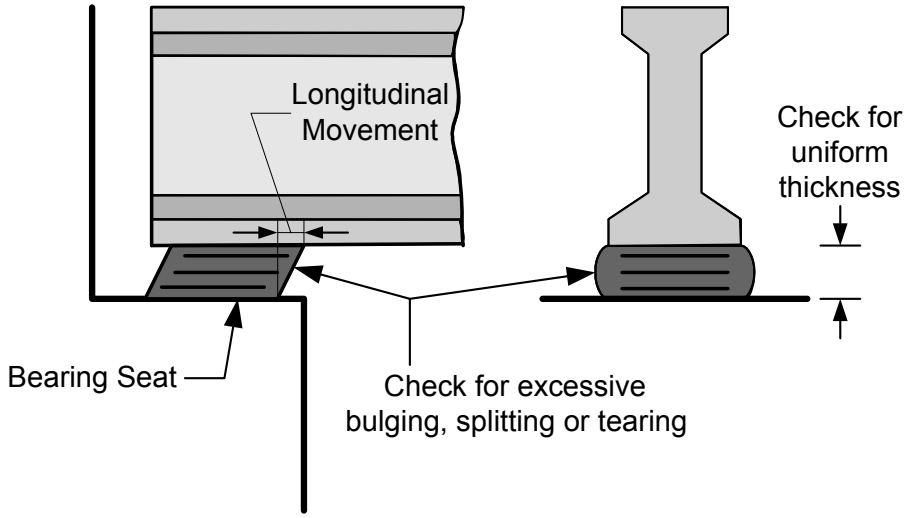


Figure 9.1.33 Elastomeric Bearing Inspection Checklist Items

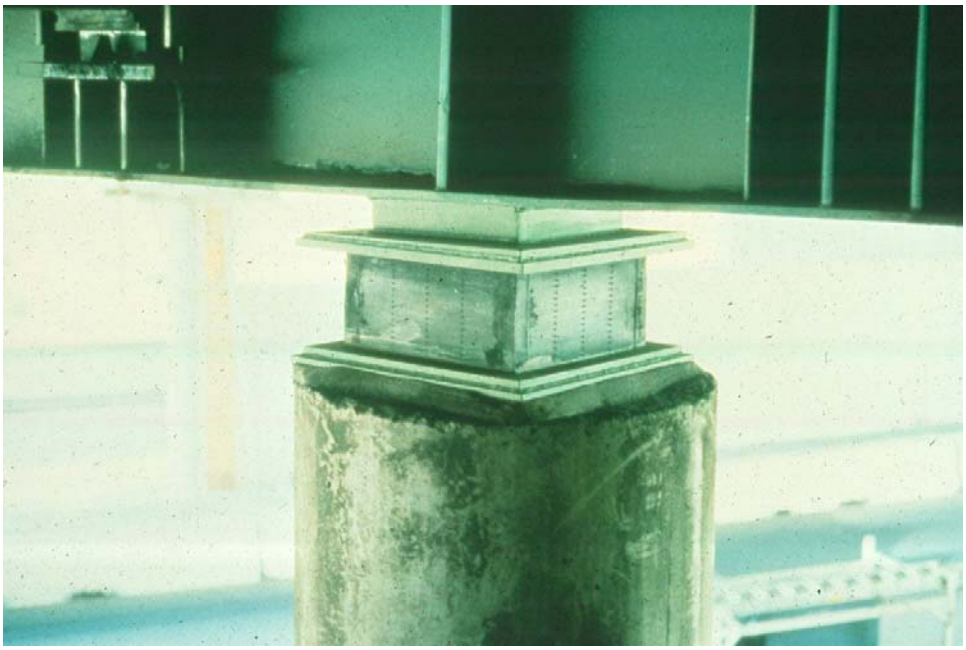


Figure 9.1.34 Neoprene Bearing Pad "Walking" Out from Under a Beam

Isolation Bearings

Isolation bearings are similar to neoprene bearing pads. They are composed of alternating layers of rubber and thin steel plates bonded together to form a unit. A lead core is tightly fitted into a preformed hole to provide rigidity (under low lateral loads such as wind and braking forces) and energy dissipation (under seismic loads). These bearings also use steel dowels to transfer shear forces.

The inspection items for isolation bearings are essentially the same as those for plain or laminated neoprene bearings. The only elements unique to isolation bearings are the lead core and steel dowels, both of which are hidden from view and can not be inspected.

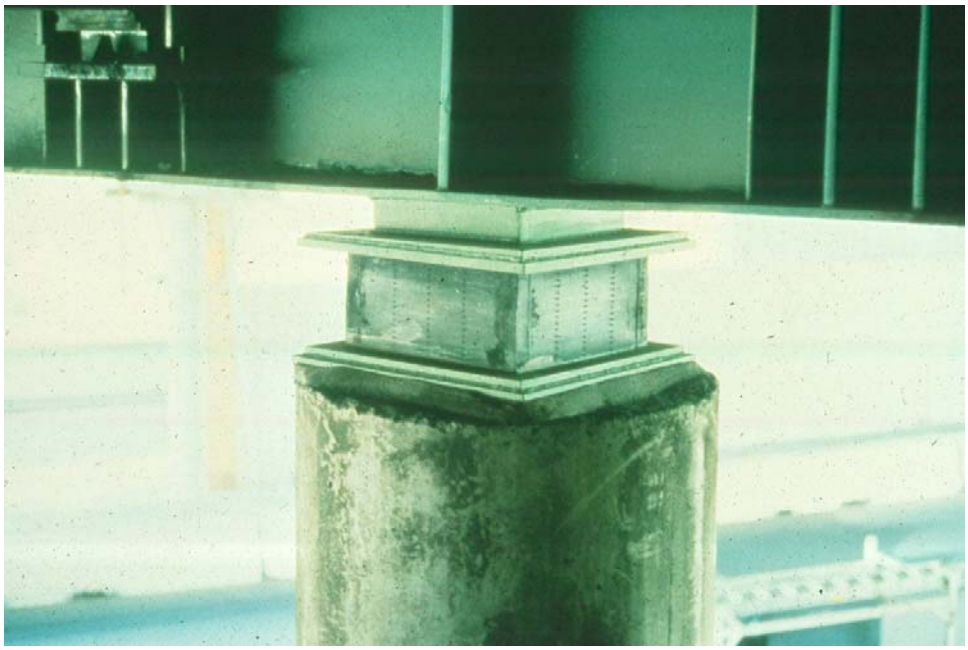


Figure 9.1.35 Lead Core Isolation Bearing

9.1.5 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 National Bridge Inspection Standards (NBIS) rating and the Pontis Bridge Management System (BMS).

Application of NBIS Rating Guidelines

Using NBIS rating guidelines, bearings fall under the superstructure on the Federal Structure Inventory and Appraisal (SI&A). The bearing type is noted, but no rating is given. More importantly, the bearing condition does not influence the superstructure component rating, unless it is an extreme condition.

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for each individual bearing. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value.

In an element level condition state assessment of a bridge bearing, the AASHTO CoRe element is one of the following, depending on the type:

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

The unit quantity for the bearing elements is "each", and each bearing element must be placed in one of the three available condition states based on the overall bearing condition. The inspector must note the condition of all the bearings individually and fit them into a given condition state description. Condition state 1 is the best possible rating for the bearings. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

No Smart Flags are currently available for bearings.

Evaluation of Bearings

General

Bearings are considered a part, or element, of the superstructure, and they must be inspected.

However, an important point to remember about the inspection of bearings is that they do not influence the superstructure condition rating, except in extreme situations.

Bearings are still an important inspection item. Small maintenance problems with bearings can grow progressively worse if ignored, eventually causing major problems for the bridge. Inoperable bearings can transfer significant stresses to the superstructure or substructure.

Serious Bearing Conditions

Serious situations could be created for a bridge superstructure if one of the following occurs:

- Horizontal failure of several bearings that could allow the superstructure to pull off the substructure
- Failure of a tiedown (restraining) device which could allow a span to collapse

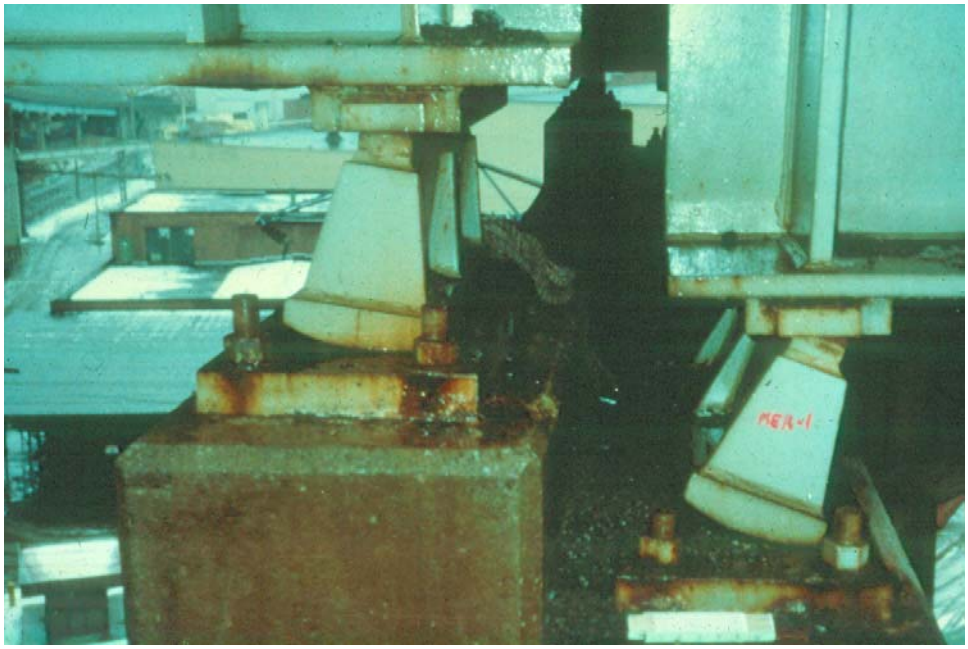


Figure 9.1.36 Critical Bearing Condition



Figure 9.1.37 Broken Pintle on a Tied Arch Bearing

If such a problem existed with the bearings, then the bearings would have a significant impact on the superstructure condition rating. Otherwise, the bearings affect the rating very little.

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Section 10
Inspection and Evaluation of Substructures

Topic 10.1 Abutments and Wingwalls

10.1.1 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.

This section describes the characteristics of two major types of substructure units:

- Abutments
- Pier (Discussed in Topic 10.2 Piers)

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

This section describes the design characteristics and inspection locations and procedures for abutments and wingwalls, including the most common structural problems and their causes.

10.1.2 Design Characteristics of Abutments

Abutment Types

Abutments are classified according to their locations with respect to the approach

embankment. The most common abutment types are presented in Table 10.1.1 and in Figures 10.1.1 to 10.1.18.

Full height or closed type

- Gravity
- Counterfort
- Cantilever
- Curtain wall/Pedestal
- Timber bent
- Crib

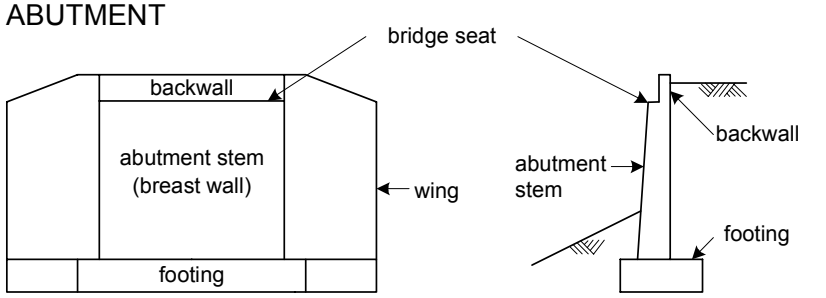
Stub, semi-stub, or shelf type

Open or spill-through type

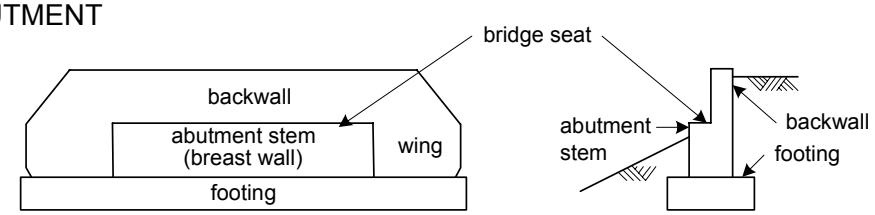
Integral type

Table 10.1.1 Common Abutment Types

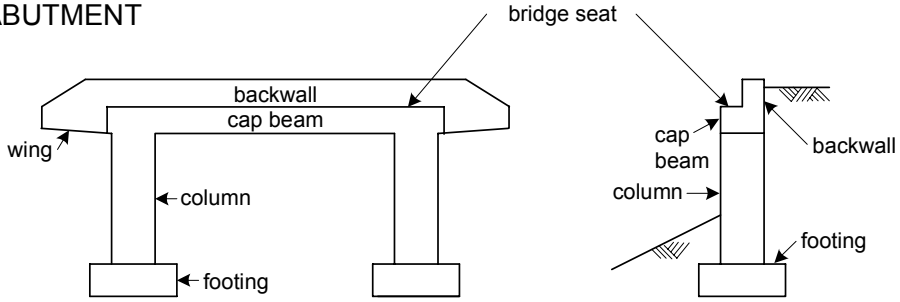
1. FULL HEIGHT ABUTMENT



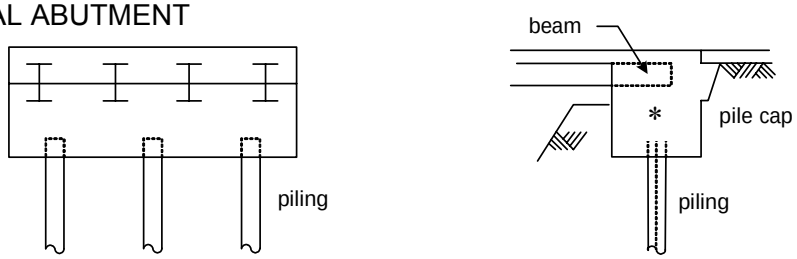
2. STUB ABUTMENT



3. OPEN ABUTMENT



4. INTEGRAL ABUTMENT



* Some states weld beam and piles prior to concrete placement

Figure 10.1.1 Schematic of Common Abutment Types



Figure 10.1.2 Full Height Abutment



Figure 10.1.3 Stub Abutment

Full Height Abutments and Stub Abutments

Full height abutments are used when shorter spans are desired. This reduces the initial superstructure costs. Stub abutments are used when it is desirable to keep the abutments away from the underlying roadway or waterway. Longer spans are required when stub abutments are used. Using stub abutments reduces the cost of the substructure and increases the cost of the superstructure.



Figure 10.1.4 Open 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 columns. Only the topmost few feet of the embankment are actually retained by the abutment cap.

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

The disadvantages are a tendency for the fill to settle around the columns since good compaction is difficult to achieve in the confined spaces, and for excessive erosion to occur in the fore slope. Rock fill is sometimes used to counter these problems.

This type of construction is not suitable adjacent to streams due to susceptibility to scour.

Integral Abutments

Most bridges have superstructures that are independent of the substructure to accommodate bridge length changes due to thermal effects. Expansion devices like deck joints and expansion bearings 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 10.1.5). Relative movement of the abutment with respect to the backfill allows the structure to adjust to thermal expansions and contractions. In this type of design, 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 bearings and joints to repair or replace, which more or less eliminates the need for maintenance. 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.

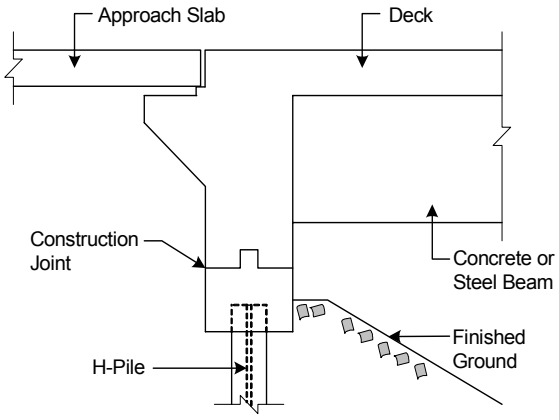




Figure 10.1.5 Integral Abutment



Figure 10.1.6 Integral Abutment

Mechanically Stabilized Earth Abutments

Another type of abutment in use is the Mechanically Stabilized Earth (MSE) Support Abutment (see Figure 10.1.7). "Reinforced Earth" and "Retained Earth" are trademarked names given to these systems by suppliers of the components. The components of such an abutment typically consist of precast concrete panels, metallic soil reinforcing strips (flat strips or welded bar grids), and backfill. 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 embankment.

To construct the wall, the 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." Soil reinforcement is are then placed and bolted into the panels and covered with more backfill (see Figure 10.1.8). This process, which allows the wall to remain stable during construction, is repeated until the designed height is attained. The inextensible metal reinforcement is placed in the soil to improve tensile resistance, which strengthens the soil and allows for construction on steep slopes.

Advantages of this structure are its internal stability and its ability to counteract shear forces, especially during earthquakes. It is generally lower in cost and has better esthetics than a comparable reinforced concrete full height abutment. Disadvantages include repair difficulties if soil reinforcement fails and limited site applications.



Figure 10.1.7 Mechanically Stabilized Earth Abutment (Note Precast Concrete Panels)



Figure 10.1.8 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, developed by the Federal Highway Administration. GRS abutments are basically constructed on a level surface starting with a base structure of common, but good 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, their durability, and their aesthetic appearance. GRS technology works well with simple overpasses; however, they are not ideal where severe flooding could occur (see Figures 10.1.9 and 10.1.10).



Figure 10.1.9 GRS Bridge Abutment Developed at the 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. Report No. FHWA-SA-96-071 (Demo 82 Manual) treats these systems in depth.



Figure 10.1.10 View of the Founders/Meadows Bridge Supported by GRS Abutments

Primary Materials

The primary materials used in abutment construction are plain cement concrete, reinforced concrete, stone masonry, steel (although not very common), timber, or a combination of these materials.



Figure 10.1.11 Plain Unreinforced Concrete Gravity Abutment



Figure 10.1.12 Reinforced Concrete Cantilever Abutment



Figure 10.1.13 Stone Masonry Gravity Abutment



Figure 10.1.14 Combination: Timber Pile Bent Abutment with Reinforced Concrete Cap



Figure 10.1.15 Combination: Stone Masonry Gravity Abutment with Reinforced Concrete Bearing Seat

Reinforcing Steel

The pattern of primary steel reinforcement used in concrete abutments depends on the abutment type (see Figure 10.1.16). In a concrete 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, horizontal bars in the bottom face of the cap beam, vertical bars in the columns and horizontal bars in the bottom of the footing. The shear steel reinforcing would consist of the shear stirrups in the cap beam and the hoops or spiral reinforcing steel in the columns. All other bars would be temperature and shrinkage reinforcement, which is secondary reinforcement.

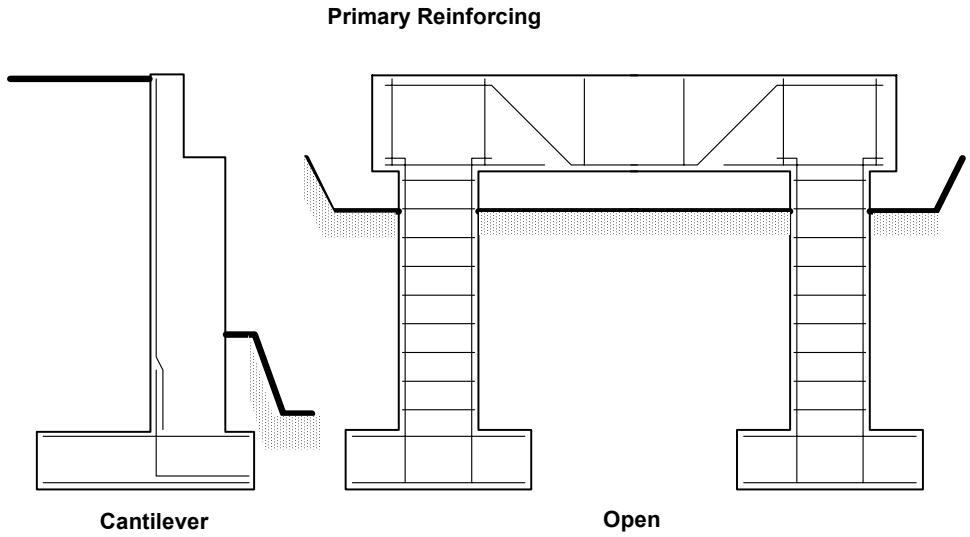


Figure 10.1.16 Primary Reinforcing Steel in Abutments

Abutment Elements

Common abutment elements include:

- Bridge seat
- Backwall
- Pile cap
- Cheek wall
- Abutment stem (breast wall)
- Tie backs
- Soil reinforcing strips
- Precast panels
- Footings
- Piles
- Geotextiles
- Wing walls

The basic abutment elements are shown in Figure 10.1.1 and described below.

The **bridge seat** provides a bearing area that supports the bridge superstructure.

The **backwall** retains the approach roadway subbase 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 10.1.17). A cheek wall is not always present.

The **abutment stem** or breast wall supports the bridge seat and retains the soil

behind the abutment.

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.

The **footing** transmits the weight of the abutment, the soil backfill loads, and the bridge reactions to the supporting soil or rock when piles are not used. 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.

Piles, if present, carry structural loads through the soil to rock (bearing piles) or dissipate the loads into the surrounding soil (friction piles). The upper ends of the piles may be embedded in the abutment stem, or, more likely, in a concrete pile cap which is similar to the footing described above (see Figure 10.1.18).



Figure 10.1.17 Cheek Wall

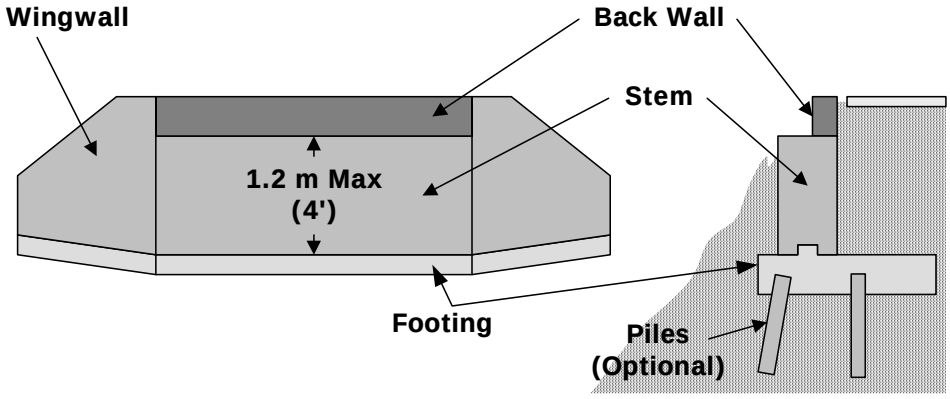


Figure 10.1.18 Stub Abutment on Piles

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**
- **Pile foundations**

A spread footing is used when the bedrock layers are 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" 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 will be placed below the recognized maximum frost line depth for that area.

A pile foundation is used when the soil is not suited for supporting the bridge or when the bedrock is over 3 meters (10 feet) or so from the ground surface. 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. 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. The terms "caisson," "drilled shafts," and "bored pile" are frequently used by engineers to denote drilled pile construction.



Figure 10.1.19 Stub Abutment on Piles with Piles Exposed

10.1.3 Inspection Procedures and Locations for Abutments

Inspection procedures for abutments are the same as discussed for superstructures, particularly when it involves material deterioration. However, because stability is a paramount concern, checking for various forms of movement is required.

The locations for inspection are not particularly specific, but can be related to common abutment problems.

The most common problems observed during the inspection of abutments are:

- Vertical movement
- Lateral movement
- Rotational movement
- Material defects
- Scour of the foundation
- Drainage system malfunction

Vertical Movement

Vertical movement can occur in the form of uniform settlement or differential settlement. A uniform settlement of all bridge substructure units, including abutments, will have little effect on the structure. Uniform settlements of 0.3 m (1 foot) have been detected on small bridges with no signs of distress.

However, differential settlement can produce serious 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 10.1.20). It may also occur under a single substructure unit (see Figure 10.1.21). 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, and deterioration of the abutment foundation material.

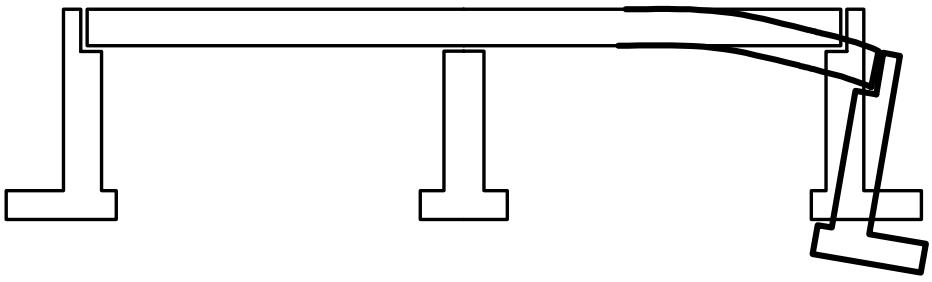


Figure 10.1.20 Differential Settlement Between Different Substructure Units

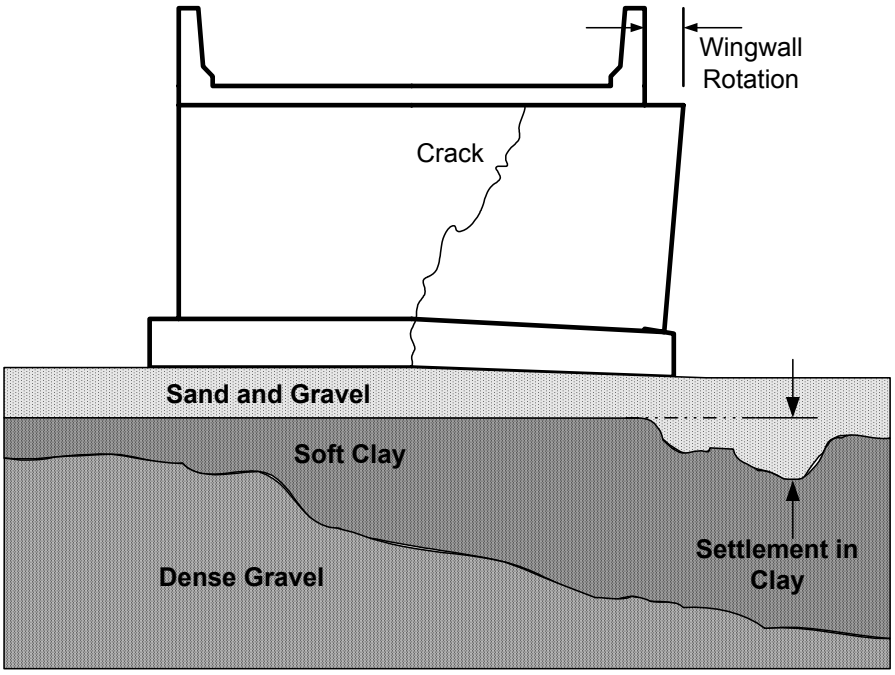


Figure 10.1.21 Differential Settlement Under an Abutment

Inspection for vertical movement, or settlement, should include:

- Inspect the joint opening between the end of the approach slab and the 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 10.1.22).
- Examine the superstructure alignment for evidence of settlement (particularly the bridge railing).
- Check for scour around the abutment footing or foundation.
- Inspect the joint that separates the wingwall and abutment for proper alignment.



Figure 10.1.22 Cracks in Abutment due to Settlement

Lateral Movement

Earth retaining structures, such as abutments and retaining walls, are susceptible to lateral movements, or sliding (see Figure 10.1.23). Lateral movement occurs when the horizontal earth pressure acting on the wall exceeds the friction forces that hold the structure in place.

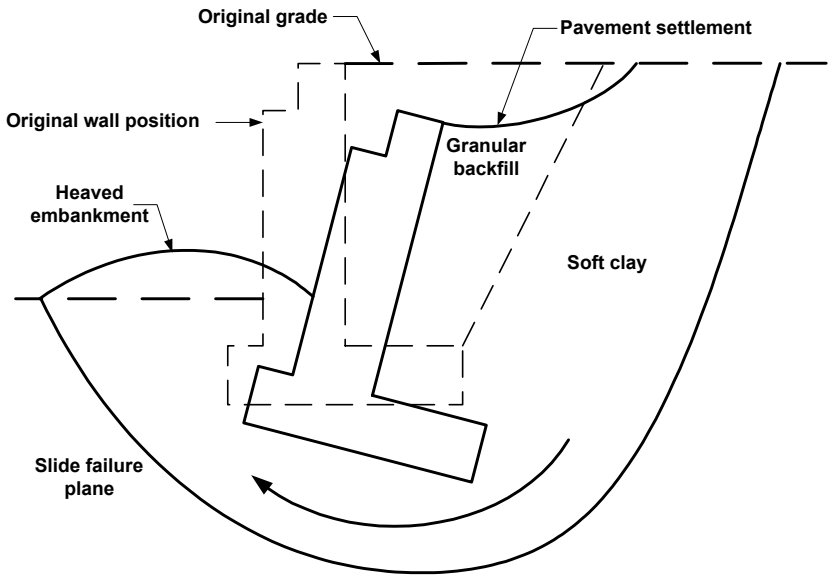


Figure 10.1.23 Lateral Movement of an Abutment due to Slope Failure

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, should include:

- Inspect the general alignment of the abutment.
- Check the bearings for evidence of lateral displacement (see Figure 10.1.24).
- 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 10.1.25).
- Settled approach pavement
- 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 or scour of the embankment material in front of the abutment (see Figure 10.1.26).



Figure 10.1.24 Excessive Rocker Bearing Displacement Indicating Possible Lateral Displacement



Figure 10.1.25 Depressed Approach Slab due to Rotating Abutment



Figure 10.1.26 Erosion at Abutment Exposing Footing

Rotational Movement

Rotational movement, or tipping, of substructure units is generally the result of unsymmetrical settlements or lateral movements due to horizontal earth pressure (see Figure 10.1.27). Abutments and walls are subject to this type of movement.

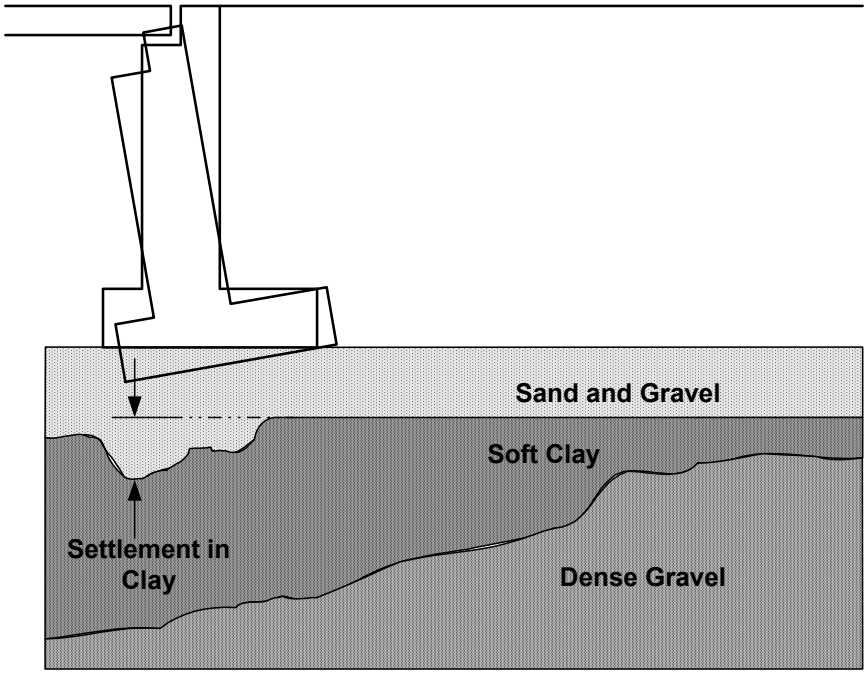


Figure 10.1.27 Rotational Movement of an Abutment

The most common causes of rotational movement are 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, should include:

- Check the vertical alignment of the abutment using a plumb bob; keep in mind that some abutments are constructed with a battered or sloped front face (see Figures 10.1.28 and 10.1.29).
- Examine the clearance between the beams and the backwall.
- Inspect for clogged drains or weep holes.
- Investigate for cracks, and record the width, length, and direction.



Figure 10.1.28 Rotational Movement at Abutment

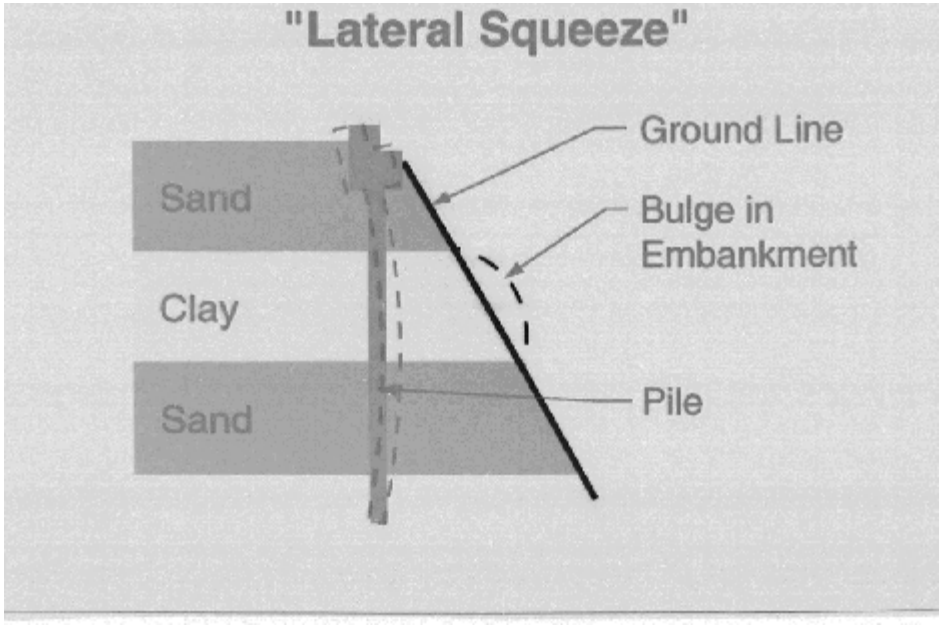


Figure 10.1.29 Rotational Movement due to "Lateral Squeeze" of Embankment Material

Material Defects

Another common problem observed during the inspection of abutments is the presence of construction material defects. Table 10.1.2 presents the most common defects for various materials used in substructure units.

Refer to Section 2 for a more detailed description of the types and causes of deterioration observed in these construction materials.

Concrete:	Steel:
➢ Cracking	➢ Corrosion
➢ Delamination	➢ Fracture
➢ Spalling	➢ Buckling
➢ Scaling	
➢ Crushing	
➢ Exposed reinforcement	
Stone masonry:	Timber:
➢ Weathering	➢ Decay
➢ Spalling	➢ Insects
➢ Cracking	➢ Marine borers
➢ Splitting	➢ Caddisflies
➢ Mortar cracking and deterioration	➢ Weathering
	➢ Fire damage

Table 10.1.2 Types of Material Defects in Substructure Units

Concrete and Stone Masonry

Inspection for concrete and stone masonry material defects in abutments should include:

- Examine the bearing seats for cracking and spalling, particularly near the edges; this is particularly critical where concrete beams bear directly on the abutment seat (see Figure 10.1.30).
- Inspect for the presence of debris and standing water on the bearing seats.
- Investigate for deteriorated concrete in areas that are exposed to roadway drainage, particularly below the joint between the backwall and the deck (see Figure 10.1.31).
- Check the backwall for cracking and possible movement.
- Examine the construction joint between the backwall and the abutment stem.
- Inspect stone masonry for mortar cracks or loss of mortar in the joints (see Figure 10.1.32).
- Examine stone masonry for vegetation, water seepage through cracks, loose or missing stones, weathering, and spalled or loose blocks.

Several advanced techniques are available for concrete inspection. Nondestructive and other methods are described in Topics 13.2.2 and 13.2.3.



Figure 10.1.30 Cracking in Bearing Seat of Concrete and Stone Abutment



Figure 10.1.31 Deteriorated Concrete in Abutment Backwall

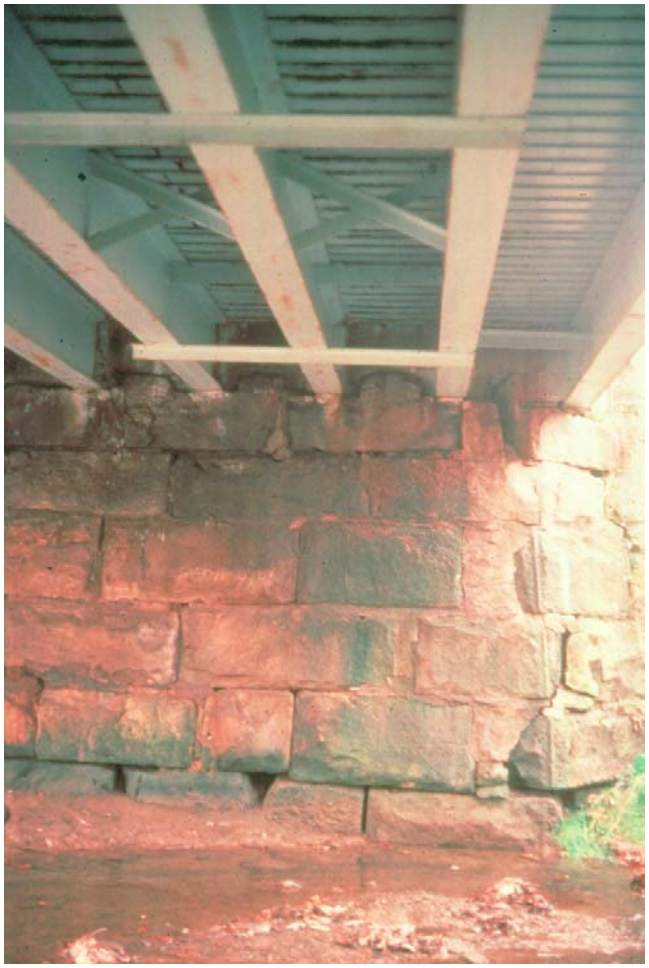


Figure 10.1.32 Deteriorated Stone Masonry Abutment

Steel

Although a steel abutment is uncommon (see Figure 10.1.33), the following items should be inspected if one is encountered:

- Examine bearing seat area for buildup of dirt and debris.
- After cleaning, check bearing seat area for corrosion and section loss.
- Inspect cap beam, piles, and any other steel elements for corrosion, cracking, and section loss.
- Investigate piles closely at the ground line.
- Check for scour and erosion around the piles.
- Examine all fasteners and connections for condition and tightness.

Several advanced techniques are available for steel inspection. Nondestructive and other methods are described in Topics 13.3.2 and 13.3.3.



Figure 10.1.33 Steel Abutment

Timber

Inspection for timber defects in abutments should include:

- Examine bearing seat for accumulated dirt and debris and prolonged exposure to moisture.
- Inspect for decay, insect damage, and crushing of the cap beam.
- Investigate for local failures in lagging or piles due to lateral movement (see Figure 10.1.34).
- Check crib timbers, timber lagging, and piles for splits, cracks, decay, insect damage, fire damage, and chemical attack (see Figure 10.1.35).
- Inspect for scour around the piles (see Figure 10.1.36).
- Examine piles very closely for decay at or near the ground line or waterline.
- Investigate splices and connections for tightness and for loose bolts.
- In marine environments, examine piles for the presence of marine borers and caddis flies.

Several advanced techniques are available for timber inspection. Nondestructive and other methods are described in Topics 13.1.2 and 13.1.3.



Figure 10.1.34 Local Failure in Timber Pile due to Lateral Movement of Abutment



Figure 10.1.35 Decay in Lagging of Timber Crib Abutment



Figure 10.1.36 Decayed Lagging and Scour at a Timber Pile Bent Abutment

Scour

Scour is the removal of material from a streambed as a result of the erosive action of running water (see Figure 10.1.37). Scour can cause undermining of abutments when streams or rivers flow adjacent to them. Refer to Topic 11.2 for a more detailed description of the various types of scour.

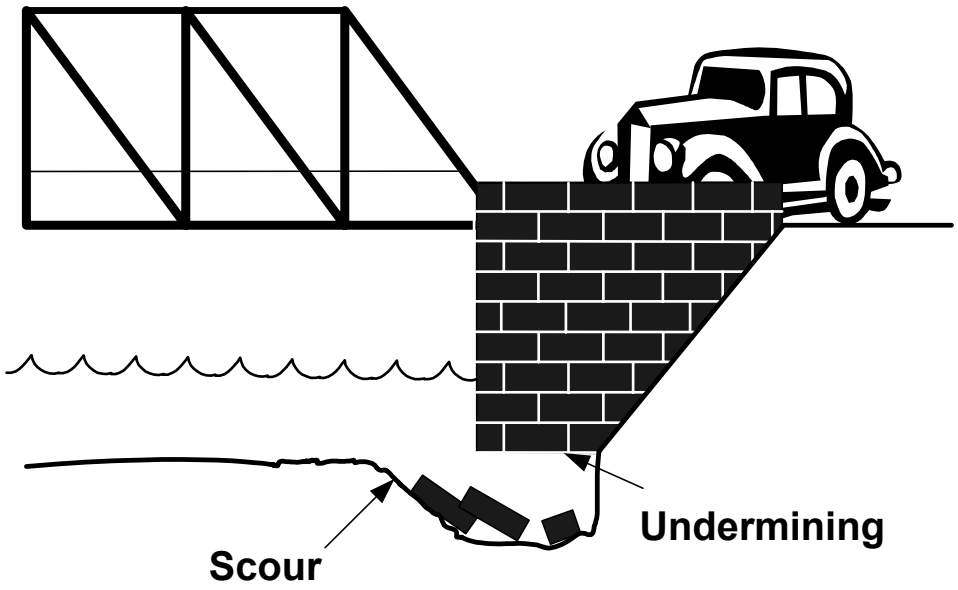


Figure 10.1.37 Abutment with Undermining due to Scour

Inspection for scour should include probing around the abutment footing for signs of undermining (see Figure 10.1.38). Sometimes silt loosely fills in a scour hole and offers no protection or bearing capacity for the abutment footing.



Figure 10.1.38 Wading Inspection

Drainage Systems

Water can build up horizontal pressure behind an abutment. Allowing the water to exit from behind the abutment relieves this pressure. Weep holes, normally 100 mm (4 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 improperly. Check the condition of any drainage system that is placed adjacent to the abutment that may result in deterioration of the abutment.

10.1.4 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.

A wingwall is similar to an abutment except that it is not required to carry any vertical loads. The absence of the vertical superstructure load usually necessitates a wider footing to resist the overturning moment (see Figure 10.1.39).



Figure 10.1.39 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 10.1.40)
- **Flared** - form an acute angle with the bridge roadway (see Figure 10.1.41)
- **U-wings** - parallel to the bridge roadway (see Figure 10.1.42)



Figure 10.1.40 Straight Wingwall



Figure 10.1.41 Flared Wingwall

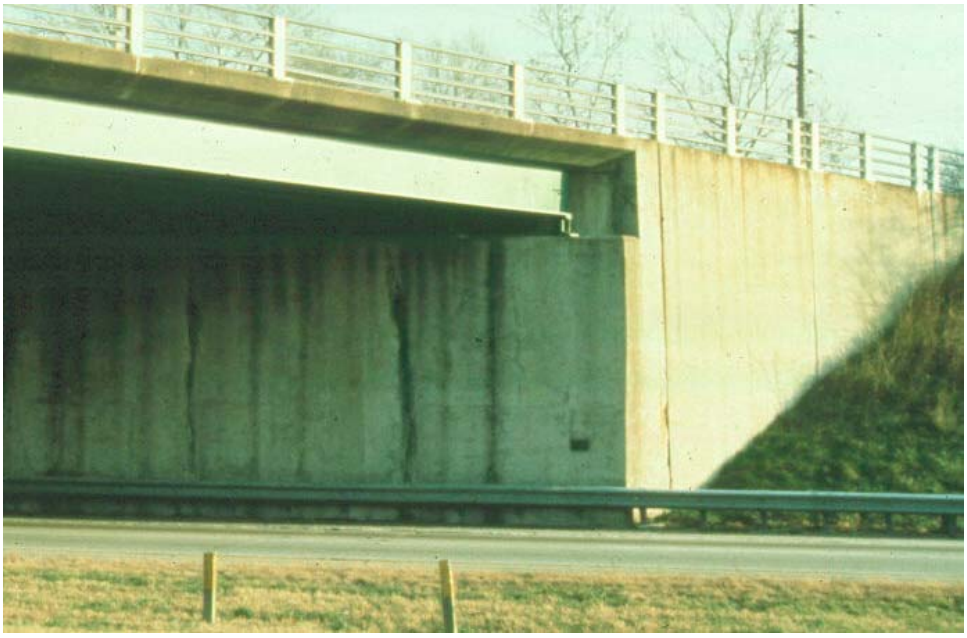


Figure 10.1.42 U-wingwall

Construction Classifications

There are also several construction classifications of wingwalls:

- **Integral** - constructed monolithically with the abutment (see Figures 10.1.43) normally cast-in-place concrete
- **Independent** - constructed separately from the abutment; usually an expansion or mortar joint separates them from the abutment breast wall (see Figure 10.1.44)

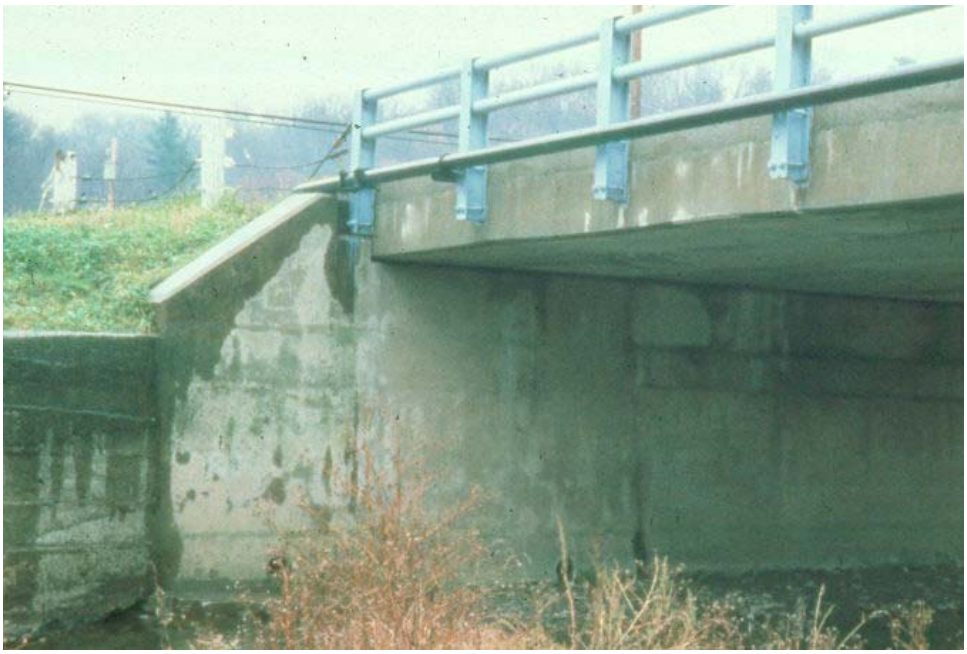


Figure 10.1.43 Integral Wingwall



Figure 10.1.44 Independent MSE Wingwall

Primary Materials

Wingwalls may be constructed of concrete, stone masonry, steel, or timber. 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). All other bars are temperature and shrinkage reinforcement (see Figure 10.1.46).



Figure 10.1.45 Masonry Wingwall

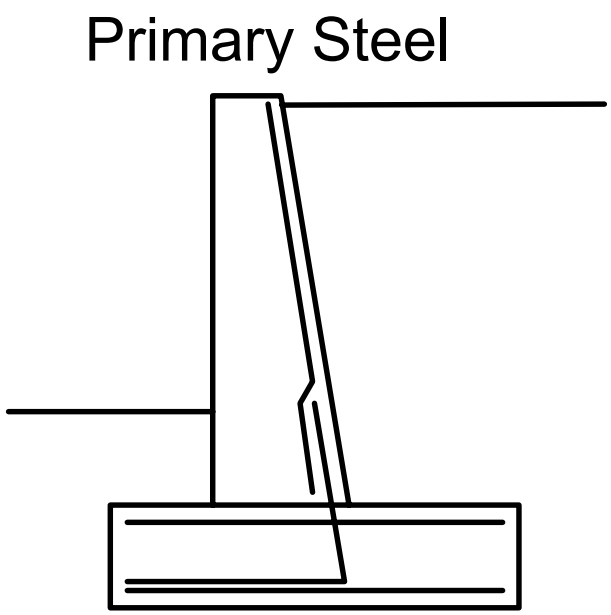


Figure 10.1.46 Primary Reinforcing Steel in Concrete Cantilever Wingwall

10.1.5 Inspection Locations and Procedures for Wingwalls

The inspection locations and procedures for most wingwalls are similar to those for an abutment (see Topic 10.1.3). Many of the problems that occur in abutments are common in wingwalls also, including:

- Vertical movement
- Lateral movement
- Rotational movement (see Figure 10.1.47)
- Material defects (see Figure 10.1.48)
- Scour (see Figure 10.1.49)
- Drainage systems



Figure 10.1.47 Rotational Movement at Concrete Wingwall



Figure 10.1.48 Deteriorated Concrete Wingwall



Figure 10.1.49 Scour Under Concrete Wingwall

Material defects to look for include shoulder erosion, cracks, concrete deterioration, stone masonry deterioration, and timber decay (see Figures 10.1.50 to 10.1.53).



Figure 10.1.50 Roadway Shoulder Erosion Behind Wingwall



Figure 10.1.51 Settlement Cracks at Integral Concrete Wingwalls



Figure 10.1.52 Deteriorating Stone Masonry Wingwall



Figure 10.1.53 Timber Wingwall

Refer to Topic 10.1.3 for a more detailed description of these common wingwall problems.

Integral wingwalls should be inspected with the abutments, and they are included in the substructure evaluation and rating. However, only that portion up to the first construction or expansion joint is considered. Independent wingwalls should also be inspected, but their condition does not affect the evaluation and condition rating of the substructure.

10.1.6 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 National Bridge Inspection Standards (NBIS) rating and the Pontis Bridge Management System (BMS).

Application of NBIS Rating Guidelines

Using NBIS rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the substructure. Rating codes range from 9 to 0, where 9 is the best rating possible (see Topic 4.2).

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the abutment and wingwall. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element Level Smart Flags are also used to describe the condition of the substructure, while identifying local problems that are not reflected in the CoRe elements.

In an element level condition state assessment of an abutment or wingwall structure, the AASHTO CoRe element typically is one of the following:

Element No.	Description
215	Abutment – Reinforced Concrete
216	Abutment – Timber
217	Abutment – Other (masonry, steel, etc.)

The unit quantity for the substructure elements is in meters (or feet), measured across the abutment and the entire element must be placed in one of the four available condition states based solely on the substructure condition. Condition State 1 is the best possible rating for the abutment or wingwall. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For settlement of the abutment or wingwall, the "Settlement" Smart Flag, Element No. 360, can be used and one of three condition states assigned. For scour at the abutments or wingwalls, the "Scour" Smart Flag, Element No. 361, can be used and one of three condition states assigned.

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Topic 10.2 Piers and Bents

10.2.1 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 10.2.1). The difference between a pier and a bent is simply in physical appearance. There is no functional difference between the two. A pier generally has only one column or shaft supported by a large footing. Bents have two or more columns or pile extensions with a cap or cross bracing.

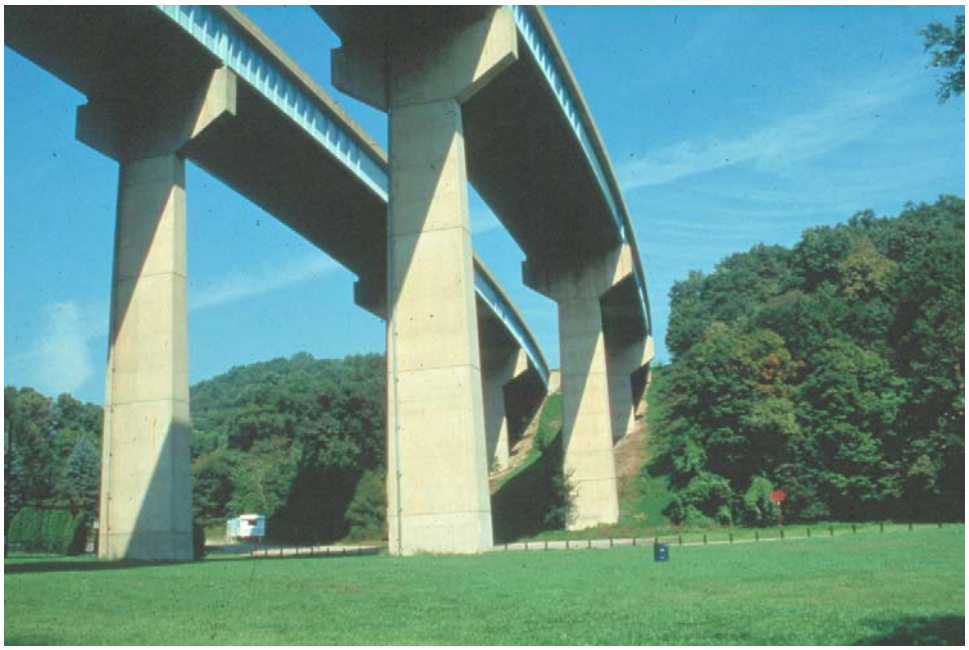


Figure 10.2.1 Example of Piers as Intermediate Supports for a Bridge

10.2.2 Design Characteristics

Pier and Bent Types

The most common pier and bent types are:

- **Solid shaft pier** (see Figure 10.2.2)
- **Column pier** (see Figure 10.2.3)
- **Column pier with web wall** (see Figures 10.2.4 and 10.2.5)
- **Cantilever pier or hammerhead pier** (see Figures 10.2.6 and 10.2.7)
- **Column bent or open bent** (see Figure 10.2.8)
- **Pile bent** (see Figure 10.2.9)

While there are many different types of piers and bents, they all function in essentially the same manner.

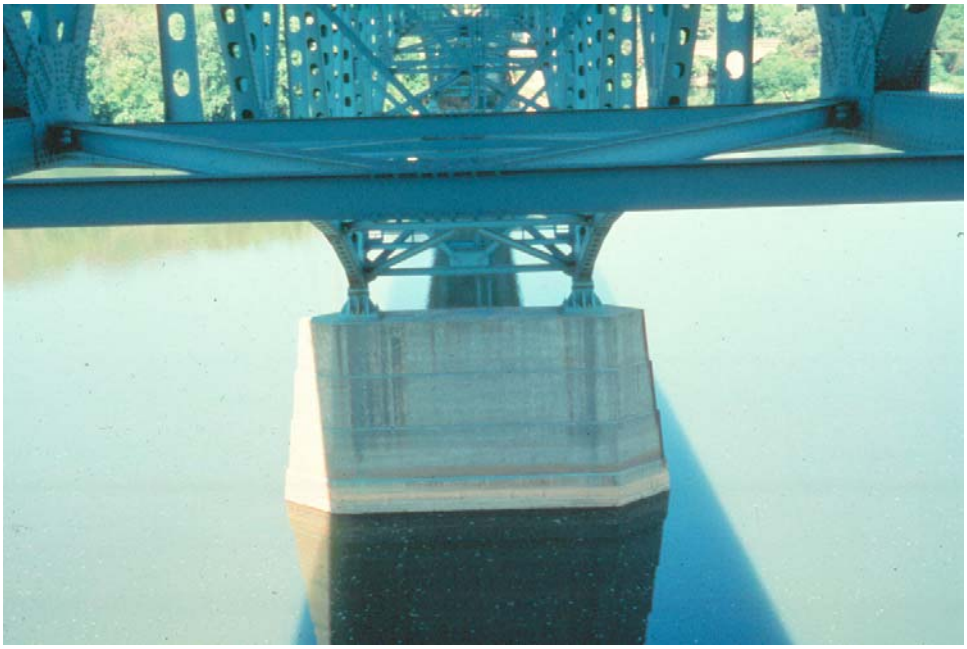


Figure 10.2.2 Solid Shaft Pier

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 10.2.3 Column Pier

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

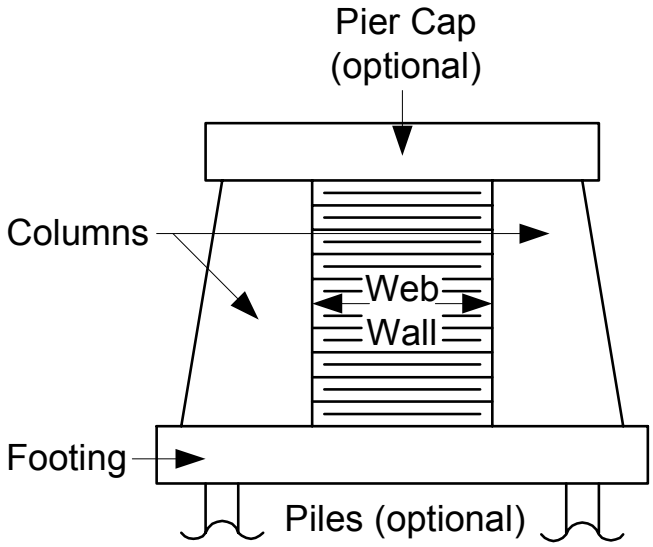


Figure 10.2.4 Column Pier with Web Wall

Columns are combined with a web wall when the column height is excessive, to add stability. They are non-structural relative to superstructure loads. Web walls also serve to strengthen the columns in the event of a vehicular collision.



Figure 10.2.5 Column Pier with Web Wall

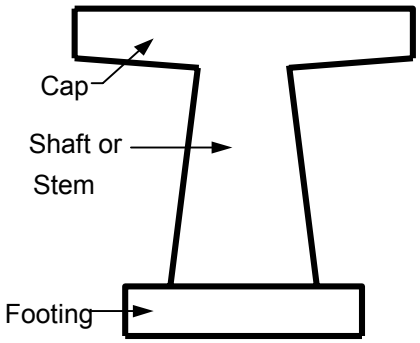


Figure 10.2.6 Single Stem Pier (Cantilever or Hammerhead)

The hammerhead pier is a modified column pier for use with multi-beam superstructures.



Figure 10.2.7 Cantilever Pier



Figure 10.2.8 Column Bent or Open Bent

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



Figure 10.2.9 Concrete Pile Bent

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

Two other specialized types of pier include the hollow pier and the integral pier. **Hollow piers** are usually tall shaft type piers built for bridges crossing deep valleys because it greatly reduces the dead load of the structure and increases ductility. Whether precast or cast-in-place, hollow piers are constructed in

horizontal 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 ease in transporting them 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 and are used in limited clearance situations. Integral piers provide for a more rigid structure, and they are typically used in situations where vertical clearance beneath the structure is limited. One example of an integral pier might be a cast-in-place cap within a steel girder superstructure. The cap would likely be post-tensioned rather than conventionally reinforced (see Figures 10.2.10 to 10.2.12).



Figure 10.2.10 Integral Pier



Figure 10.2.11 Integral Pier Cap



Figure 10.2.12 Integral Pier Cap

Primary Materials

The primary materials used in pier and bent construction are plain cement concrete, reinforced concrete, stone masonry, steel, timber, or a combination of these materials (see Figures 10.2.13 to 10.2.17).



Figure 10.2.13 Reinforced Concrete Piers under Construction



Figure 10.2.14 Stone Masonry Pier



Figure 10.2.15 Steel Bent



Figure 10.2.16 Timber Pile Bent



Figure 10.2.17 Combination: Reinforced Concrete with Steel Pier Cap

Primary 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 spirals or hoops (shear stirrups) in the columns. Pier caps are reinforced according to their beam function. Cantilevered caps have primary tension steel near their tops. Caps spanning between columns have primary tension steel near their bottoms. Primary shear steel consists of vertical stirrups, usually more closely spaced near supports.

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

All the concrete faces should be reinforced in both the vertical and horizontal directions. If primary steel is not required at a given location, then temperature and shrinkage steel will be provided.

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 large enough and thick enough to permit 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 10.2.18, 19, 20, and 21 illustrate typical reinforcement patterns.

In addition, new designs sometimes specify epoxy coated reinforcement if the structure will be subjected to de-icing chemicals.

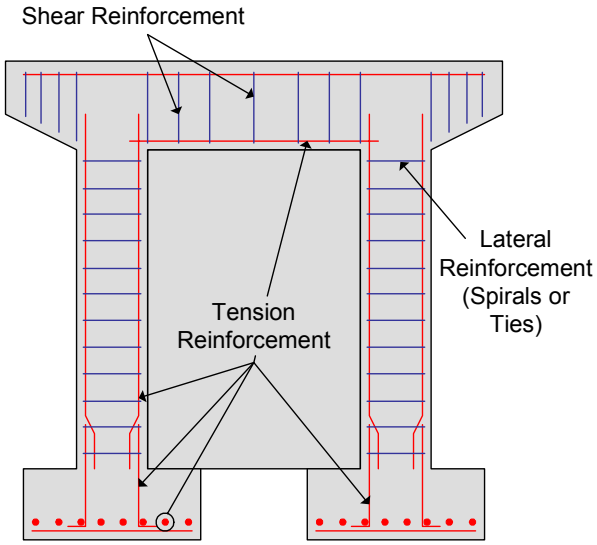


Figure 10.2.18 Primary Reinforcing Steel in Column Pier with Web

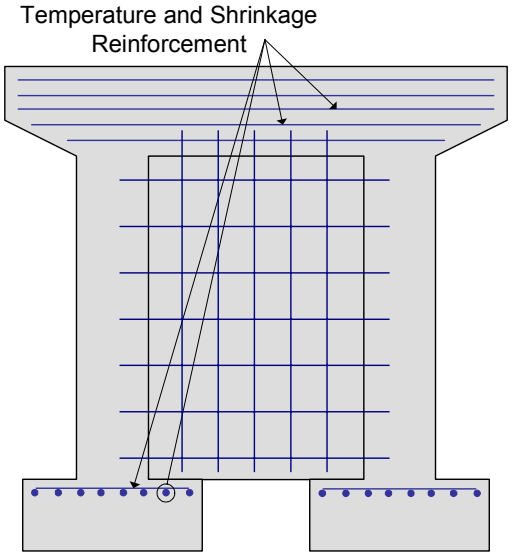


Figure 10.2.19 Secondary Reinforcing Steel in Column Bent with Web

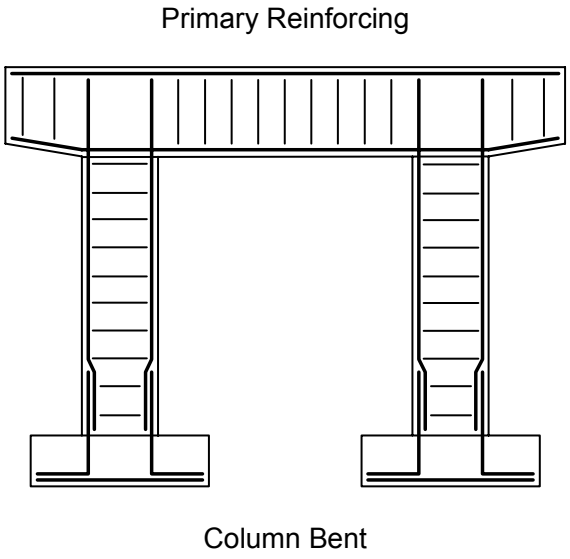


Figure 10.2.20 Primary Reinforcing Steel in Column Bents

Pier and Bent Elements

The primary pier and bent elements are:

- Pier cap or bent cap
- Pier wall / stem / or shaft
- Column
- Footing
- Piles (Foundation)

The **pier or bent cap** provides support for the bearings and the superstructure (see Figure 10.2.23).

The **pier wall** or **stem** transmits loads from the pier cap to the footing (see Figure 10.2.23).

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 when piles are not used. The footing also provides stability to the pier or bent against overturning and sliding forces (see Figure 10.2.23).

Piles are vertical or inclined column or members placed in the ground to carry the loads from the substructure to a suitable bearing stratum. They may be installed by driving, jacking or drilling into place. Piles may be timber, steel H shapes, steel pipes, precast concrete or cast in place concrete. They may transfer load directly to bedrock by point bearing or they may transfer load to the surrounding soil by friction. Piles may be entirely below grade, or may extend above grade to serve in lieu of columns. The above grade portions are referred to as “pile extensions”.

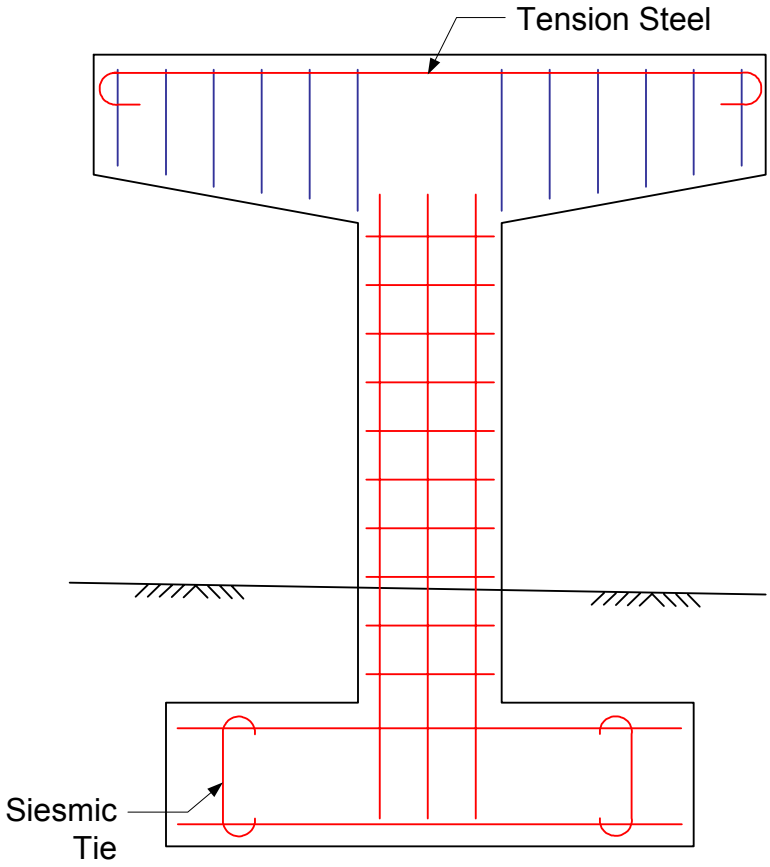


Figure 10.2.21 Cantilevers



Figure 10.2.22 Two Column Bent Joined by a Web Wall



Figure 10.2.23 Pile Bent

Pier Protection

Piers can be vulnerable to collision damage from trucks, trains, ships and ice flows. Wall type piers are resistant to this type of damage and are often used in navigable waterways and waterways subject to freezing for this reason. Web walls between columns also serve this function (see Figures 10.2.23, 24, and 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 marine or other traffic (see Figures 10.2.26 and 10.2.27). **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 10.2.28 and 10.2.29).

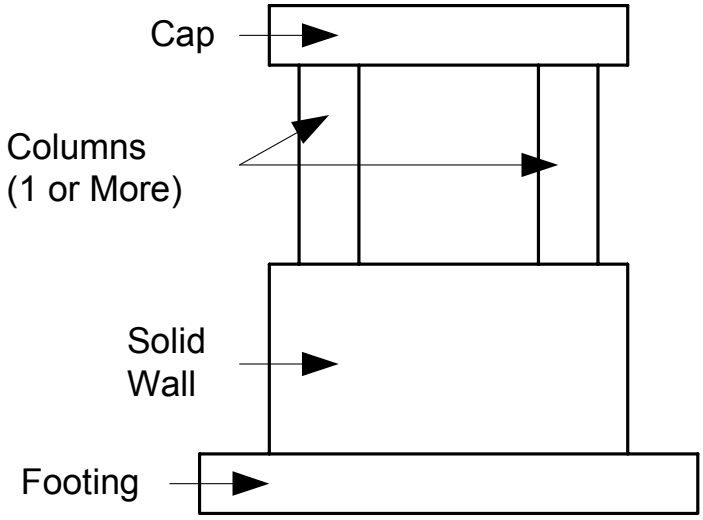


Figure 10.2.24 Collision Wall



Figure 10.2.25 Collision Wall Pier



Figure 10.2.26 Concrete Block Dolphin



Figure 10.2.27 Timber Dolphin



Figure 10.2.28 Pier Fender



Figure 10.2.29 Fender System

10.2.3 Inspection Locations and Procedures

Inspection procedures for piers and bents are the same as discussed for superstructures, particularly when it involves material deterioration. However, because stability is a paramount concern, checking for various forms of movement is required.

The locations for inspection are not particularly specific, but can be related to common pier and bent problems.

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

- Vertical movement
- Rotational movement and lateral movement
- Exposed Footings
- Material defects
- Dolphins and Fenders

Vertical Movement

Differential settlement at piers can cause serious problems in a bridge (see Figure 10.2.30). 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, and subsidence from mining or solution cavities.

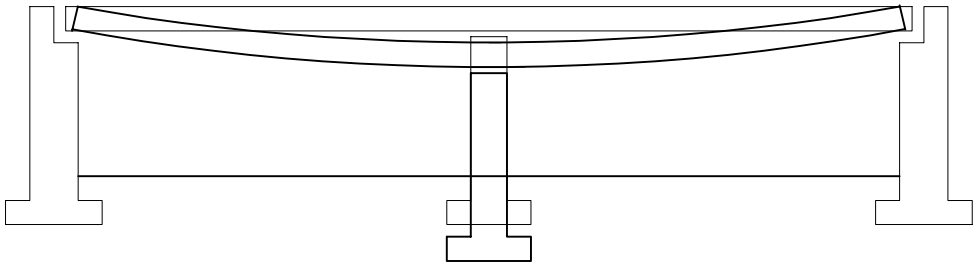


Figure 10.2.30 Differential Settlement Between Different Substructure Units

Inspection for vertical movement, or settlement, should include:

- 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, rails, etc. (see Figure 10.2.31).
- 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 10.2.31 Superstructure Evidence of Pier Settlement

Rotational Movement and Lateral Movement

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.

Inspection for rotational movement, or tipping, should include:

- Checking vertical alignment of the pier using a plumb bob or level.
- Investigating the clearance between the ends of beams at piers and between beams and backwall.
- Inspecting for cracking or spalling that may otherwise be unexplained; in the case of inspections after earthquakes, such damage will be readily apparent (see Figure 10.2.32).

Inspection for lateral movement should include checking the linear alignment of the bridge railing or barrier.

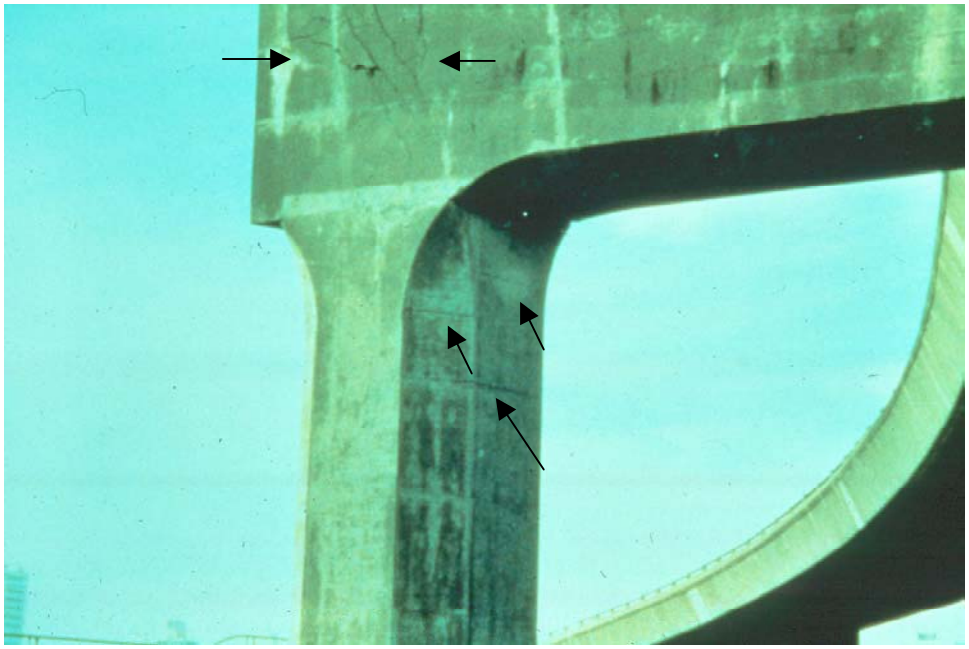


Figure 10.2.32 Cracks in Bent Cap due to Lateral Movement of Bent during Earthquake

Exposed Footings

Scour is the removal of material from a streambed as a result of the erosive action of running water (see Figure 10.2.33). Scour around piers has been the cause of several fatal bridge collapses. Refer to Topic 11.2 for a more detailed description of the various types of scour.



Figure 10.2.33 Pier Movement and Superstructure Damage due to Scour

Inspection for scour should include the following:

- Probe around the pier or bent for undermining (see Figure 10.2.34).
- Underwater inspection by divers may be required.
- Remote sensing using ground-penetrating radar.



Figure 10.2.34 Tipping of Bent due to Scour

Material Defects

Another common problem encountered during the inspection of piers and bents is material defects. Refer to Section 2 of this manual for detailed descriptions of the types and causes of deterioration observed in timber, concrete, steel, and masonry.

Concrete

Inspection of concrete in piers and bents should include the following:

- Inspect for disintegration of the concrete, especially in the splash zone, at the waterline, at the ground line, and wherever concrete is exposed to roadway drainage (see Figure 10.2.35).
- Examine the pier columns and the pier bent caps for cracks (see Figure 10.2.36).
- Check the pier caps and bearing seats for cracking and spalling (see Figure 10.2.37).

- Examine grout pads and pedestals for cracks, spalls, and deterioration.
- Investigate any significant changes in clearance for pier movement.
- Check all pier and bent members for structural damage caused by collision or overstress (see Figure 10.2.38).
- Determine whether any earth or rock fills have been piled against piers, causing loads which were not provided for in the original design and producing unstable conditions.



Figure 10.2.35 Concrete Deterioration due to Contaminated Drainage

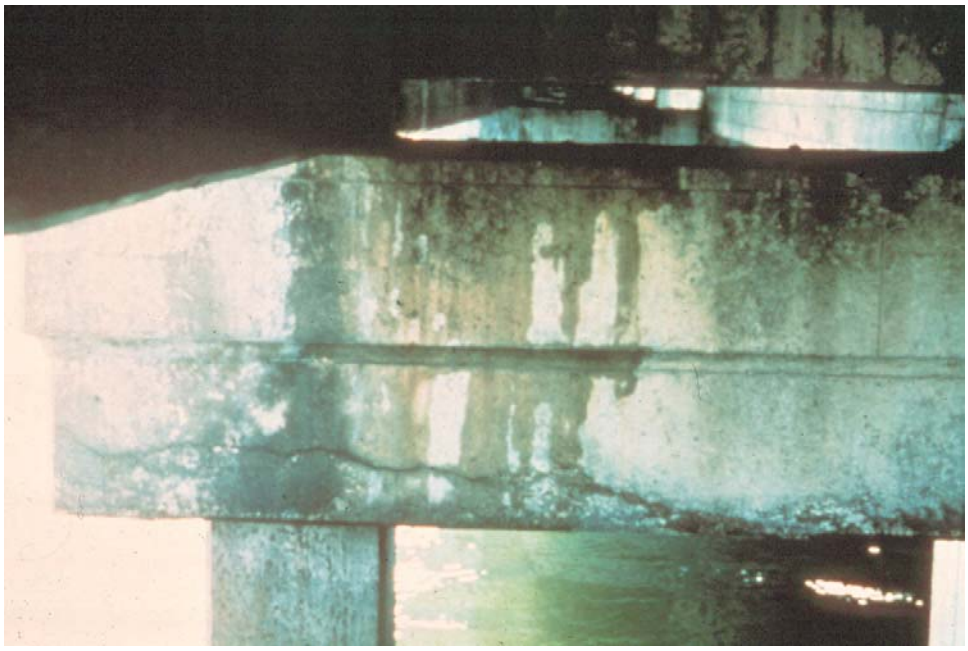


Figure 10.2.36 Crack in Concrete Bent Cap



Figure 10.2.37 Severe Concrete Spalling on Bent Cap



Figure 10.2.38 Collision Damage to Pier Column

Steel

Inspection of steel in piers and bents should include the following:

- Check pile bents for the presence of corrosion, especially at the ground line.
- Over water crossings, investigate the splash zone (i.e., up to 600 mm (2 feet) above high tide or mean water level) and the submerged part of the piles for indications of corrosion and loss of section (see Figure 10.2.40).
- Check for debris around the pile or pier bases; debris will retain moisture and promote corrosion.
- Examine the steel caps for rotation due to eccentric connections.
- Inspect the bracing for broken connections and loose rivets or bolts (see Figure 10.2.41).
- Check the condition of the web stiffeners, if present.
- Check the pier columns and pier caps for cracks (see Figure 10.2.42).
- When there are any significant changes in clearance, visually inspect and

measure for pier movement.

- Examine all pier and bent members for structural damage caused by collision, buckling, or overstress.
- Where a steel cap girder and continuous longitudinal beams are framed together, inspect the top flanges, welds, and webs for cracking.



Figure 10.2.39 Deterioration of Concrete Pedestal Supporting Steel Column



Figure 10.2.40 Corrosion and Debris at Steel Pile Bent



Figure 10.2.41 Steel Column Bent



Figure 10.2.42 Steel Column Bent with Cantilever

Timber

Inspection of timber in piers and bents should include the following:

- Check for decay in the piles, caps, and bracing. The presence of decay can be determined by tapping with a hammer or by test boring the timber. Drilling with a decay detection device can also be used (see Figure 10.2.43). Refer to Topic 13.1 for a more detailed description of the various types of advanced inspection techniques. Inspect particularly at the ground line or waterline, joints and splices, checks in the wood, bolt holes, caps, or other connections, since decay usually begins in these areas (see Figures 10.2.44 to 10.2.46).
- Examine splices and connections for tightness and for loose bolts.
- Investigate the condition of the cap at those locations where the beams bear directly upon it and where the cap bears directly upon the piles. Note particularly any splitting or crushing of the timber in these areas.
- Observe caps and piles that are under heavy loads for excessive deflection(see Figure 10.2.47).
- In marine environments, check for the presence of marine borers, shipworms, and caddisflies (see Figures 10.2.48 to 10.2.49).
- Check for evidence of insect damage.



Figure 10.2.43 Drilling a Timber Bent Column for a Core Sample

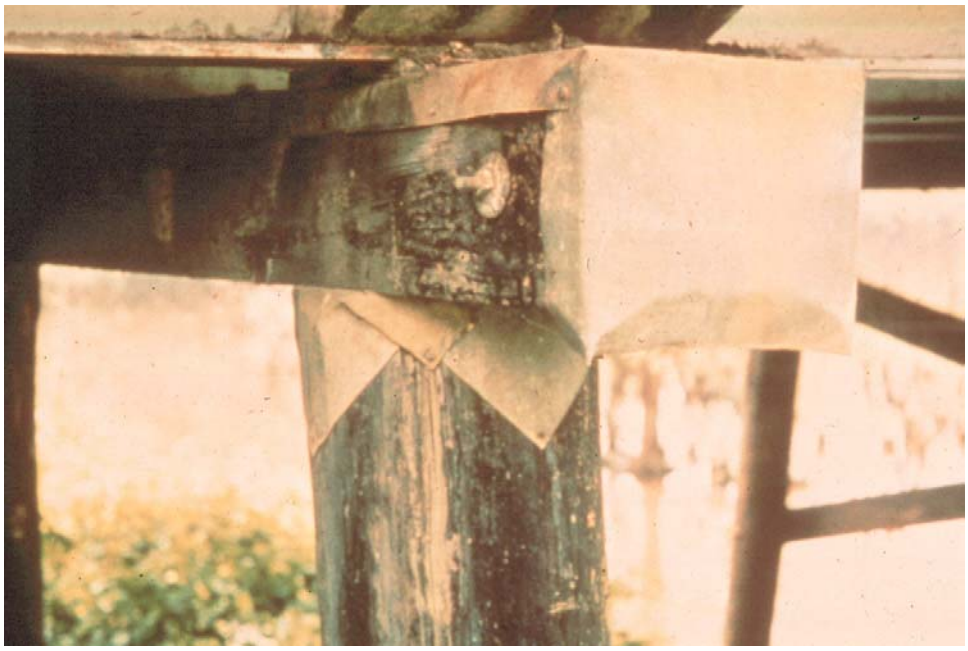


Figure 10.2.44 Decay in Timber Bent Cap Adjacent to "Protective" Cover



Figure 10.2.45 Timber Bent Columns in Water



Figure 10.2.46 Decay of Timber Bent Column at Ground Line



Figure 10.2.47 Timber Pile Bent with Partial "Brooming" Failure at First Pile



Figure 10.2.48 Timber Pile Damage due to Limnoria Marine Borers



Figure 10.2.49 Timber Bent Damage due to Shipworm Marine Borers

Another common problem encountered during the inspection of piers and bents is material defects. Refer to Section 2 of this manual for detailed descriptions of the types and causes of deterioration observed in timber, concrete, steel, and masonry.

Stone Masonry

Inspection of stone masonry in piers and bents should include the following:

- Check stone masonry piers for mortar cracks, water and vegetation in the cracks, and for spalled, split, loose, or missing stones (see Figure 10.2.50).



Figure 10.2.50 Deteriorated and Missing Stone at Masonry Pier

Dolphins and Fenders

The condition of dolphins and fenders should be 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 10.2.51). Investigate for hour-glass shaping of piles due to abrasion at the waterline, and check for structural damage caused by marine traffic.



Figure 10.2.51 Concrete Dolphins

In steel pier protection members, observe the splash zone (i.e., up to 0.6 m (2 feet) above high tide or mean water level) carefully for severe corrosion. Where there are no tides, check the area from the mean water level to 0.6 m (2 feet) above it. Also examine all other steel parts for corrosion, and check for structural damage (see Figure 10.2.52).



Figure 10.2.52 Steel Fender

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 10.2.53). Also, check for hourglass shaping of piles at the waterline.



Figure 10.2.53 Timber Fender System

10.2.4 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 National Bridge Inspection Standards (NBIS) rating and the Bridge Management System (BMS).

Application of NBIS Rating Guidelines

Using NBIS rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the substructure. Rating codes range from 9 to 0, where 9 is the best rating possible (see Topic 4.2).

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the pier or bent. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element Level Smart Flags are also used to describe the condition of the substructure, while identifying local problems that are not reflected in the CoRe elements.

In an element level condition state assessment of a pier or bent structure, the AASHTO CoRe element typically is one of the following:

Element No.	Description
201	Unpainted Steel Column or Pile Extension (m or ft)
225	Unpainted Steel Submerged Pile (EA)
230	Unpainted Steel Cap (m or ft)
202	Painted Steel Column or Pile Extension (m or ft)
231	Painted Steel Cap (m or ft)
204	Prestressed Concrete Column or Pile Extension (m or ft)
226	Prestressed Concrete Submerged Pile (EA)
233	Prestressed Concrete Cap (m or ft)
205	Reinforced Concrete Column or Pile Extension (m or ft)
210	Reinforced Concrete Pier Wall (m or ft)
220	Reinforced Concrete Submerged Pile Cap/Footing (EA)
227	Reinforced Concrete Submerged Pile (EA)
234	Reinforced Concrete Cap (m or ft)
206	Timber Column or Pile Extension (m or ft)
228	Timber Submerged Pile (EA)
235	Timber Cap (m or ft)
211	Other Pier Wall (m or ft)

The unit quantity for the pier cap elements is in meters (or feet), measured across the pier cap and the entire element must be placed in one of the four available condition states based solely on the substructure condition. The unit quantity for columns is each. Condition State 1 is the best possible rating for the abutment or wingwall. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For settlement of the pier or bent, the "Settlement" Smart Flag, Element No. 360, can be used and one of three condition states assigned. For scour at the piers or bents, the "Scour" Smart Flag, Element No. 361, can be used and one of three condition states assigned.

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Section 11
Inspection and Evaluation of Waterways

Topic 11.1 Waterway Elements

11.1.1 Introduction

Rivers are the most dynamic geomorphic system that engineers must 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, bridges do not usually move, other than in keeping with planned structural deflections resulting from anticipated static and dynamic loading of the structure.

There are several ways in which channels can change and thereby jeopardize the stability and safety of bridges. The channel bed can erode (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. 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.

Flooding is the main cause of bridge failures. Bridge damage from flooding is mainly caused by a phenomenon called scour, which erodes unstable material from the channel bottom, and which can undermine bridge foundations (see Figure 11.1.1). The extent of scour depends on the type and integrity of the major waterway elements. Of all the bridges in the National Bridge Inventory (NBI), approximately 86% are built over waterways.

Consequently, bridge inspectors need to understand the relationship between the bridge, stream, and floodplain. This understanding involves being able to recognize and identify the streambed, embankments, and stream flow so that an accurate assessment and record of the present condition of the bridge and waterway can be determined.



Figure 11.1.1 Pier Foundation Failure

11.1.2 Properties Affecting Waterways

An issue of major concern in bridge inspection is the safety of bridges that span active waterways. Properties of waterways that can affect structures are:

- The physical characteristics of the waterway (including streambed material and bank erosion)
- The physical characteristics of the bridge
- 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 waterway
- Changes in the river channel or flow due to development projects (such as dams, diversions, and channel stabilization) or natural phenomena
- The physical interaction between the abutments, piers, and footings supporting the bridge and the impact of hydraulic conditions on general scour and local scour (i.e., erosion of the channel bed)
- The physical condition of the riprap, revetments, spurs, and other structural devices that may have been utilized to help protect the bridge and adjacent channel
- Changes in the sediment balance in the stream due to nearby streambed stream gravel mining or landslides can cause streams to aggrade or degrade and become laterally unstable

11.1.3 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. 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 should be noted.

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 a very important step in ensuring the safety of the bridge. Over time, vertical changes, due to either degradation or aggradation processes, and 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.

11.1.4 Definition and Function of a Channel

The Channel is the well-defined depression that contains and guides stream flow during normal flow conditions (see Figure 11.1.2). It can be separated into three parts:

Elements of a Channel

- Streambed - the bottom or floor of the channel. The lowest elevation of the streambed is the "thalweg" elevation.
- Embankments - the sloped sides of the channel, which extend from the streambed to the surrounding ground elevation (floodplain).
- Streamflow - the water, suspended particles, chemicals, and any debris moving through the channel.
- Thalweg – the line defining the lowest points along a channel.

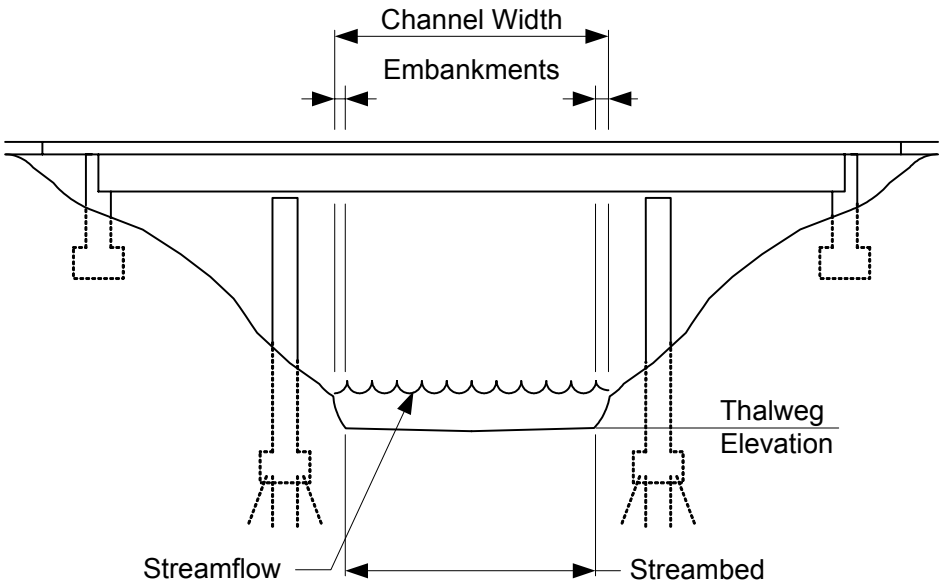


Figure 11.1.2 Typical Waterway Cross Section Showing Well Defined Channel Depression

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 forms 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 small velocities. They change plan form at a relatively slow rate and in a predictable manner, except during catastrophic flood events. Figure 11.1.3 illustrates the major characteristics of a meandering river.

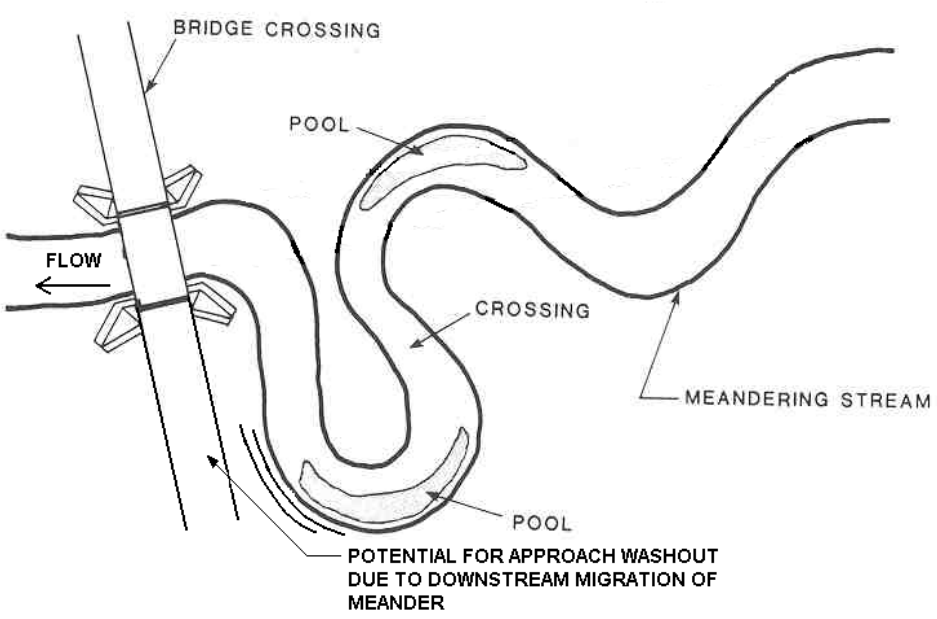


Figure 11.1.3 Meandering River

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. When compared with other forms of rivers, this type of channel:

- Has a steeper slope
- Experiences higher velocities
- Transports larger quantities of sediments
- Causes larger scour or erosion problems
- Is more difficult to "train"
- Requires careful engineering and continual maintenance of bridges subjected to this environment

Such rivers can change local plan form 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 11.1.4 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 control measures, such as riprap or spurs. The characteristics of straight rivers are identified in Figure 11.1.4.

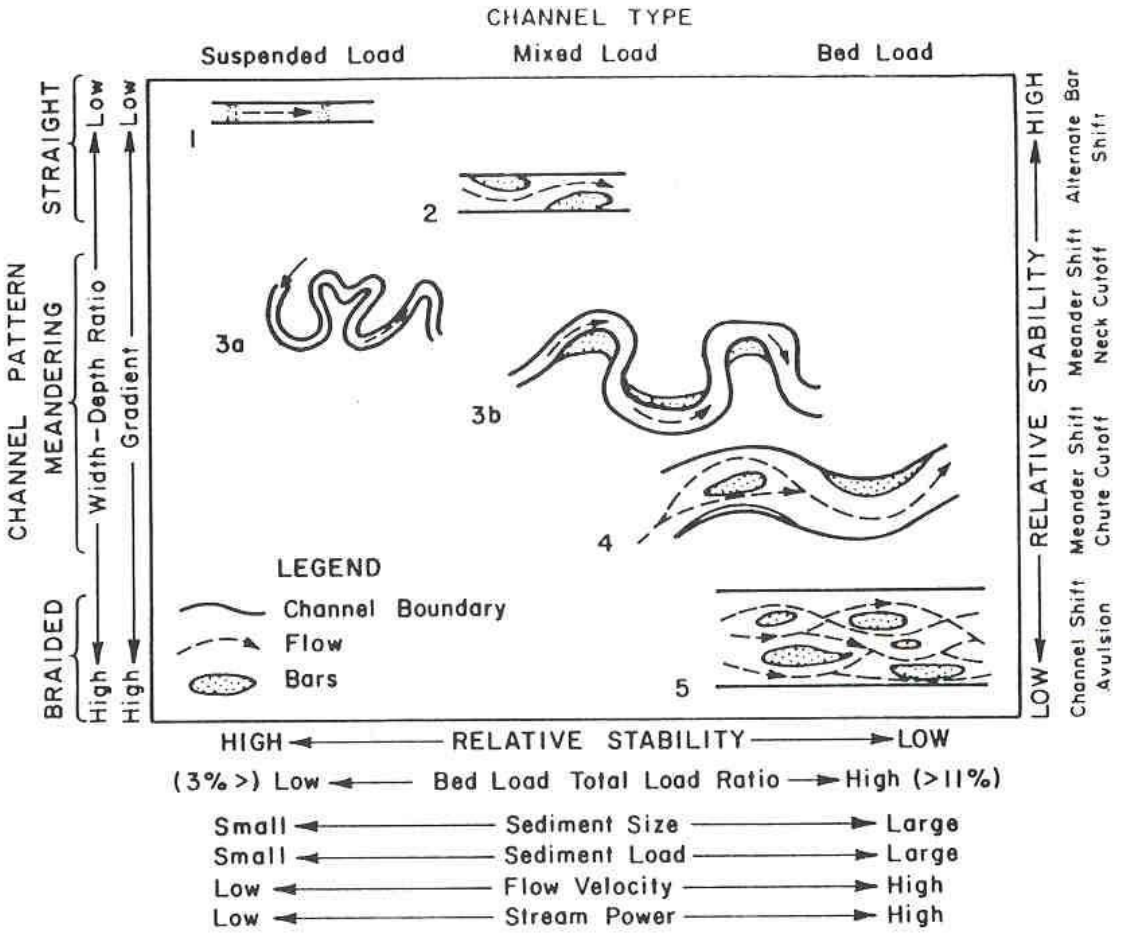


Figure 11.1.4 Plan View of Rivers

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 changes during episodic flood events.

11.1.5 Definition and Function of a Floodplain

The floodplain is the overbank area outside the channel that carries flood flows in excess of channel capacity (see Figure 11.1.5). It is common to find bridges built on 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.

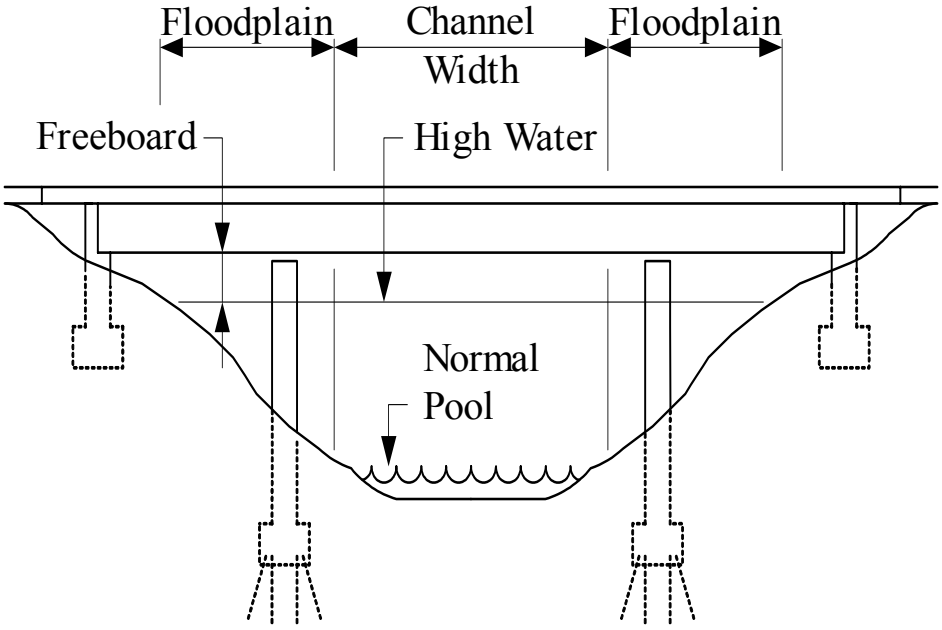


Figure 11.1.5 Typical Floodplain

11.1.6 Definition and Function of Hydraulic Openings (Bridge Waterway Openings)

The hydraulic opening is the entire area beneath the bridge which is available to pass flood flows (see Figure 11.1.6). The bottom of the superstructure, the two bridge abutments, and the streambed or ground elevation binds the hydraulic, or waterway, opening. For multiple spans, intermediate supports such as piers and columns restrict the hydraulic or bridge waterway opening.

A common term, freeboard, is used to describe the distance from the bottom of the superstructure to the top of the water surface at a specific reference point. Measurements of freeboard can fluctuate due to the flow rate in the waterway and the elevation of the streambed and floodplain. This measurement, in conjunction with average water depth, enables bridge inspectors to detect sudden or drastic changes in the water elevation from inspection to inspection by comparing freeboard measurements from past inspections. The term design freeboard is the expected freeboard at design flood flow.

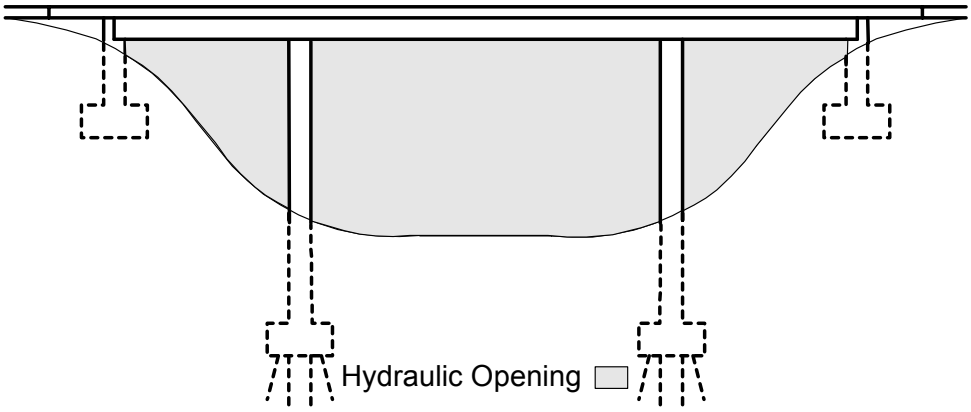


Figure 11.1.6 Hydraulic Waterway Opening

11.1.7 Definition and Function of Hydraulic Control Structures

To provide protection for bridges against lateral migration of the channel and against high velocity flows and scour, structural methods are often utilized. A hydraulic control structure is a man-made or man-placed device designed to direct stream flow and protect against undermining or lateral migration. These flow control structures may be utilized either at the bridge, upstream from the bridge, or downstream from the bridge. Hydraulic control structures can be riprap, spurs, guidebanks, gabions, slope stabilization, channel linings, footing aprons, or other flow-control structures installed to control stream flow and flood flows within the watercourse and through the bridge waterway opening. Hydraulic control structures can be man-made or natural materials placed by man. Some of the more common hydraulic control structures include:

- Riprap consists of properly sized and graded rock that is either natural or manmade, placed adjacent to abutments, piers, or along embankments (see Figure 11.1.7). 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. Such riprap must be of suitable dimensions and of proper gradation. It must be placed on an adequately flat slope to be able to resist the forces of the flowing water and of gravity. This generally requires placement of the riprap on side-slopes that range between 1.5 horizontal to 1 vertical (1.5H:1V to 3H:1V), depending upon the design criteria followed. However, flatter side-slopes of 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.

- Spurs are devices designed to protect as well as redirect stream flow (see Figure 11.1.8). Common applications occur on meandering rivers. The spurs are placed at the outside of the meandering bends to redirect the flow and minimize lateral migration.

- Guidebanks are constructed so as to redirect flood flows smoothly through the bridge waterway opening without endangering end substructure units from scour (see Figure 11.1.9). Scour hole formation occurs at the ends of the guidebanks rather than at the structure.

- Gabions consist of rectangular rock-filled wire mesh baskets anchored together and generally anchored to the surface which they are designed to protect, such as embankments and substructure footings (see Figure 11.1.10). Gabions may be placed on steeper slopes than riprap or may even be stacked vertically, depending upon the design procedure and the objectives of the placement of the gabions.

- Slope Stabilization is the placement of geotextiles, wire mesh, or plantings on the existing channel embankments (see Figure 11.1.11).

- Channel Lining is concrete, or other pavement on the channel embankment sometimes extending across the streambed. Channel linings also may be revetment mats or some other form of bed armoring (see Figure 11.1.12).

- 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 11.1.13). Footing aprons protect footings from undermining. The aprons are not a structural element of the abutment or pier footings.



Figure 11.1.7 Crushed Stone Riprap



Figure 11.1.8 Spurs Constructed on Mackinaw River (Illinois Route 121)



Figure 11.1.9 Guidebanks Constructed on Kickapoo Creek Near Peoria, Illinois



Figure 11.1.10 Gabion Basket Serving as Slope Protection



Figure 11.1.11 Wire Mesh and Grass Slope Stabilization



Figure 11.1.12 Concrete Revetment Mat (Photo Courtesy of CSI Geosynthetics)

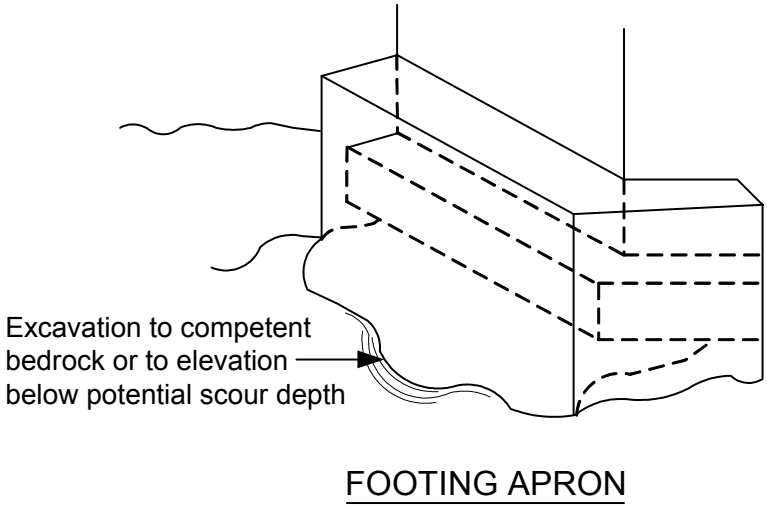


Figure 11.1.13 Concrete Footing Apron on a Masonry Arch Bridge

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Topic 11.2 Inspection of Waterways

11.2.1 Introduction

The bridge inspector must be able to correctly identify and assess waterway deficiencies when performing a bridge waterway inspection. Accurate bridge waterway inspections are vital for the safety of the motoring public. For this to happen, the bridge inspector should have a thorough understanding of the different types of waterway deficiencies, as well as the various inspection techniques.

Waterway deficiencies are properties of the waterway or substructure members that work to act negatively on the structural integrity of the bridge. They are mostly interrelated and when a change in one of these properties occurs, others are also affected.

11.2.2 Waterway Performance Factors

Waterway Alignment

In general, bridges are designed so that the flow passes through the waterway parallel to the axes of the abutments and the piers. If the path of flow shifts in direction as a result of continued bank erosion so that it approaches the abutments and the piers at a significant skew angle, the capacity of the waterway can be reduced. More significantly, local scour will be increased and may lead to the failure of the structure. This depends upon the original design conditions and the degree of change resulting in misalignment in the flow with the critical elements supporting the structure. Any change in direction of the approach of the flow to the bridge and any change in the angle at which the flow hits or impinges on the abutments and piers should be carefully noted. Observations of local change in flow directions and surveys of changes in bed and bank elevations must also be made. Evaluation of aerial photographs over time is extremely useful in assessing changes in flow directions. All of this information may be utilized to rate the severity of increasing misalignment in the flow on bridge safety.

Example of channel misalignment: If the approaching flow impinges on rectangular piers at an angle of 45 degrees versus flowing parallel to the axis of the piers, the depth of scour may be increased by a factor of two or more. The actual factor of increase depends upon the characteristics of the bed material, the pier type, and the duration of the flood.

For bridges spanning over wide floodplains keep in mind that the approach angle of the low flow channel may not be significant. In these cases it is the alignment of the floodplain flow during the larger floods that will determine the magnitude of local scour.

Streamflow Velocity

Streamflow velocity is a major factor in the rate and depth of scour. During flood events, the streamflow velocity is increased, which produces accelerated scour rates and depths. At high streamflow velocities, bridge foundations have the greatest chance to become undermined (see Figure 11.2.1).



Figure 11.2.1 Flood Flow Around a Pier Showing Streamflow Velocity

The streamflow velocity depends on many variables. One of these variables is the stream grade. A steep stream grade will produce high streamflow velocities, while a flat stream grade produces low streamflow velocities. Other variables that affect the streamflow velocity include the waterway alignment, the hydraulic opening, any natural or man-made changes to the stream, flooding, etc.

Hydraulic Opening

It is necessary to consider the adequacy of the hydraulic opening (the cross-sectional area under the bridge) to convey anticipated flows, including the design flood, without damage to the bridge. It is essential to maintain a bridge diary comparing original conditions in the waterway at the time the bridge was constructed to changes in the cross-sectional area of the channel under the bridge over time.

The primary method of assessing loss of cross-sectional area of the hydraulic opening is to determine channel bed elevation changes. This can be determined by a periodic survey of the channel bed or by taking soundings from the bridge. Typically, a number of survey or sounding points spaced across the bridge opening are established to determine changes in cross-sectional area. The lateral location of these surveyed points should be noted so that as subsequent inspections are conducted, the survey points can be repeated to maintain consistency. Photographs from key locations can be used to document debris and vegetation that can block the bridge opening.

Stream gages in the vicinity of the bridge may be useful in evaluating the adequacy of the waterway in relationship to changing hydraulic conditions. For example, stage-discharge curves based on discharge measurements by the United States Geological Survey (USGS) or other agencies and shifts in rating curves may indicate changes in channel bed elevation and cross section.

Streambed Material

The size, gradation, cohesion, and configuration of the streambed material can affect scour rates, depths, and significance of scour. The size of the streambed material has little effect on the depth of scour, but can affect the amount of time needed for this depth to be attained. Cohesive streambed materials that are fine usually have the same ultimate depth of scour as sand streambeds. The difference is that the cohesive streambeds take a longer period to reach this ultimate scour depth. Because of these reasons, the streambed type is important and should be correctly evaluated by the bridge inspector. Streambed rates of scour for different types of material are described later in this section.

Substructure Shape

Substructure members on old bridges were not necessarily designed to withstand the effects of scour. Wide piers and piers skewed to the flow of the stream are examples of substructure waterway deficiencies. These deficiencies potentially increase the depth of scour. Due to increased awareness of bridge waterway scour, recent substructure members have been designed to allow the stream to pass through with as little resistance as possible. Many newer piers have rounded or pointed noses, which can decrease the scour depth by up to 20%.

Foundation Type

The foundation type for the substructure members should be determined. Members that are undermined, but founded on piles are not as critical as spread footings that are undermined. The inspector should know the substructure foundation type, in order to properly evaluate the substructure and the waterway. The foundation type may often be determined from design and/or construction drawings. In some older bridges, the foundation type is not known. In this case, advanced inspection techniques by a trained professional may be required to verify the foundation type.

11.2.3 Waterway Deficiencies

Scour

The most common bridge waterway deficiency is scour, which may adversely impact bridge piers and abutments. Scour is the removal of material from the streambed or embankment as a result of the erosive action of streamflow.

Degradation and aggradation are long-term streambed elevation changes. Degradation is the gradual and even lowering of the streambed elevation due to a deficiency in sediment load available for transport via the stream (see Figure 11.2.2). Aggradation is the gradual and even rise in streambed elevation from deposition or buildup of streambed material, due to an overabundance of sediment load available for transport via the stream. (see Figure 11.2.3).



Figure 11.2.2 Streambed Degradation



Figure 11.2.3 Streambed Aggradation

The rate of scour will vary for different streambed materials, and for different streamflow rates. For a given streamflow rate, a streambed material will scour to a maximum depth in a given time. The following are examples for different types of streambeds and their corresponding scour rate:

- Dense granite: centuries
- Limestone: years
- Glacial tills, sandstone and shale: months
- Cohesive soils (clay): days

- Sand and gravel: hours

There are four forms of scour that must be considered in evaluating the safety of bridges:

- Long term scour (aggradation / degradation)
- Contraction scour
- Local scour
- Lateral stream migration

Long Term or Degradational Scour

Long term or degradation scour is the type that would occur whether or not there was a bridge crossing or constriction in the stream. Long term scour degrades the bed along some considerable length of the river (see Figure 11.2.4).



Figure 11.2.4 Severe Degradation at a Multi-Span Bridge Site

Long term scour may be a result of the natural erosion and downcutting process that rivers experience through the years. Long term scour may be accelerated by natural cutoffs in a meandering river, which steepens the channel gradient, increasing both the velocity of flow and hence scour. Long term scour may also be accelerated by various types of development or river modification, such as:

- Upstream dam construction
- Dredging
- Straightening or narrowing of the river channel

Changes in downstream elevation, such as at the confluence with another river which is undergoing long term scour of its own, can cause long term scour in the upstream river. Since long term scour involves degradation of the channel bed along some considerable distance of channel, major facilities are sometimes used to control erosion. These facilities can include a series of drop structures (small dam-like structures) or other erosion protection of the riverbed. Presence of such structures may be indicative that the channel is experiencing scour.

Factors that may cause changes in long term scour include:

- Water resources development, such as upstream diversions and upstream dams
- Changes in channel alignment
- Changes in channel dimensions
- Urbanization of the watershed (conversion of a more natural or agricultural area to a city)
- Other land use changes

These changing conditions may cause aggradation, loss of waterway cross section, or long term scour. This may reduce the degree of safety experienced by the abutments and the piers, considering the changed hydraulic conditions and the changed channel geometry. In this case, it is essential to refer to the bridge diary and study historical changes that have occurred in the general bed elevation through the waterway. If possible, these changes should be related to specific causes to assess the present safety of the bridge. These changes also provide insight as to future conditions that may be imposed by changed flow conditions, watershed development, or other conditions affecting the safety of the bridge.

Contraction Scour

Contraction scour results from the acceleration of flow due to a natural contraction, a bridge contraction, or both (see Figures 11.2.5 and 11.2.6). Contraction scour occurs whenever the bridge restricts the flow. Structural contraction of the channel width is usually dictated by savings in cost of the bridge but may be offset by increased foundation costs. Whenever the bridge constricts the natural width of the channel, contraction scour must be considered in the design and maintenance of the bridge (see Figure 11.2.7).

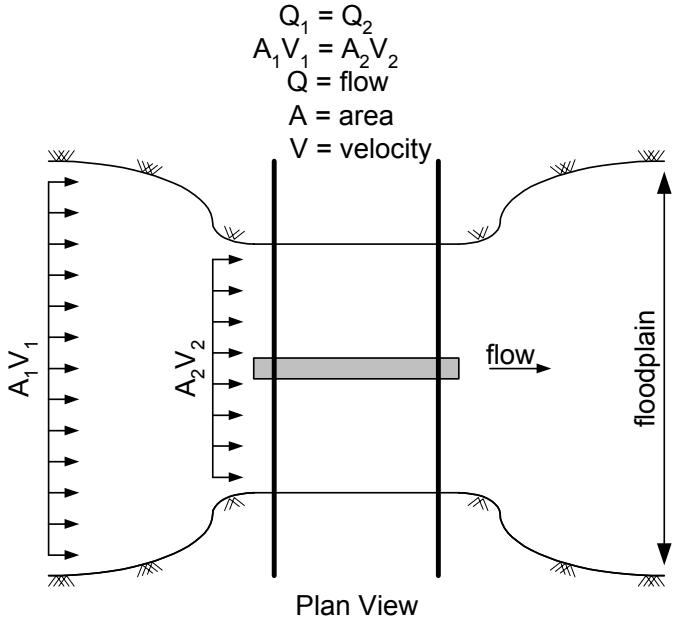


Figure 11.2.5 Severe Contraction Scour



Figure 11.2.6 Severe Contraction Scour at a Multiple-Span Bridge Site



Figure 11.2.7 Large number of Piers Combine to Reduce the Waterway Opening

Some common causes of contraction scour include:

- A natural stream constriction such as hard rock on embankment slopes.
- Excessive number of piers in the waterway (see Figure 11.2.7)
- Heavy vegetation in the waterway or floodplain (see Figure 11.2.8).

- Bridge roadway approach embankments built in the floodplain constricting the waterway opening. The overbank area of the floodplain is restricted by the bridge approach embankments extending partially across the floodplain.

- Formation of sediment deposits within the waterway along the inside radius of curved waterways (sandbars), and along embankments that constrict or reduce the available waterway opening (see Figure 11.2.9).

- Ice formation or ice jams that temporarily reduce the waterway opening and produce contraction (see Figure 11.2.10).

- Debris buildup, which often reduces the waterway opening (see Figure 11.2.11).

- Streamflow velocity increases due to downstream control of the water surface elevation (spillways).

The effects of contraction scour can be very severe.



Figure 11.2.8 Vegetation Constricting the Waterway



Figure 11.2.9 Sediment Deposits Within the Waterway Opening



Figure 11.2.10 Ice Jam



Figure 11.2.11 Debris Build-up in the Waterway

Local Scour

Local scour occurs around an obstruction that has been placed within a stream, such as a pier or an abutment and can either be clear-water scour or live-bed scour.

Clear-water scour occurs when there is no bed material transport upstream of the bridge. It occurs in streams where the bed material is coarse, the stream grade is flat, or the streambed is covered with vegetation except in the location of substructure members.

Live-bed scour occurs when local scour at the substructure is accompanied by bed material transport in the upstream waterway.

The cause of local scour is the acceleration of streamflow resulting from vortices induced by obstructions (see Figure 11.2.12). Some common obstructions are:

- Abutments – floodplain overbank flow is collected along and forced around abutments at high velocities (see Figure 11.2.13).
- Wide Piers - scour depth is proportional to width (see Figure 11.2.14).
- Long Piers - can produce multiple vortices and greater scour depth if the pier is at an angle to the flow direction (see Figure 11.2.15).

- Unusually Shaped Piers - can increase vortex magnitude. A square-nosed pier will have maximum scour depth, about 20 percent deeper than a sharp-nosed pier and 10 percent deeper than a cylinder or round-nosed pier.

- Bridge Piers Skewed to the Direction of Streamflow - can increase both contraction scour and local scour because of increased (projected) pier width effects. This skew can be dramatically different during low flow versus high flows.

- Depth of Streamflow - increases vortex effect on the streambed. An increase in flow depth can increase scour depth by a factor of 2 or more (see Figure 11.2.16).

- Streamflow Velocity - as streamflow velocity increases vortex action can be magnified considerably.

- Unstable Streambed Material - can contribute to the occurrence of local scour.

- Irregular Waterway Cross Section - can result in local scour at substructure units in the waterway.

- Debris Accumulation - and ice cakes piled up against piers can produce the same effect as a wider pier, increasing both contraction and local scour effects. Debris should be removed as a safety precaution to prevent pier failure

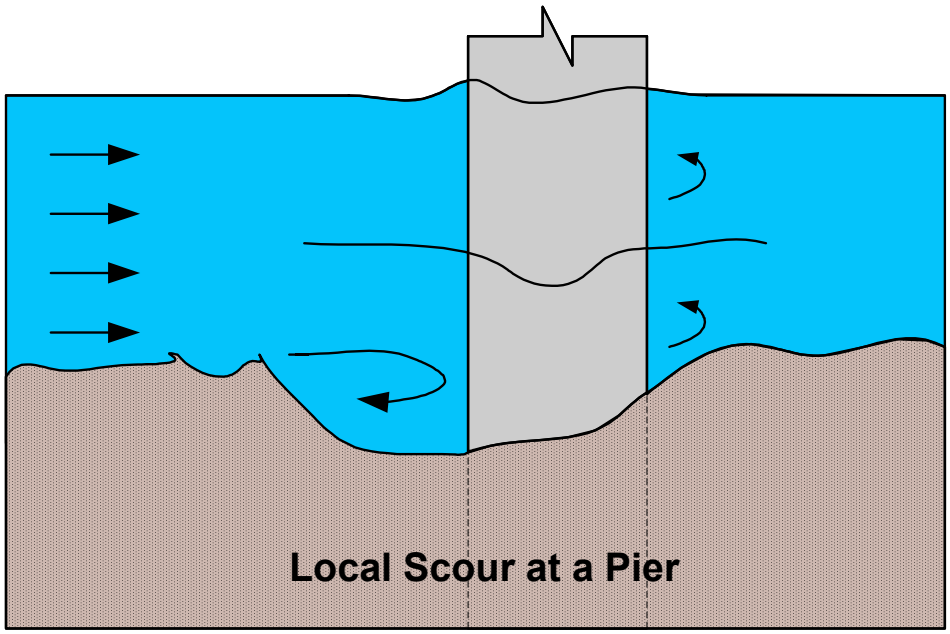


Figure 11.2.12 Horseshoe and Wake Vortices



Figure 11.2.13 Local Scour at an Abutment



Figure 11.2.14 Wide Pier



Figure 11.2.15 Long Pier

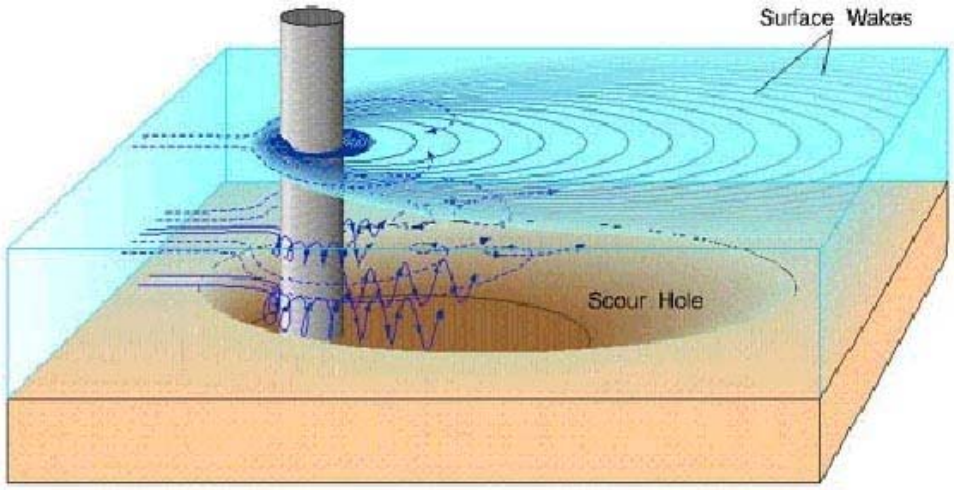


Figure 11.2.16 Local Scour Due to Streamflow Behavior in Deep Water

Generally, scour depths resulting from local scour are much larger than those from long term scour, often by a factor of ten. However, if there are major changes in hydrologic conditions resulting from such factors as construction of large dams and water resources development, the long-term bed scour can be the larger element in the total scour.

Bridges in tidal situations are particularly vulnerable to local scour. A strong tidal current whose direction reverses periodically causes a complex local scour phenomenon around a bridge pier. This local scour is caused by an imbalance between the input and output sediment transport rates around the pier, and it has a negative influence on the stability of the bridge.

To properly evaluate local scour and impacts of changes in hydrologic and

hydraulic conditions on local scour, it is essential to develop and refer to that component of the bridge diary which deals with local scour. With each inspection, critical supporting elements of the bridge should be subjected to careful survey to determine the degree of local scour that has developed over time. By referring to this history of change in local scour, it can be determined whether or not the maximum local scour has occurred and the relationship of this maximum local scour to bridge safety.

If the survey of the magnitude of local scour indicates increased local scour with time and furthermore verifies that the local scour exceeds the anticipated maximum local scour when the bridge was designed, remedial measures must be taken to protect the bridge. Surveys of local scour along the abutments and around the piers are most often done during periods of low flow when detailed measurements can be made, either by wading and probing, by probing from a boat, by the use of divers, or by sonic methods. The pattern of survey should be established and remain the same during the history of the bridge, following either a fixed radial or a rectangular grid. Changes in magnitude of local scour can then be compared at specific points over time.

The greatest problem associated with determining the magnitude of local scour relates to maximum local scour occurring at flows near flood peak followed by a period of deposition of sediments in the scour hole after the flood peak has passed and during low-flow periods. Consequently, a bridge rating should be based upon maximum scour that occurred during floods but not based upon examination of bed levels around abutments and piers during low-flow periods. Hence, it is necessary to use a variety of techniques to differentiate between maximum scour that may have occurred during flood periods and apparent scour after periods of low flow.

The inspector should consider utilizing straight steel or aluminum probing rods to probe loose sediments deposited along abutments and around footings; if sediment is finer than average bed material sizes or if the sediment is easily penetrated by the rod, it is indicative that the present sediment has accumulated in the scour hole and local scour is more severe than indicated by present accumulations of sediments. Core samples may also be used to differentiate between backfill in the scour hole and the bottom of the scour hole. It may be possible to use geotechnical means as another alternative to differentiate between materials that have deposited in the scour hole and the bottom of the scour hole. It may also be necessary to use underwater surveys using divers, or perhaps to even divert water away from critical elements to allow removal of loose backfill material. The inspector can then determine the true level of maximum scour in relationship to the bridge's supporting structural elements.

The problem of accurately determining maximum local scour and rate of change of local scour over time is one of the most difficult aspects of bridge inspection and is one of the most important aspects of evaluating bridge safety. Additional research is being conducted to provide better guidelines for investigating local scour in relationship to bridge safety.

Lateral Bank Migration

Another very important type of scour that can also threaten the stability of bridge crossings is lateral bank migration (see Figure 11.2.17). Embankment erosion typically results from lateral stream movement at a bridge opening and has often been the real culprit in a number of bridge collapses around the country, e.g., at the Hatchie River crossing in Tennessee. Bridge abutments are often threatened by this type of scour.



Figure 11.2.17 Lateral Bank Migration Endangering a Full Height Abutment

Lateral bank erosion is very common and can result from a variety of causes. Channel changes contributing to lateral bank erosion include:

- Stream Meander Changes (see Figure 11.2.18)
- Channel Widening or Degradation (see Figure 11.2.19)
- Manmade Channel Changes



Figure 11.2.18 Stream Meander Changes



Figure 11.2.19 Channel Widening

11.2.4 Effects of Waterway Deficiencies

Material Defects

Material defects that can be caused by waterway deficiencies include the deterioration and damage (i.e. abrasion, corrosion, scaling, cracking, spalling, and decay) to channel protection devices and substructure members.

As an integral part of the waterway inspection, careful consideration should be given to the identification of material defects. A loss of quality and quantity of materials required to provide bridge safety may occur in a variety of ways. Again, a careful record of changes in characteristics of materials should be recorded in the bridge diary. Using this procedure, changes over time can be compared and any decision concerning maintenance requirements or replacement becomes more straightforward with such historic information available.

Bridge Damage

Waterway deficiencies that are severe have the capability to cause damage to bridges. Effects of waterway deficiencies on bridge members include undermining, settlement, and failure.

Undermining

Undermining is the scouring away of streambed and supporting foundation material from beneath the substructure (see Figure 11.2.20). Local scour often produces undermining of both piers and abutments. Such undermining is a very serious condition, which requires immediate correction to assure the stability of the substructure unit.

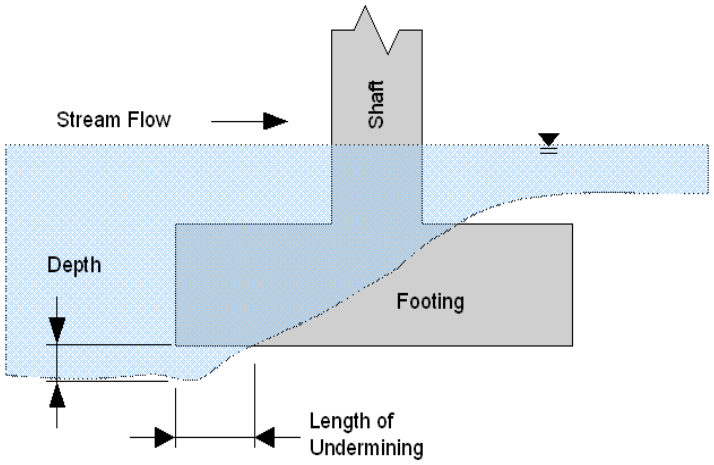


Figure 11.2.20 Longitudinal Cross Section Illustrating Undermining

The undermining of structural elements is basically an advanced form of scour. It is essential to determine whether or not undermining has a potential to develop, as well as whether it has already occurred. Undermining can pose an immediate threat to safety and must be dealt with immediately.

With small bridges, L-shaped rods can be used to probe at the base of abutments and around exposed footings to determine possible undermining. On the other hand, undermining may be very difficult to identify due to the redeposition of sediments during periods of low flow after undermining has occurred. However, in those channels where the bed is formed of coarse rock and the sediment supply to the bridge crossing is small, it is possible to inspect the footings and abutments because the backfill with fine sediments during periods of low flow generally does not occur. For these types of systems, an L-shaped rod can be used to identify that there has been some undermining and that critical conditions may exist.

For areas not accessible to effective probing from above water, it is essential to employ underwater inspection techniques utilizing divers. Whenever possible, the inspector should take detailed measurements, showing the height, width, and penetration depth of the cavities that have been eroded under these key-supporting elements of bridges. Refer to Topic 11.3 for a more detailed description of underwater inspections.

Settlement

Local scour undermining is typically most severe at the upstream end of the pier and, if not corrected, may result in pier settlement, with the upstream end settling first (see Figure 11.2.21).



Figure 11.2.21 Bridge Pier Under Flood Conditions that Settled at the Upstream End Due to Undermining Because of Local Scour

Failure

When undermining and settlement go undetected for some length of time, the bridge may become unstable, and be subject to failure or collapse. Failure may occur over a period of time, or it may be a very rapid process, as in the case of

failures due to flooding.

11.2.5 Inspection Preparation

It is necessary to identify and assemble the documentation and equipment required to conduct the waterway inspection. The required equipment will depend upon the characteristics of the river, the characteristics of the bridge, and the accessibility of the site.

Information Required

The following is a list of necessary information required for a comprehensive, well-organized inspection.

- Scour Evaluation Studies - Examine any previous hydraulic engineering scour evaluation studies on the bridge. These studies provide theoretical ultimate scour depths for the bridge substructure elements.
- Previous Bridge Inspection Reports - Review previous report data taken from successive inspections to establish whether the waterway is stable, degrading or aggrading.
- Streambed Material/Foundation Design - Determine streambed material, if possible, and type of substructure foundation from as-built, design and construction drawings.
- Site Conditions - Become familiar with site conditions such as channel protection installations, waterway depth, alignment, and previously reported waterway conditions. Also establish floodplain elevation and frequency.

Considering the complexity of the inspection and the equipment and materials needed to execute the inspection, the inspector should develop a detailed plan of investigation, as well as forms for recording observations. A systematic procedure should be used each time the bridge is surveyed to provide a means of accurately identifying changes that have occurred at the bridge site, which may affect the safety of the bridge.

Inspection Requirements

Prior to beginning the inspection, the bridge inspector should understand the type and extent of the inspection required. Waterway inspections are typically accomplished by either of two means:

- Surface or "Wading" Inspection - Submerged substructure, streambed and embankments are often accessible by inspectors using hip boots or chest waders and probing rods (see Figure 11.2.22). Additionally, boats are often used as a surface platform from which to gather waterway data, including channel cross-sections, pier soundings, etc.
- Underwater Diving Inspection – Site conditions often require waterway and submerged substructure units to be evaluated using divers, in order to obtain complete, accurate data. This is especially true when water depths are too great for wading inspection, and/or undermining of substructure elements is suspected.

Equipment

The type of equipment needed for a waterway inspection is dependent on the type of inspection. The following is a list that represents the most common waterway inspection equipment.

- Probing rods
- Waders (see Figure 11.2.22)
- Sounding line (lead line to measure depths of scour)
- Fathometer
- Diving equipment (see Figure 11.2.23)
- Truck, car, and/or trailer
- Boat, oars, motor, and anchor
- Surveying equipment (level or transit)
- Survey tapes and chains
- Level rod
- Compass
- Digital camera or camera and film (underwater or conventional)
- Video equipment and tapes (underwater or conventional)
- Underwater to surface communication equipment
- Forms or computer for recording information
- Past climatic and hydrologic records
- Stopwatch

Refer to Topic 3.4 for a more detailed list and description of inspection equipment.



Figure 11.2.22 Probing Rod and Waders

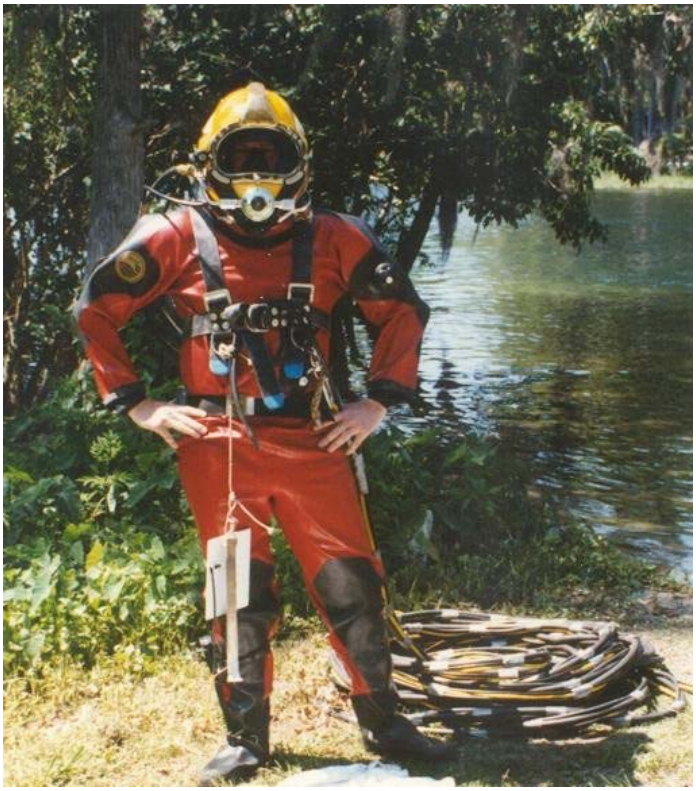


Figure 11.2.23 Surface Supplied Air Diving Equipment

Special Considerations

Special considerations should be given to the site conditions and the navigational controls that may adversely affect the safety of the bridge inspector and others. Some examples of these special considerations include:

- Site Conditions - Extreme flow velocity, pollution levels, safety concerns, conditions requiring special attention (see Figure 11.2.24).

- Navigational Control - The Coast Guard should be notified in advance of inspections where navigational controls are needed. Other navigational controls include boat traffic, operational status and condition of dolphins and fenders, dam releases (see Figure 11.2.25).



Figure 11.2.24 Rapid Flow Velocity



Figure 11.2.25 Navigable Waterway

11.2.6 Inspection Procedures and Locations

Inspection Procedures

- Visual
- Probing
- Measure/Document

Visual

The primary method used to inspect waterways is visual. The inspector must look at the site in the vicinity of the bridge. The inspector also needs to look at the flood plain. This observation may have to be done during periods of high water flow.

Probing

After the inspector gets the general condition by visually inspecting the bridge site, the next step is to probe for any scour or undermining. Care should be taken to adequately press the probing rod into the soil in the streambed. Sometimes scour holes are loosely filled with silt. This silt may be washed away quickly during the next period of high stream flow velocity, permitting additional scour.

Measure/Document

Measurements to obtain the cross section and profile must be taken. These measurements are used to analyze the area of the hydraulic opening and help determine need for and design of mitigation measures. The cross section under the bridge can be measured with a surveyor's tape or rod. The stream profile is measured with a hand level, survey tape and surveying rod (see Figures 11.2.26 and 11.2.27). The streambed profile and hydraulic opening should be compared to previous inspections.

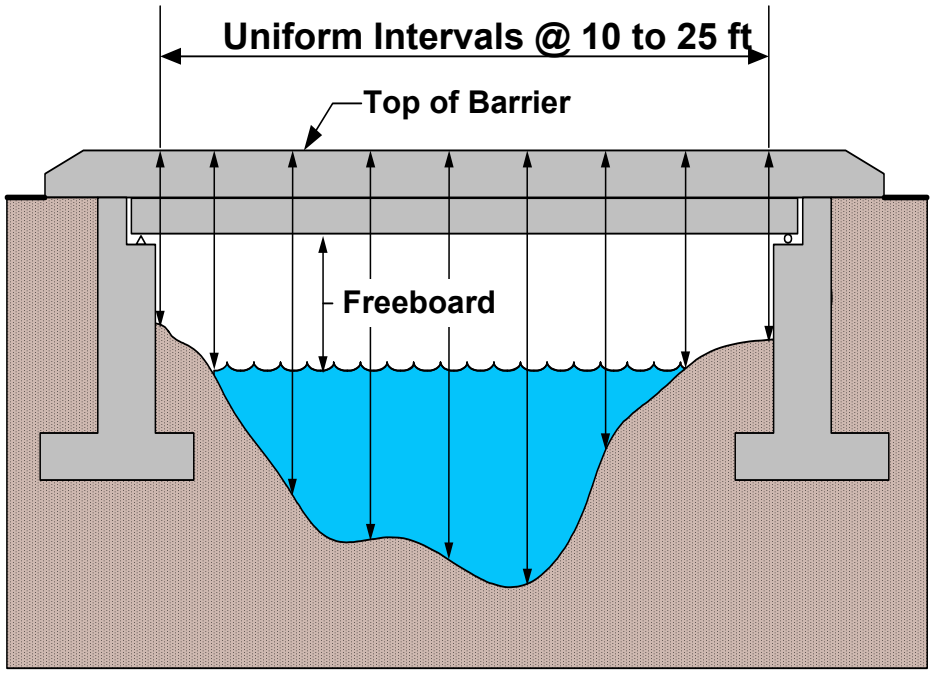


Figure 11.2.26 Streambed Cross-Section

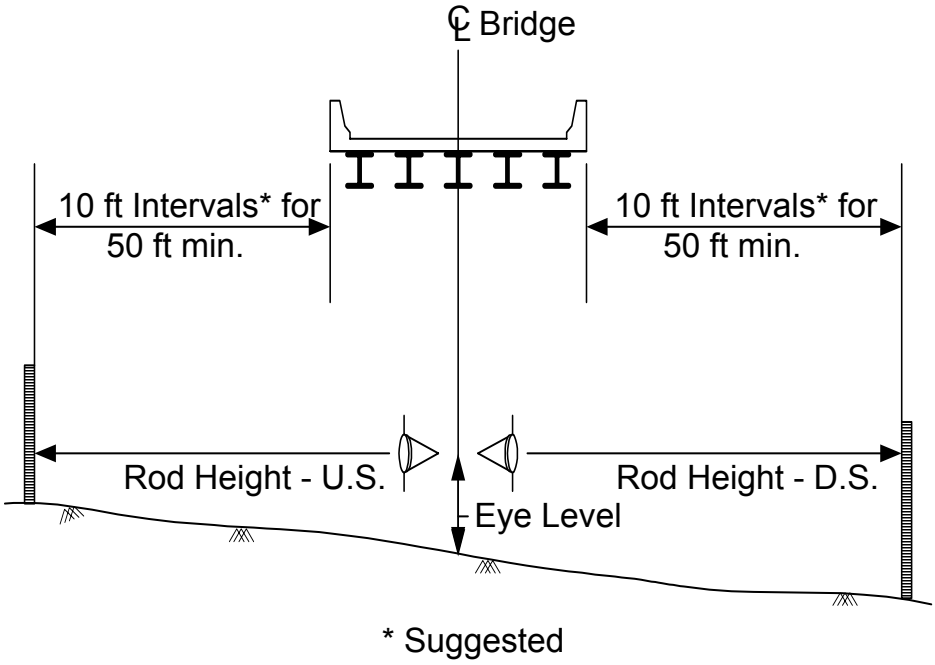


Figure 11.2.27 Streambed Cross-Section

When inspecting the bridge waterway, three main areas are of concern. These areas include the channel under the bridge, the upstream channel, and the downstream channel.

Channel Under the Bridge

Substructure

- Inspect substructure units below water level for defects, damage and foundation condition (see Figure 11.2.28).
- Measure heights and lengths of foundation element exposures, and dimensions of foundation undermining (opening height, width, and penetration depth), as applicable. Document with sketches and photos.
- Note location of high water mark on abutments/intermediate piers.
- Plumb face of abutments and piers for local settlement (see Figure 11.2.29).
- Check abutments and piers for accumulations of debris (drift).
- In case of damage to scour countermeasures check condition and function of channel protection devices adjacent to substructure units.
- In case of changes in streambed elevations generate streambed profile.
- In case of changes in streambed cross section generate streambed cross-sections for typical upstream, downstream, and under structure waterway configurations.
- Locate and contour large scour holes at the substructure.
- Establish a grid system for depth soundings at substructure elements, which can be repeated in subsequent inspections.
- Take photographs to document conditions of abutments, piers, and channel features.



Figure 11.2.28 Pile Bent Deterioration Normally Hidden Underwater

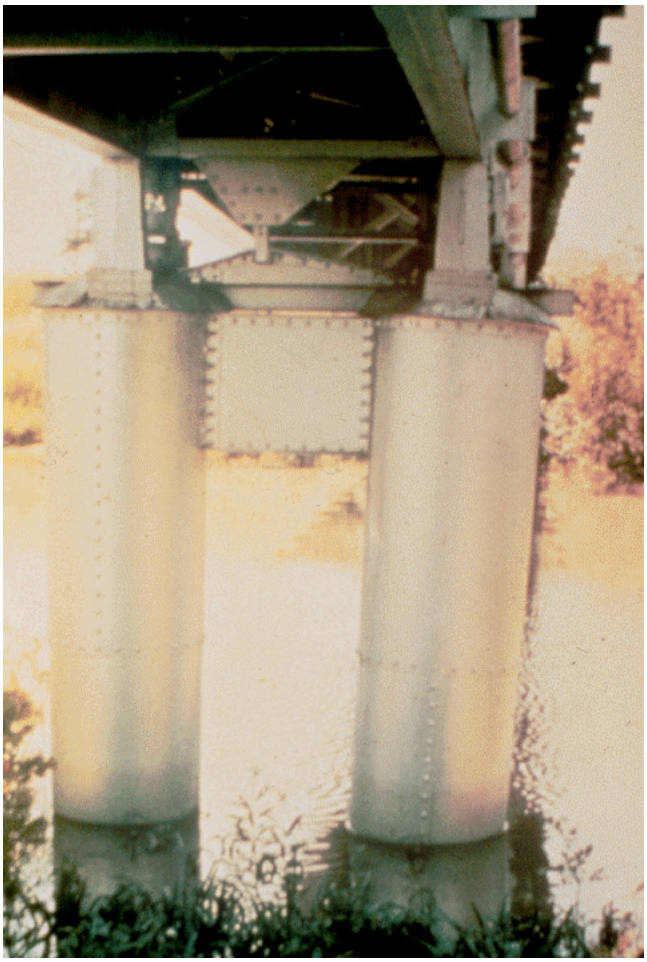


Figure 11.2.29 Out of Plumb Pier Column

Superstructure

During a waterway inspection, the superstructure can be a good indicator of existing waterway deficiencies.

The following items should be reviewed:

- Check to see if the superstructure is tied to the substructure to prevent washout.
- Sight along the superstructure to reveal irregularity in grade or horizontal alignment caused by settlement (see Figure 11.2.30).
- Check to see if debris is lodged in superstructure elements or tree limbs above the superstructure (see Figure 11.2.31).
- Check for high watermarks or ice scars on trees.
- Talk to local residents.
- Check any hydraulic engineering scour evaluation studies for overtopping flow elevation and frequency.



Figure 11.2.30 Superstructure Misalignment



Figure 11.2.31 Drift Lodged in a Superstructure

The following list should be checked when dealing with obstruction to flood flows.

- Check to see if the superstructure is in the floodplain.
- Check to see if the superstructure presents a large surface of resistance.
- Check bridge seats and bearings for transverse movement.

Bridge Design

Note if the superstructure is vulnerable to collapse in the event of excessive

foundation movement (i.e., simple span and non-redundant vs. continuous) (see Figure 11.2.32).



Figure 11.2.32 Typical Simple Multi-Span Bridge

Channel Protection and Scour Countermeasures

- Examine any river training and bank protection devices to determine their stability and condition.
- Check for any gaps or spreading that have occurred in the protective devices.
- Check for separation of slope pavement joints.
- Check for exposure of underlying erodible material.
- Inspect for steepening of the protective material and the surface upon which these materials are placed.
- Check for evidence of slippage of protective works.
- Check the condition and function of riprap as well as changes in size of riprap.
- Check for evidence of failed riprap in the stream (see Figure 11.2.33).
- Check for the proper placement, condition, and function of guidebanks, or spurs.
- Check the streambed in the vicinity of the channel protection for evidence of scour under the device.
- Check to see if the streamflow is impinging behind the protective devices.



Figure 11.2.33 Failed Riprap

It is essential to identify any change that is observable, including changes in the gradation of riprap. It is also essential to carefully inspect the integrity of the wire basket where gabions have been used.

Disturbance or loss of embankment and embankment protection material is usually obvious from close scrutiny of the embankment. Unevenness of the surface protection is often an indicator of the loss of embankment material from beneath the protective works. However, loss of embankment material may not be obvious in the early stages of failure. The inspector should also look for irregularities in the embankment slope.

It is even more difficult to determine conditions of the protective works beneath the water surface. In shallow water, evidence of failure or partial failure of protective works can usually be observed. However, with deeper flows and sediment-laden flows, it will be necessary for the inspector to probe or sound for physical evidence to identify whether failure or partial failure exists.

Waterway Area

- Check the hydraulic opening with respect to the floodplain.
- Determine the streambed material.
- Check for degradation (see Figure 11.2.34).
- Check for local scour around piers and abutments and record data.
- Inspect during drought conditions when applicable.
- Check for contraction scour due to abutment placement, sediment build-up, and vegetation.
- Check for debris underwater, which may constrict flow or create local scour conditions.
- Check to see if the approach roadways are in the floodplain (see Figure 11.2.35).
- Examine approaches for signs of overtopping.

- Determine if the hydraulic opening is causing or has the potential to cause scour under the bridge.



Figure 11.2.34 Severe Streambed Degradation Evident at Low Water



Figure 11.2.35 Approach Roadway Built in the Floodplain

Upstream Channel

Banks

- Stable - gradually sloped, grass covered with small trees. Banks are still basically in their original locations. Slope stabilization measures are in place and intact (see Figure 11.2.36).

- Unstable - bank is sloughing due to scour, evidence of lateral movement,

damage to slope stabilization measures (see Figure 11.2.37).



Figure 11.2.36 Stable Banks



Figure 11.2.37 Unstable, Sloughing Banks

Main Channel

- Record the flow conditions (e.g. low or high).
- Estimate velocities using floats.
- Check for sediment buildup and debris, which may alter the direction of stream flow (see Figure 11.2.38).
- Check for cattle guards and fences, which may collect debris. The results may be sediment buildup, channel redirection, or an increase in velocity and contraction scour (see Figure 11.2.39).
- Determine the streambed material.
- Check for aggradation or degradation.
- Check the basic alignment of the waterway with respect to the structure and compare it to its original alignment (see Figure 11.2.40).
- Record the direction and distribution of flow between piers and abutments.
- Make sketches and take pictures as necessary to document stream alignment, conditions of bank protection works, and anything that appears unusual at each inspection.



Figure 11.2.38 Sediment Accumulation Redirecting Streamflow



Figure 11.2.39 Stick and Barbed wire Cattle Guard

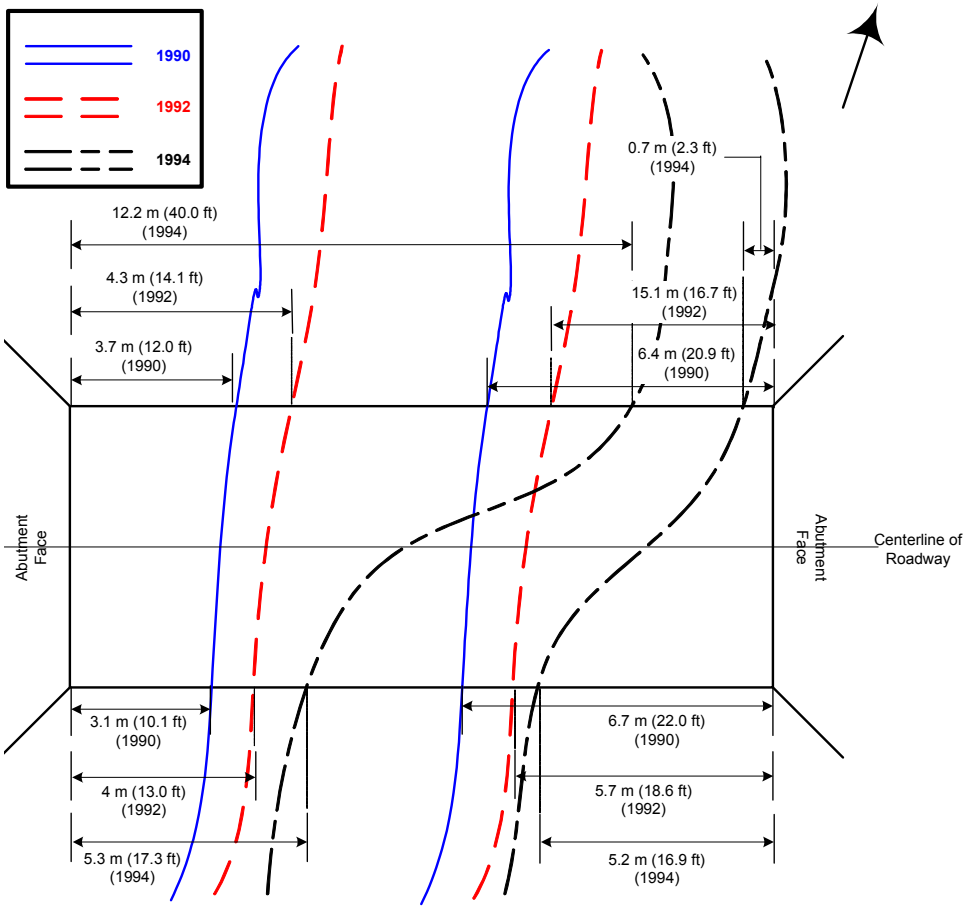


Figure 11.2.40 Waterway Alignment 1990 - 1994

Floodplain

- Check for evidence of embankment sloughing, undermining, and lateral embankment movement resulting from significant stream flow.
- Check for amounts and locations of debris, sediment accumulations, tree scaring, and amounts of vegetation growth, all of which may indicate the frequency of stream flow on the floodplain.
- Check for accumulations of sediments, debris, or significant vegetation growth in the waterway that may impact sufficient waterway adequacy and adversely affect streamflow under the main channel span (see Figure 11.2.41).
- Check for damage to the approach pavement, shoulders, and embankments to determine if the stream flow overtops the approach roadway during flood flows or returns to the main channel to flow under the structure.
- Check the extent of structures, trees, and other obstructions that could impact stream flow and adversely affect the bridge site.

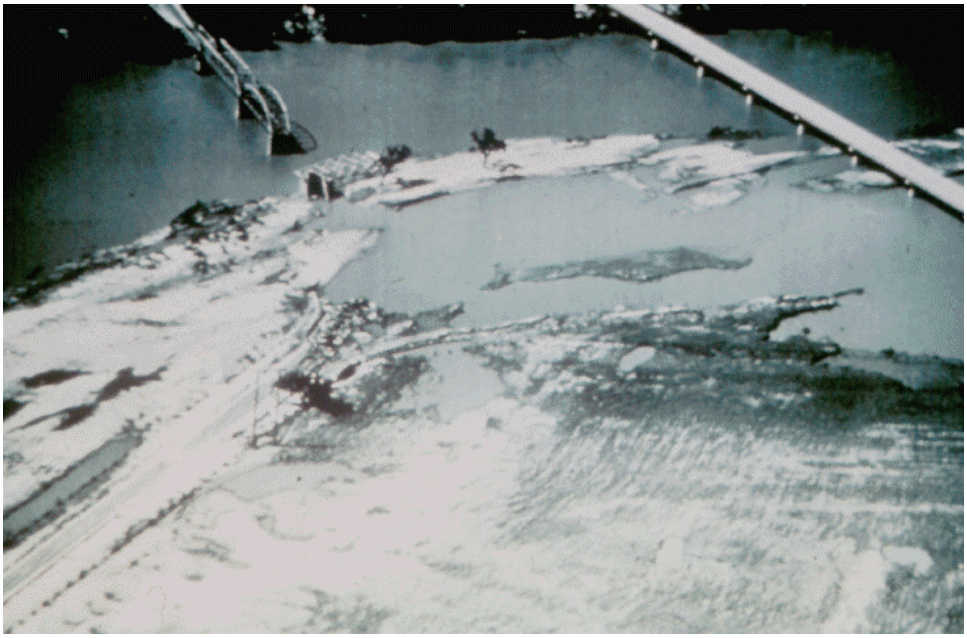


Figure 11.2.41 Approach Spans in the Floodplain

Other Features

- Check for streamflow impact of any other features such as tributaries, confluence of another waterway, dams, and substructure units from other bridges (see Figure 11.2.42).
- Report any recent construction activity (e.g. causeways, fishing piers, and stranded vessels) which may affect stream flow under the bridge.



Figure 11.2.42 Upstream Dam

Downstream Channel

Banks

Stable - gradually sloped grass covered with small trees. Banks are still basically in their original locations. Slope stabilization measures are in place and intact.

Unstable - bank is sloughing due to scour, evidence of lateral movement, damage to slope stabilization measures.

Main Channel

- Check for general alignment and buildup of sediment, which could redirect the stream flow (see Figure 11.2.43).
- Check for cattle guards and fences, fallen timber in the waterway, boulders or debris that may block or deflect the stream flow.
- Determine the streambed material type.
- Check for aggradation or degradation; check several hundred feet downstream for determining factors.
- Check the banks for evidence of lateral movement.
- Record the location of the waterway with respect to the bridge.
- Make sketches and take pictures as necessary to document stream alignment at each inspection.



Figure 11.2.43 Debris and Sediment in the Downstream Channel

Floodplain

- Check for any obstructions that would prevent constricted flow under the structure from returning to the floodplain.

Other Features

- Note any dams or confluence with larger waterways, which may cause variable tailwater depths. This may create conditions for high velocity flow through the bridge.

11.2.7 Evaluation

Scour Potential Assessment

Bridges over streams and rivers are subject to scour and should be evaluated to determine their vulnerability to floods and to determine whether they are scour critical.

Purpose and Objective

In a scour evaluation, professional engineers have to make engineering decisions on:

- Priorities for making bridge scour evaluations.
- The scope of the scour evaluations to be performed in the office and in the field.
- Whether a bridge is a scour critical bridge.

- Which scour countermeasures may reduce the bridge's vulnerability.
- Which scour countermeasures are most suitable and cost-effective for a given bridge.
- Priorities for installing scour countermeasures.
- Monitoring and inspecting scour critical bridges.

A responsibility of the bridge inspector is to gather on-site for a scour potential assessment that:

- Accurately records the present condition of the bridge and the stream.
- Identifies conditions that are indicative of potential problems with scour and stream stability.

To accomplish these objectives, the inspector needs to recognize and understand the potential for scour and its relationship with the bridge and stream. When an actual or potential scour problem is identified by a bridge inspector, the bridge should be further evaluated by an interdisciplinary team made up of structural, geotechnical, and hydraulic engineers (see Figure 11.2.44).



Figure 11.2.44 Scour at a Pile Abutment

Recognition of Scour Potential

The inspector must identify and record waterway conditions at the bridge, upstream of the bridge, and downstream of the bridge. Indications that could establish a scour potential include:

Waterway

- Streamflow Velocity - Major factor in the rate of scour. High velocities produce accelerated scour rates (see Figure 11.2.45).

- Streambed Materials - Loose cohesive soils, sand or gravel material,

which are highly susceptible to accelerated scour rates (see Figure 11.2.46).

- Orientation of Waterway Opening - Misaligned or skewed structure foundation elements, which can frequently generate adverse streamflow conditions which lead to scouring of the streambed especially during flood flows (see Figure 11.2.47).

- Floodplain - Large floodplains constricted to a narrow hydraulic opening under a structure can result in accelerated scour during flood flow, due to high velocities and changes in local flow direction (see Figure 11.2.48).

- Banks - Banks that are sloughing, undermined, or moving laterally are signs of potential scour at a bridge (see Figure 11.2.49).



Figure 11.2.45 Fast Flowing Stream



Figure 11.2.46 Streambed with Loose Gravel



Figure 11.2.47 Typical Misaligned Waterway



Figure 11.2.48 Typical Large Floodplain



Figure 11.2.49 Lateral Movement of Embankments

Substructure

The following condition of bridge foundations and substructure units should be considered in the inspector's scour potential assessment:

- Orientation - Piers and abutments that are not parallel with the stream flow especially during flood flow conditions, can lead to local scour of foundations (see Figure 11.2.450).

- Movement - Rotational or vertical movement of piers and abutments are evidence of undermining (see Figure 11.2.51).

- Type of Foundation - Spread footing foundation levels above maximum calculated scour depth determined for a particular streambed material are subject to undermining and failure. Exposed piling can be damaged or deteriorated and can lead to failure. Loss of supporting surrounding soil can also diminish pile capacity (see Figure 11.2.52).

- Hydraulic Opening - Restriction of the general waterway opening beneath the structure due to numerous large piers or simply an inadequate span length between abutments can increase streamflow velocities and lead to accelerated general scour (see Figure 11.2.53).



Figure 11.2.50 Parallel Orientation of Abutments

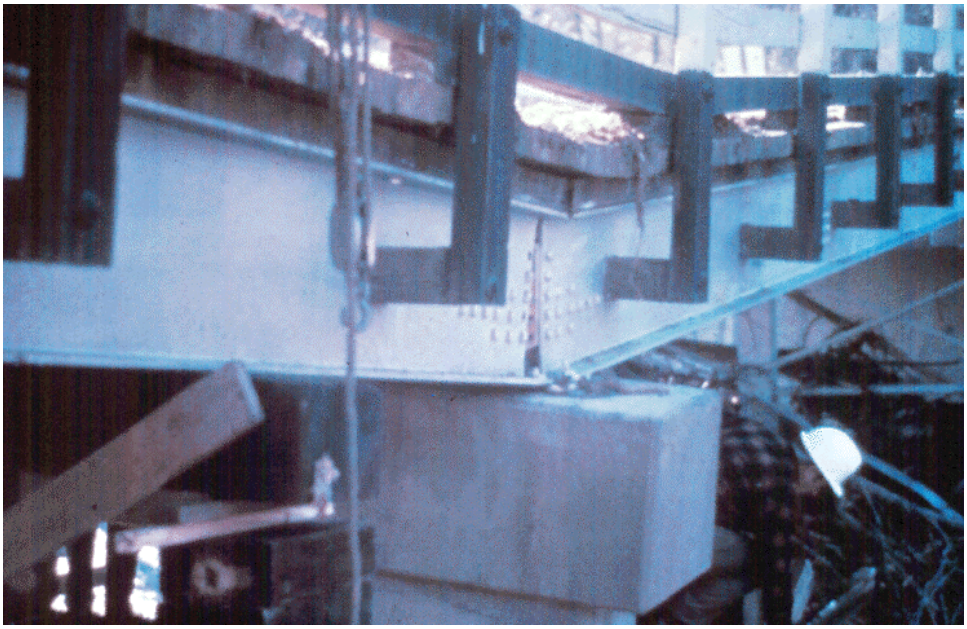


Figure 11.2.51 Rotational Movement and Failure Due to Scour



Figure 11.2.52 Exposed Piling Due to Scour



Figure 11.2.53 Accelerated Flow Due to Restricted Waterway

Superstructure

The following conditions associated with the superstructure should be considered in recognizing scour potential:

- Overtopping - Evidence of overtopping indicates insufficient hydraulic opening and excessive flow velocities.
- Freeboard - Insufficient freeboard can trap debris, increasing the potential for a washout.
- Design - Simple span designs are most susceptible to collapse in the event of foundation movement.

Application of National Bridge Inspection Standards (NBIS) Rating Guidelines

Scour Evaluation

The factors to be considered in a scour evaluation require a broader scope of study and effort than those considered in a bridge inspection. The scour evaluation is an engineering assessment of existing and potential problems and making a sound judgement on what steps can be taken to eliminate or minimize future damage.

In assessing the adequacy of the bridge to resist scour, the inspector and engineer need to understand and recognize the interrelationships between several items. The inspector can expedite the engineers' evaluation by considering the following:

- Substructure Condition Rating (Item 60)
- Channel and Channel Protection Condition Rating (Item 61)
- Waterway Adequacy Appraisal Rating (Item 71)
- Scour Critical Bridges (Item 113)

Substructure (Item 60)

This is the key item for rating the bridge foundations for vulnerability to scour damage. When a bridge inspector finds that a scour problem has already occurred, it should be considered in the condition rating of the substructure. If the bridge is determined to be scour critical, the condition rating for Item 60 should be further evaluated to ensure that any existing problems have been properly considered.

Channel and Channel Protection (Item 61)

This item permits rating the physical channel condition affecting streamflow through the bridge waterway. The condition of the channel, adjacent rip-rap, bank protection, guidebanks, and evidence of erosion, channel movement or scour should all be considered in establishing the rating for Item 61 (see Topic 4.2).

Waterway Adequacy (Item 71)

This is an appraisal item, rather than a condition item, and permits assessment of the adequacy of the bridge waterway opening to pass flood flows.

Scour Critical Bridges (Item 113)

This item permits a rating of current bridge conditions regarding its vulnerability to flood damage. A scour-critical bridge is one with abutment or pier foundations that are considered unstable due to:

- Observed scour at the bridge site, or
- Having scour potential as determined by a scour evaluation

When an actual or potential scour problem is identified by a bridge inspector, the bridge should be further evaluated by an interdisciplinary team comprised of structural, hydraulic and geotechnical engineers.

In this process, the effects of a 100-year flood (a flood which has a one percent chance of occurring in any year) would be considered, but the effects of a "superflood" (approximately, a 500-year flood or about 1.7 times the 100-year flood) would also be assessed and given one of three conditions.

- Safe Condition - if calculations indicate that the likely scour depth of the superflood would be above the top of the footing, the bridge would be considered safe or stable (see Figure 11.2.54).

- Evaluate Condition - if calculations indicate a scour depth within the limits of a spread footing or piles, further structural or foundation evaluation may be needed to establish the likely stability of the foundation (see Figure 11.2.55).

- Fix Condition - where there are indications that scour depth will lie below the bottom of the footing or piles, then the bridge would be considered clearly scour critical and would be at risk to damage or collapse (see Figure 11.2.56).

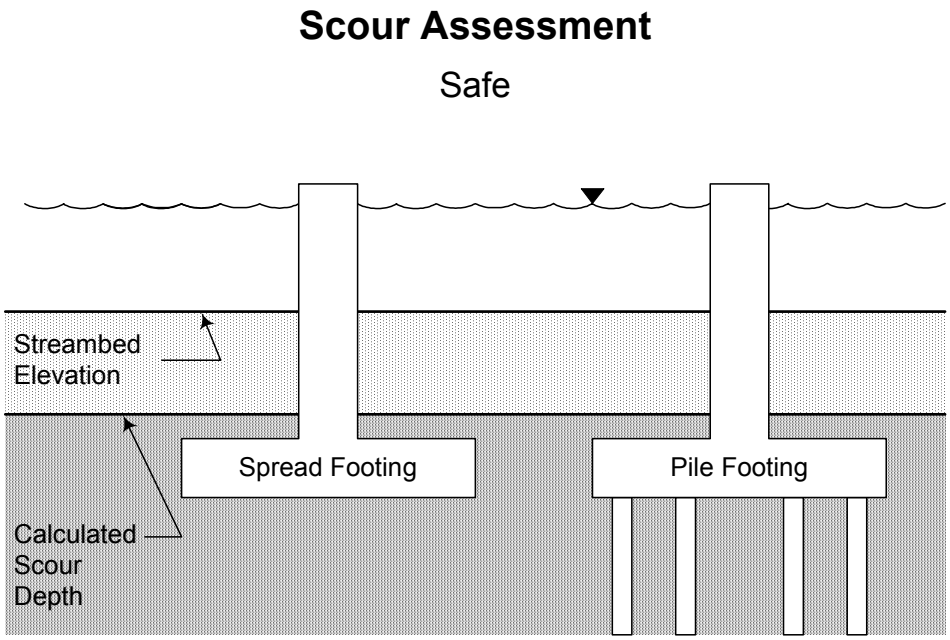


Figure 11.2.54 Scour Assessment - Safe

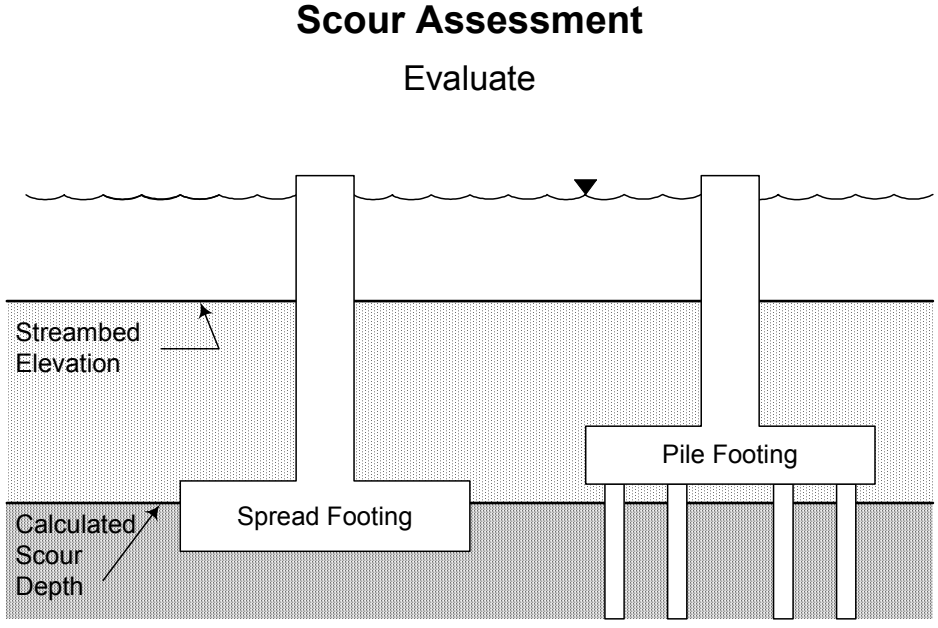


Figure 11.2.55 Scour Assessment - Evaluate

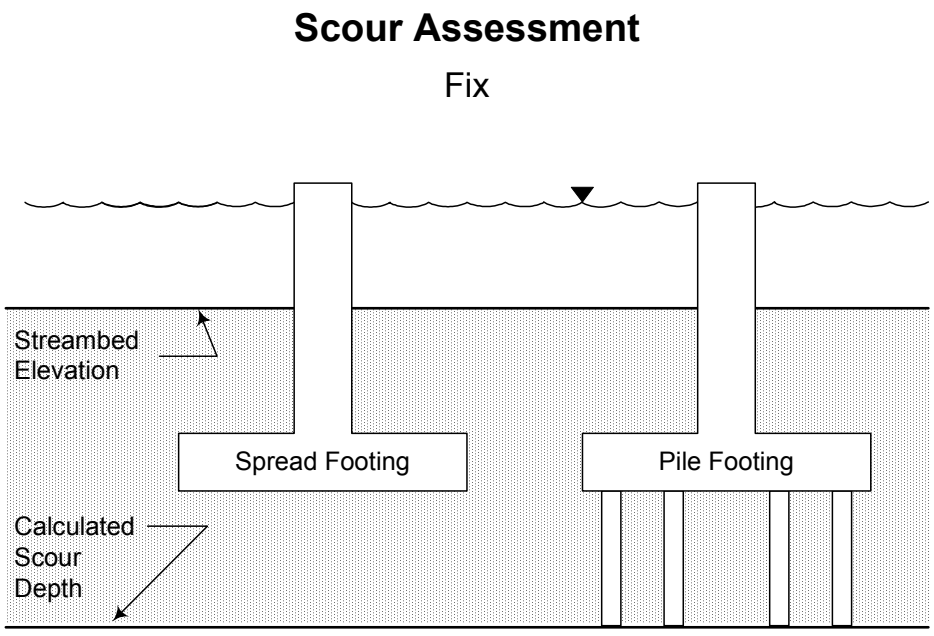


Figure 11.2.56 Scour Assessment - Fix

For scour critical bridges, a plan of action would then be developed for monitoring and correcting the scour problem. Such a plan would address the type and frequency of future inspections to be made and would include a schedule of timely design and construction actions for appropriate countermeasures to protect the bridge. The countermeasures might include the possibility of riprap, bed armoring, or flow-control structures or embankments.

Washouts of scour critical bridges, which appeared to be stable in the past, have still occurred, however, and probably will occur again in the future.

11.2.8 Culvert Waterway

The following excerpt is from a reproduction of the out-of-print Culvert Inspection Manual (Supplement to Manual 70), July 1986 – Chapter 5, Section 3:

Section 3. WATERWAYS

5-3.0 General.

The primary function of most culverts is to carry surface water or traffic from one side of a roadway embankment to the other side. The hydraulic design of culverts usually involves the determination of the most economical size and shape of culvert necessary to carry the design discharge without exceeding the headwater depth allowable. It is essential that the culvert be able to handle the design discharge. If the culvert is blocked with debris or the stream changes course near the ends of the culvert, the culvert may be inadequate to handle design flows. This may result in excessive ponding, flooding of nearby properties, and washouts of the roadway and embankment. In addition changes in upstream land use such as clearing, deforestation, and real estate development may change the peak flow rates and stream stability. It is therefore important to inspect the condition of the stream channel, SI&A item 61, and evaluate the ability of the culvert to handle

peak flows, SI&A item 71.

5-3.1 Stream Channel--What to Look for During Inspection.

The stream channel should be inspected to determine whether conditions exist that would cause damage to the culvert or surrounding properties. Factors to be checked include culvert location (horizontal and vertical alignment), scour, and accumulation of sediment and debris. These factors are closely related to each other. Poor culvert location can result in reduced hydraulic efficiency, increased erosion and sedimentation of the stream channel, and increased damage to the embankment and surrounding properties. A brief discussion of each of these factors is provided.

a. Horizontal Alignment - The inspector should check the condition of the stream banks and any bank protection at both ends of the culvert. He should also check for erosion and indications of changes in the direction of the stream channel. Sketches and photographs should be used to document the condition and alignment at the time of inspection. Abrupt stream alignment changes retard flow and may require a larger culvert; they cause increased erosion along the outside of the curve, damage to the culvert, and increased sedimentation along the inside of the curve. Where sharp channel curves exist at either the entrance or exit of a culvert, the inspector should check for sedimentation and erosion.

b. Vertical Alignment - Vertical alignment problems are usually indicated by scour or accumulation of sediment. Culverts on grades that differ significantly from the natural gradient may present problems. Culverts on flat grades may have problems with sediment build up at the entrance or within the barrel. Culverts on moderate and steep grades generally have higher flow velocities than the natural stream and may have problems with outlet scour. Scour and sediment problems may also occur if the culvert barrel is higher or lower than the streambed.

c. Scour - Erosion generally refers to loss of bank material and a lateral movement of the channel. Scour is more related to a lowering of the streambed due to the removal and transporting of stream bed material by flowing water. Scour may be classified into two types: local scour and general scour.

(1) Local scour is located at and usually caused by a specific flow obstruction or object, which causes a constriction of the flow. Local scour occurs primarily at the culvert outlet.

(2) General scour extends farther along the stream and is not localized around a particular obstruction. General scour can involve a gradual, fairly uniform degradation or lowering of the stream channel. It can also result in abrupt drops in the channel that move upstream during peak flows. This type of scour is referred to as head cutting.

Head cutting may be a serious problem if it is occurring in the channel downstream from the culvert, since it may threaten the culvert as it moves upstream. Head cutting may also occur in the stream channel immediately upstream from depressed inlets. Where upstream head cutting is usually not as serious a problem for the culvert, it can affect upstream structures and properties.

The upstream channel should be checked for scour that may undermine the culvert or erode the embankment. Scour that is undermining trees or producing sediment that could block or reduce the culvert opening should also be noted. The stream channel below the culvert should be checked for local scour caused by the culvert's discharge and for general scour that could eventually threaten the culvert.

d. Accumulation of Sediment and Debris - Deposits of debris or sediment that could block the culvert or cause local scour in the stream channel should be noted. Accumulations of debris or sediment in the stream may cause scour of the streambanks and roadway embankment, or could cause changes in the channel alignment. Debris and sediment accumulations at the culvert inlets or within the culvert barrel reduce the culvert's capacity and may result in excessive ponding. It also increases the chances for damage due to buoyant forces. Downstream obstructions, which cause water to pond at the culvert’s outlet, may also reduce the culvert's capacity. Debris collectors are used in some culverts so that the opening is not blocked by floating materials.

5-3.2 Waterway Adequacy - What to Look for During Inspection.

The preceding paragraphs dealt with evaluating the condition of the stream channel and identifying conditions that could cause damage to the culvert or reduce the hydraulic efficiency of the culvert. A closely related condition that must be evaluated is the waterway adequacy or ability of the culvert to handle peak flows, changes in the watershed, and changes in the stream channel which might affect the hydraulic performance. Guidelines for rating SI&A item 71, Waterway Adequacy, are presented in the Coding Guide.

a. High Water Marks - The high water elevation will vary with each flood but should still be checked to evaluate waterway adequacy. Ideally, culverts should be checked during or immediately after peak flows to determine whether water is being ponded to excessive depths, flooding adjoining properties, or overflowing the roadway, as shown in exhibit 63. High water marks are needed to define the upstream pond elevation and the downstream tailwater elevation. Several high water marks should be obtained, if possible, to insure consistency. High water marks in the culvert barrel, in the drain down area near the inlet, or near turbulent areas at the outlet are generally misleading. An inspection can also determine high water levels for peak flows by looking for debris caught on fences, lodged in trees, or deposited on the embankment. Information may also be obtained by interviewing area residents. Indications of excessive ponding, flooding, or

overtopping of the roadway should be investigated to determine the cause. If the cause is apparent, such as a blocked inlet, it should be reported for scheduling of appropriate maintenance. If the cause is not apparent, the culvert should be reported for evaluation by a hydraulic specialist.

b. Drainage Area - The inspector should be aware that changes in the drainage might have an effect on the discharge that culverts must handle. Replacement of an upstream culvert with a larger structure may eliminate upstream ponding, causing more water to reach the culvert sooner. Land clearing, construction, channel improvements, or removal of upstream dams or sediment basins may also affect discharge rates. Similarly, changes in land use may increase or decrease the amount of rainfall that infiltrates the ground and the amount that runs off. The inspector should note in the inspection report any apparent changes that are observed and be aware that changes a considerable distance upstream may affect the performance of downstream structures. Obstructions downstream from a culvert that back water up to the culvert may also affect the performance of the culvert.

c. Scour - As previously discussed, scour that changes the stream alignment at the ends of the culvert can reduce the hydraulic efficiency.

d. Sedimentation and Debris - Accumulation of debris and sediment at the inlet or within the culvert barrel reduces both the size of the opening and the culvert's capability to handle peak flows. Severe drift and sediment accumulations are illustrated in exhibits 64 and 65. However, culverts are occasionally designed with fill in the bottom to create a more natural streambed for fish.



Figure 11.2.57 (Exhibit 63) Culvert Failure Due to Overtopping



Figure 11.2.58 (Exhibit 64) Culvert Almost Completely Blocked by Sediment Accumulation

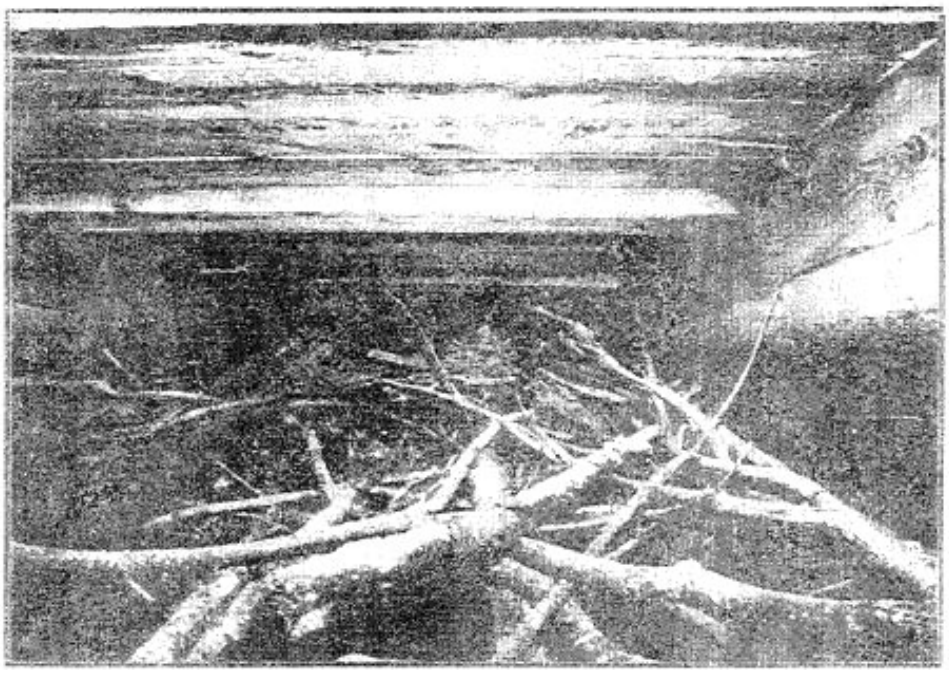


Figure 11.2.59 (Exhibit 65) Drift and Debris Inside Timber Box Culvert

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Topic 11.3 Underwater Inspection

11.3.1 Introduction

The need for underwater inspections is great. Approximately 86 percent of the bridges in the National Bridge Inventory (NBI) are built over waterways, and most bridge failures occur because of underwater problems. Underwater members must be inspected to the extent necessary to determine with certainty that their condition has not compromised the structural safety of the bridge.

Several bridge collapses during the 1980's, traceable to underwater deficiencies, have led to revisions in the National Bridge Inspection Standards (NBIS) (see Figure 11.3.1). As a result, bridge owners have been mandated to develop a master list of bridges requiring underwater inspections.



Figure 11.3.1 Schoharie Creek Bridge Failure

In general, the term "underwater inspection" is taken to mean a hands-on inspection requiring underwater breathing apparatus and related diving equipment. The expense of such inspections necessitates careful consideration of candidate bridges.

Bridge Selection Criteria

Bridges that cross waterways often have foundation elements located in water to provide the most economical total design. Where these elements are continuously submerged (see Figure 11.3.2), underwater inspection and management techniques must be used to establish their condition so that failures can be avoided.



Figure 11.3.2 Mississippi River Crossing

In many cases, a multi-disciplinary team including structural, hydraulic and geotechnical engineers must evaluate a bridge located over water. Underwater inspection is therefore only one step in the investigation of a bridge.

Selection Criteria

Various factors influence the bridge selection criteria. As a minimum, all structures must receive routine underwater inspections at intervals not to exceed five years. This is the maximum interval permitted between underwater inspections for bridges which are both in excellent condition underwater and which are located in passive, nonthreatening environments. More frequent routine and in-depth inspections may be desirable for many structures and necessary for critical structures. The bridge owner must determine the inspection interval that is appropriate for each individual bridge. Factors to consider in establishing the inspection frequency and levels of inspection include:

- Age
- Type of construction materials
- Configuration of the substructure
- Foundation Depth
- Adjacent waterway features such as dams, dikes, or marinas
- Susceptibility of streambed materials to scour
- Maintenance history
- Saltwater environment
- Waterway pollution
- Damage due to waterborne traffic, debris, or ice

Selected Bridges

Those bridges that require underwater inspection must be noted on individual inspection and inventory records as well as be compiled in a master list. For each bridge requiring underwater inspection, the following information should be included as a minimum:

- Type and location of the bridge
- Type and frequency of required inspection
- Location of members to be inspected
- Inspection procedures to be used
- Dates of previous inspections
- Maximum water depth and velocity (if known)
- Special equipment requirements
- Findings of the last inspection
- Follow-up actions taken on findings of the last inspection
- Type of foundation
- Bottom of foundation elevation or pile tip elevation

11.3.2 Methods of Underwater Inspection

There are three general methods used to perform underwater inspections:

- Wading inspection
- Self-contained diving (SCUBA)
- Surface-supplied diving

Wading Inspection

Wading inspection is the basic method of underwater inspection used on structures over wadable streams. The substructure units and the waterway are evaluated using a probing rod, sounding rod or line, waders, and possibly a boat. Regular bridge inspection teams can often perform wading inspections with waders and a life preserver or a boat (see Figure 11.3.3).



Figure 11.3.3 Wading Inspection

Self-contained Diving

In this mode, the diver operates independently from the surface, carrying his/her own supply of compressed breathing gas (typically air). SCUBA, an acronym for Self-Contained Underwater Breathing Apparatus, is the most common type of self-contained diving equipment used (see Figure 11.3.4). Self-contained diving is often employed during underwater bridge inspections. This dive mode is best used at sites where environmental and waterway conditions are favorable, and where the duration of the dive is relatively short. Extreme care should be exercised when

using self-contained equipment at bridge sites where the waterway exhibits low visibility and/or high current, and where drift and debris may be present at any height in the water column.



Figure 11.3.4 Self-Contained Inspection Diver

Surface-Supplied Diving

As its name implies, surface-supplied diving uses a breathing gas supply that originates above the water surface. This breathing gas (again, typically compressed air) is transported underwater to the diver via a flexible umbilical hose. Surface-supplied equipment provides the diver with a nearly unlimited supply of breathing gas, and also provides a safety tether line and hard-wire communications system connecting the diver and above water personnel. Using surface-supplied equipment, work may be safely completed under adverse conditions that often accompany underwater bridge inspections, such as: fast current, cold and/or contaminated water, physically confined space, submerged drift and debris, and dives requiring heavy physical exertion or of relatively long duration (see Figure 11.3.5).



Figure 11.3.5 Surface-Supplied Diving Inspection

Method Selection Criteria

In determining whether a bridge can be inspected by wading or whether it requires the use of diving equipment, water depth should not be the sole criteria. Many factors combine to influence the proper underwater inspection method:

- Water depth
- Water visibility
- Current velocity
- Streambed conditions (softness, mud, "quick" conditions, and slippery rocks)
- Debris
- Substructure configuration

11.3.3 Diving Inspection Intensity Levels

Originating in the United States Navy and offshore diving industry, the designation of standard levels of inspection has gained acceptance. Three diving inspection intensity levels have evolved as follows:

- Level I: Visual, tactile inspection
- Level II: Detailed inspection with partial cleaning
- Level III: Highly detailed inspection with nondestructive testing

Level I

Level I inspection consists of a "swim-by" overview at arm's length with minimal cleaning to remove marine growth. Although the Level I inspection is referred to as a "swim-by" inspection, it must be detailed enough to detect obvious major damage or deterioration. A Level I inspection is normally conducted over the total (100%) exterior surface of each underwater element, involving a visual and tactile inspection with limited probing of the substructure and adjacent streambed. The results of the Level I inspection provide a general overview of the substructure condition and verification of the as-built drawings. The Level I inspection can also indicate the need for Level II or Level III inspections and aid in determining the extent and selecting the location of more detailed inspections.

Level II

Level II inspection is a detailed inspection that requires that portions of the structure be cleaned of marine growth. It is intended to detect and identify damaged and deteriorated areas that may be hidden by surface growth. A Level II inspection is typically performed on at least 10% of all underwater elements. In some cases, cleaning is time consuming and should be restricted to critical areas of the structure. The thoroughness of cleaning should be governed by what is necessary to determine the condition of the underlying material. Removal of all growth is generally not needed. Generally, the critical areas are near the low waterline, near the mud line, and midway between the low waterline and the mud line. On pile structures, horizontal bands, approximately 150 to 300 mm (6 to 12 inches) in height, should be cleaned at designated locations:

- Rectangular piles - the cleaning should include at least three sides
- Octagonal piles - at least six sides
- Round piles - at least three-fourths of the perimeter
- H-piles - at least the outside faces of the flanges and one side of the web

On large elements, such as piers and abutments, areas at least 0.09 m^2 (1 square foot) in size should be cleaned at three or more levels on each face of the element (see Figure 11.3.6). Deficient areas should be measured, and the extent and severity of the damage documented.



Figure 11.3.6 Diver Cleaning Pier Face For Inspection

Level III

A Level III inspection is a highly detailed inspection of a critical structure or structural element, or a member where extensive repair or possible replacement is contemplated. The purpose of this type of inspection is to detect hidden or interior damage and loss in cross-sectional area. This level of inspection includes extensive cleaning, detailed measurements, and selected nondestructive and partially destructive testing techniques such as ultrasonics, sample coring or boring, physical material sampling, and in-situ hardness testing. The use of testing techniques is generally limited to key structural areas, areas that are suspect, or areas that may be representative of the entire bridge element in question.

11.3.4 Types of Inspection

A comprehensive review must be made of all bridges contained in an agency's inventory to determine which bridges require underwater inspection. Many combinations of waterway conditions and bridge substructures exist. For any given bridge, the combination of environmental conditions and structure configuration can significantly affect the requirements of the inspection. It is generally accepted that there are five different types of inspections:

- Inventory
- Routine
- Damage
- In-depth
- Interim

Underwater inspections are typically either routine or in-depth inspections.

Inventory Inspections

An inventory inspection is the first inspection of a bridge as it becomes a part of the bridge inventory. An inventory inspection may also apply when there has been a change in the configuration of the structure such as widening, lengthening, or even bridge replacement (see Figure 11.3.7). The inventory inspection is a fully documented investigation, and it must be accompanied by an analytical determination of load capacity, which includes scour analyses if appropriate.



Figure 11.3.7 Bascule Bridge on the Saint Croix River

There are two purposes for an inventory inspection:

- Collection of Structure Inventory and Appraisal (SI&A) data
- Establish as-built conditions

The second important aspect of the inventory inspection is the determination of baseline structural conditions and the identification and listing of existing problems or locations in the structure that may have potential problems.

Aided by a prior detailed review of plans, it is during this inspection that any underwater members (or details) are noted for subsequent focus and special attention.

Routine Inspections

A routine inspection is a regularly scheduled, intermediate level inspection consisting of sufficient observations and measurements to determine the physical and functional condition of the bridge, to identify any change from "inventory" or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.

The routine inspection must fully satisfy the requirements of the NBIS with respect to maximum inspection frequency, updating of SI&A data, and the qualifications of the inspection personnel.

Routine inspections of substructures in water must be conducted at least once every five years. Structures having underwater members which are partially deteriorated or which are in unstable channels may require shorter inspection intervals.

The scope of work for a routine inspection should include:

- A Level I inspection should be made on 100% of the underwater portion of the structure to determine obvious problems.
- A Level II inspection should be made on at least 10% of underwater units selected as determined by the Level I inspection.
- A Level III inspection may need to be performed to gain additional data so that the structural conditions can be evaluated with certainty.

The dive team should also conduct a scour evaluation at the bridge site, including:

- Inspect the channel bottom and sides for scour.
- Cross sections of the channel bottom should be taken and compared with as-built plans or previously taken cross sections to detect lateral channel movement or deepening (see Figure 11.3.8).
- Soundings should be made in a grid pattern (see Figure 11.3.9) about each pier and upstream and downstream of the bridge, developing contour elevations of channel bottom, to detect areas of scour. Permanent reference point markers should be placed on each abutment/pier (see Figure 11.3.10). Data obtained from the soundings should be correlated with the original plans (if available) of the bridge foundations and tied to these markers for reference during future underwater inspections.
- Local scour should be determined with probes in the vicinity of piers and abutments (see Figure 11.3.11). In streams carrying large amounts of sediment, reliable scour depth measurements may be difficult at low flow due to scour hole backfilling.

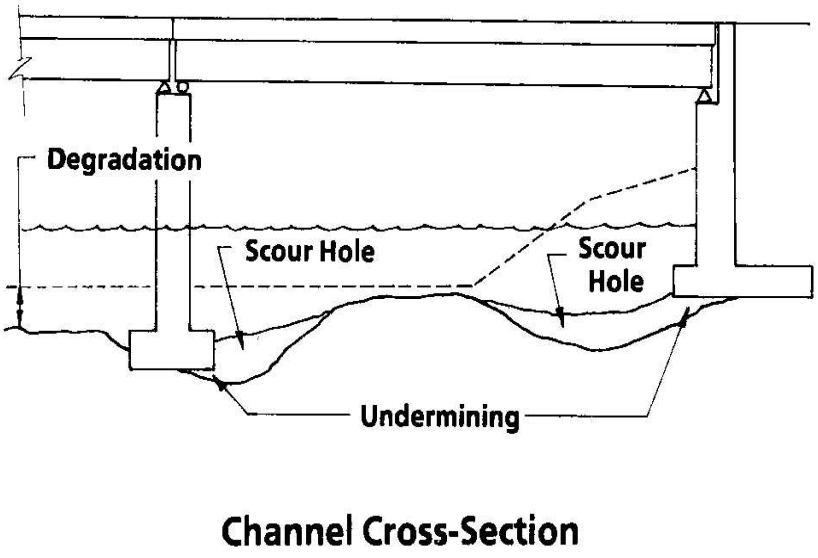


Figure 11.3.8 Channel Cross Section

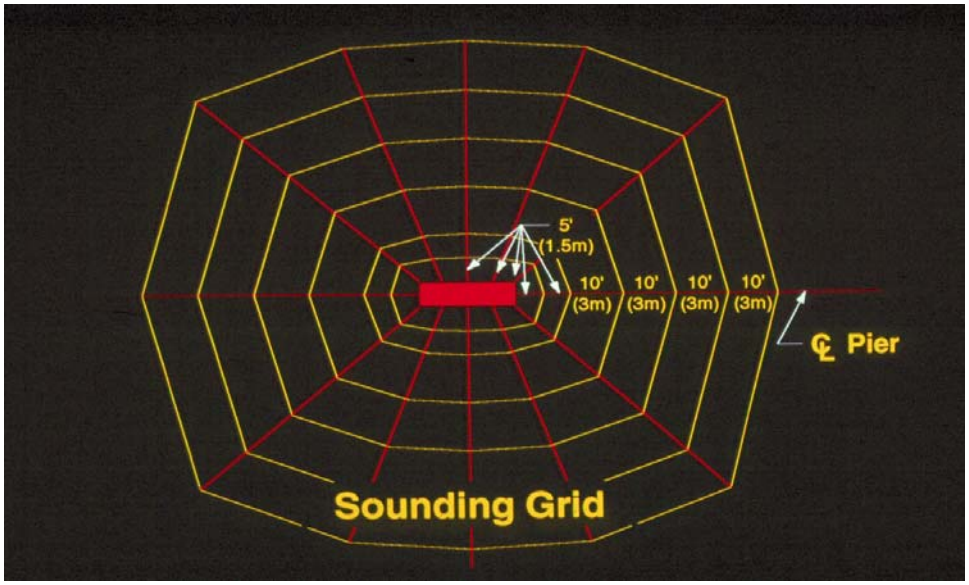


Figure 11.3.9 Pier Sounding Grid



Figure 11.3.10 Permanent Reference Point (Bolt Anchored in Nose of the Pier, Painted Orange)



Figure 11.3.11 Local Scour; Causing Undermining of a Pier

Damage Inspections

Certain conditions and events affecting a bridge may require more frequent, or unscheduled, inspections to assess structural damage resulting from environmental or accident related causes.

The scope of inspection must be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic and to assess the level of effort necessary to repair the damage. The amount of effort expended on this type of inspection will vary significantly depending upon the extent of the damage. If major damage has occurred, inspectors must evaluate section loss, make measurements for misalignment of members, and check for any loss of foundation support.

Situations that may warrant a damage inspection include:

- Floods - bridge elements located in streams, rivers, and other waterways with known or suspected scour potential should be inspected after every major runoff event to the extent necessary to ensure bridge foundation integrity (see Figure 11.3.12).
- Vessel impact - bridges should be inspected underwater if there is visible damage above water; this should be done in order to determine the extent of damage and to establish the extent of liability of the vessel owner for damages.
- Ice floes - ice floes can damage substructure elements, and accumulations of ice on the elements can cause scouring currents or increase the depth of scour.
- Prop wash from vessels - prop wash (i.e., turbulence caused by the propellers of marine vessels) can cause scouring currents and may propel coarse-grained bottom materials against substructure elements in a manner similar to that of blast cleaning operations.
- Buildup of debris at piers or abutments - this material buildup effectively widens the unit and may cause scouring currents or increase the depth of scour (see Figure 11.3.13).
- Evidence of deterioration or movement - many underwater deficiencies only become apparent above water when the distress extends above the waterline or is manifested by lateral movement or settlement. Bridges should also by inspected underwater following significant earthquakes (see Figure 11.3.14).



Figure 11.3.12 Flood Conditions. Note pier settlement.



Figure 11.3.13 Buildup of Debris At Pier



Figure 11.3.14 Movement of a Substructure Unit

In-Depth Inspections

An in-depth inspection is a close-up, hands-on inspection of one or more members below the water level to detect any deficiencies not readily apparent using routine inspection procedures. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, nondestructive tests may need to be performed.

The inspection may include a load rating to assess the residual capacity of the member or members, depending on the extent of the deterioration or damage.

One or more of the following conditions may dictate the need for an in-depth inspection:

- Inconclusive results from a routine inspection
- Suspect hidden or internal damage
- Critical structures whose loss would have significant impact on life or property
- Unique structures whose structural performance is uncertain
- Prior evidence of distress
- Consideration of reuse of an existing substructure to support a new superstructure or planned major rehabilitation of the superstructure
- Adverse environmental conditions such as brackish and polluted water

The in-depth inspection typically includes Level II inspection over extensive areas and Level III inspection of limited areas. Nondestructive testing is normally performed, and the inspection may include partially destructive testing methods, such as extracting samples for laboratory analysis and testing, boring, and probing.

All findings should be recorded using notes and sketches. Underwater photographs and video recordings should also be used where visibility permits.

The distinction between routine and in-depth inspections is not always clearly defined. For some bridges, such as steel pile supported structures in an actively corrosive environment, it may be necessary to include Level III, nondestructive

testing inspection techniques as part of routine inspections.

Interim Inspections

An interim inspection is an inspection scheduled at the discretion of the individual in charge of bridge inspection activities. An interim inspection is used to monitor a particular known or suspected deficiency (e.g., foundation settlement or scour).

11.3.5 Qualifications of Diver-Inspectors

The National Bridge Inspection Standards (NBIS) do not specify minimum requirements necessary for a diver to be considered fully trained and experienced in the inspection and evaluation of substructure and streambed conditions.

The underwater inspector must have knowledge and experience in bridge inspection. All underwater inspections should be conducted under the direct supervision of a qualified bridge inspection team leader. A diver not fully qualified as a bridge inspection team leader must be used under close supervision.

As the ability of the underwater inspector to safely access and remain at the underwater work site is paramount to a quality inspection, the individual must possess a combination of commercial diving training and experience, which demonstrates his/her competence as a working diver.

Federal Commercial Diving Regulations

Underwater bridge inspection, using either self-contained or surface-supplied equipment, is a form of commercial diving. In the United Sates, commercial diving operations are federally regulated by both the Occupational Safety and Health Administration (OSHA), and the U.S. Coast Guard. OSHA regulates all commercial diving operations performed inland and on the coast (through 29 CFR Part 1910, Subpart T-Commercial Diving Operations). This reference should be consulted for details on commercial diving procedures and safety.

Diver Training and Certification

OSHA Safety Requirements

The OSHA standard delineates diving personnel requirements, including general qualifications of dive team members. The standard also provides general and specific procedures for diving operations, and provides requirements and procedures for diving equipment and recordkeeping.

ANSI Standards for Commercial Diver Training

American National Standards Institute (ANSI) Standards exist, which define minimum training standards for both recreational SCUBA and commercial divers. These two, separate standards provide clear-cut distinctions between recreational and commercial diver training. While not federal law, these standards constitute the consensus of both the recreational and commercial diving communities, following ANSI's requirements for due process, consensus, and approval.

The American National Standard for Divers- Commercial Diver Training-Minimum Standard (ANSI/ACDE-01-1998) requires a formal course of study, which must contain at least 625 hours of instruction. This training may come from an accredited commercial diving school, military school, or may be an equivalent degree of training achieved prior to the effective date of the Standard, which includes a documented combination of field experience and/or formal classroom instruction.

ADC International Requirements

The Association of Diving Contractors International (ADC) is a non-profit organization representing the commercial diving industry. The ADC publishes "Consensus Standards For Commercial Diving Operations", which have been developed to present the minimum standards for basic commercial diving operations conducted either offshore or inland. The Consensus Standards, in part, duplicate the ANSI standard for commercial diver training, but subdivide the minimum 625 hours of training into both a formal course of study (317 hours, minimum), and on the job training (308 hours, minimum). The ADC also formally issues OSHA-recognized Commercial Diver Certification Cards to individuals meeting minimum training standards. On the world-wide web, go to *www.adc-usa.org* for more information.

Dive Team Requirements

The Federal Highway Administration's main concern is whether the diver has knowledge and experience in underwater bridge inspection. The individual employers are in the best position to determine the specific requirements of their dive teams. Regarding staffing levels, OSHA requires a minimum of three (3) dive team members, whether conducting self-contained or surface supplied diving operations.

11.3.6 Planning an Underwater Inspection

Planning for underwater bridge inspections is particularly important because of:

- The complexity and potential hazards involved in conducting the inspection
- Unknown factors which may be discovered during the diving
- The difficulty for the bridge owner to verify the thoroughness of the inspection
- The cost of conducting underwater inspections

These factors are most influential for first-time (inventory) underwater inspections that set a benchmark for future inspections. It is, therefore, important to distinguish between the first-time and follow up inspections.

The effectiveness of an underwater inspection depends on the agency's ability to properly consider all factors:

- Method of underwater inspection (i.e., Dive mode)
- Diving inspection intensity level
- Type of inspection
- Qualifications of diver-inspectors
- Specific bridge site conditions, including access requirements, and waterway and climate conditions

With these factors considered, an agency may opt for a lower level of inspection. Depending on conditions and the type of damage found, a higher level may then be necessary to determine the actual bridge condition. It is also possible that different levels may be required at various locations on the same bridge.

11.3.7 Substructure Units and Elements

The underwater portions of bridge structures can be classified into five broad categories: bents, piers, abutments, culverts, and protection systems. Proper identification is important since various elements may require different inspection procedures, levels of inspection, or inspection tools.

Bents

The majority of substructure units requiring underwater inspection are piers and bents. We can divide bents into two groups:

- Pile bents
- Column bents

Pile Bents

Pile bents carry the superstructure loads through a pile cap directly to the underlying soil or rock. The piles (and pile cap) can be constructed of timber, steel, or concrete. Pile bents are generally distinguished from piers by the presence of some battered piles and also bracing which provides stability for the individual piles. See Figures 11.3.15 through 11.3.17 for photographs of pile bents of different material types.



Figure 11.3.15 Timber Pile Bent



Figure 11.3.16 Steel Pile Bent



Figure 11.3.17 Concrete Pile Bent

Important items to be noted by the inspector are collision damage, corrosion, and section loss. Scour of the river bottom material at the bottom of the piles can result in instability of the piles. The underwater inspector must compare present scour and resultant pile length with that observed in previous inspections.

Column Bents

Column bents have two or more columns supporting the superstructure and may in turn be supported by piling below the mud line. The column bents are typically constructed of concrete, but the piling may be timber, concrete, or steel.

Piers

Piers carry superstructure loads from the pier cap to the footing, which may be a spread footing or may be supported on piles. Piers can be constructed of steel, timber, concrete, or masonry and are usually distinguished by two to four large columns or a single large shaft. As with pile bents, collision damage, deterioration, and scour are important items to look for in an underwater inspection. It is also important for the inspector to note if the pier shaft or columns are plumb. There are three common types of piers the inspector is likely to encounter:

- Column pier with solid web wall (see Figure 11.3.18)
- Cantilever or hammerhead pier (see Figure 11.3.19)
- Solid shaft pier (see Figure 11.3.20)



Figure 11.3.18 Column Pier with Solid Web Wall



Figure 11.3.19 Cantilever or Hammerhead Pier

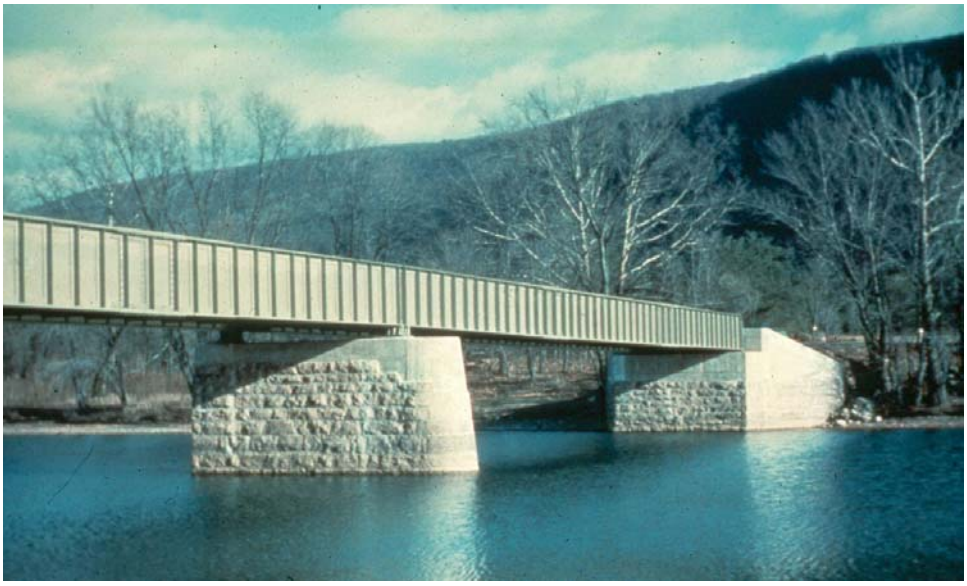


Figure 11.3.20 Solid Shaft Pier

Abutments

Abutments carry the superstructure loads to the underlying soil or rock and also retain the earth at the end of the structure. In most cases, the abutments are dry during low water periods and do not require a diving inspection. However, occasionally the abutments remain continually submerged and must be inspected underwater. Abutments can be constructed from concrete, masonry, or timber and may be supported by spread footings, piles, caissons, or pedestals.

Scour is probably the most critical item to be aware of when performing an underwater abutment inspection. Extreme local scour (undermining) could result in a forward tilting or rotation of the abutment, especially on those abutments without pile foundations (see Figure 11.3.21).



Figure 11.3.21 Severe Flood-Induced Abutment Erosion

Culverts

The underwater inspection of culvert structures present unique challenges to the inspection team, as culverts exist in a wide range of sizes, shapes, lengths, materials, and environments. Areas of special concern to the dive team when conducting culvert inspections include confined space, submerged drift and debris, and animal occupation.

Physically confined space issues arise when inspecting culverts containing individual pipes, barrels, or cells with small interior dimension, or non-linear layout. Additionally, many culverts are continually either completely submerged, or exhibit limited freeboard. In northern environments, winter inspections may also include ice as a contributing factor (see Figure 11.3.22). Diving operations in physically confined space must be conducted in compliance with Federal commercial diving regulations, as well as the individual agency's Safe Practices Manual. The ADC "Consensus Standards For Commercial Diving Operations" also offers guidance for the safe conduct of confined space diving operations.

Submerged drift and debris is a persistent threat to the underwater inspection team, combining with the physically confining nature of most culvert structures to greatly increase the threat of diver entanglement. The diver may be completely unaware of the presence of drift until fouled. Surface-supplied air diving equipment should be used when conducting diving operations in physically

confined and/or debris-laden culverts.

Another threat to the diver involves animals living or seeking shelter inside the culvert. Snakes are often found in and around accumulations of sediment and drift, while, in the southeast United States, alligators often reside inside culvert structures. Those structures exhibiting debris accumulations, which partially or fully constrict one end of a culvert, should be approached with caution, as excited animals may try to leave the culvert in haste, while the inspector is entering.



Figure 11.3.22 Inspection of Culvert With Limited Freeboard and Ice Cover

Protection Systems

Dolphins and fenders are often placed around substructure units to protect them from impact damage. Since these systems are usually at least partially underwater, a diving inspection should be conducted in concert with the substructure unit inspection. Additional protection systems and scour countermeasures include spur dikes, streambed armoring, rip rap, wing dams, and check dams (see Figure 11.3.23).



Figure 11.3.23 Damaged Protective System

11.3.8 Scour Investigations

Divers may be able to note scour degradation under certain conditions. The most important assessment is how much of the bent or pier is exposed when compared to plans and typical designs.

Local scour is often detectable by divers since this type of scour is characterized by holes near bents, piers, or abutments. Divers should routinely check for such scour holes. A typical approach is to take depth measurements around the substructure, both directly adjacent and at concentric intervals. It should also be noted that divers typically operate in low current situations. Sediment often refills scour holes during these periods, making detection of even local scour difficult. However, since this refilled sediment is usually soft, a diver using a probing rod can often detect the soft areas indicating scour refilling.

Depth measurements will not directly reveal the more general scour of significant sections of the streambed. However, the diver may find evidence of such scour from examination of the structure if parts of the substructure are exposed, or by comparing successive cross sections.

The diver's role is primarily to point out a potential scour problem. Almost invariably, an additional interdisciplinary engineering investigation will be needed. The diver's primary role in scour investigation is to measure scour by one of two methods:

- Sounding devices
- Diver investigations

Sounding Devices

Although sounding-sensing devices can be used independently of diving, they are commonly part of an underwater inspection. With the exception of poles and lead lines, sounding-sensing devices depend on some type of signaling system. While these systems are quite effective, they can be misinterpreted. An on-site diver can investigate questionable readings and more fully determine the bottom conditions.

Black and White Fathometer

The most commonly used device is the black and white fathometer. A transducer floats just below the waterline and bounces sound waves off the bottom. Depths are continuously recorded on a strip chart.

Advantages of the black and white fathometer include the following:

- Inexpensive
- Effective
- "User-Friendly" output

Disadvantages include the following:

- False readings can occasionally occur due to heavy drift or heavy turbulence
- The strip chart moves at a constant rate and does not record a horizontal scale; unless the boat can be kept at a constant speed, the scale becomes distorted
- Fathometers may also fail to detect refilled scour holes during calm water

Dual Frequency and Color Fathometer

Dual frequency and color fathometers can be used to detect refilling, since more than one frequency is utilized. With color fathometers, materials of different densities are displayed as different colors. The primary drawback is that a hard copy cannot be obtained except with videotape recordings.

Fathometer/Theodolite

The horizontal scale problem can be solved by using equipment, which combines a fathometer with a total station theodolite. The theodolite is set up on shore, it tracks and records the coordinates of the transducer, and it automatically records depths at specified increments using a microprocessor. The data can be processed and plotted as a topographic map.

Ground-Penetrating Radar

Ground-penetrating radar and tuned transducer (low frequency sonar) equipment are also used in scour surveys. These are good in shallow water but not very effective in salty, brackish water.

Fixed Instrumentation

An alternative to the sounding and scour sensing devices used during inspections is to permanently install fixed instrumentation directly on the bridge substructure. With fixed instrumentation, local scour is continuously monitored and recorded as it occurs, unaffected by washing back of silts and sands, and making information readily available to the bridge owner by setting off a beacon-type alarm on the bridge deck (or relayed back to an office). One such instrument consists of a steel rod inside of a conduit attached to the substructure unit. The rod acts as a probe, resting on the vulnerable soil supporting the substructure. As local scour occurs the soil is washed away and the rod drops a measured distance.

Other fixed instrumentation includes fixed sonar units, sliding magnetic collars, and buried "float-out" buoys, which float to the water surface after being uncovered by local scour, activating an electronic alarm system.

Researchers are studying a new method for scour detection and monitoring. The new method is based on time domain reflectometer (TDR) technology, which uses pulse transmissions to show changes in a particular environment. The TDR bridge scour monitoring system consists of a probe, which is completely buried in the sediment at appropriate locations around and near the bridge pier and footings. As erosion occurs, part of the probe is exposed to water. Then, the probe reflects a specific pulse back to the TDR box, which is on the surface, indicating how much of the probe is exposed and producing wave forms to show scour depth. The probes are designed to be left at bridge sites to detect/monitor scour.

Diver Investigations

Diver investigations include:

- Laying out a grid pattern and taking depth measurements
- Sampling soils to determine backfilling of scour holes
- Probing to check for refilling
- Detecting undermining and scour holes (see Figure 11.3.24)
- Detecting small diameter but deep scour holes around piles
- Protective system evaluation (e.g., rip rap)

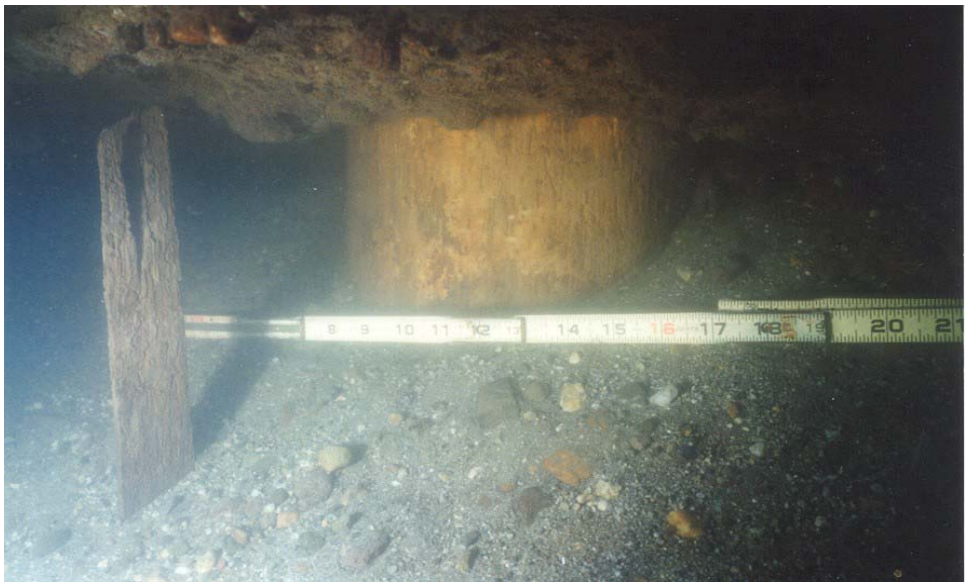


Figure 11.3.24 Pier Undermining, Exposing Timber Foundation Pile

11.3.9 Underwater Inspection for Material Defects

The materials typically used in bridge substructures are concrete, timber, steel, and masonry. An estimated 75% of all underwater elements are concrete. The balance consists of timber, steel, and masonry, in descending order of use.

Concrete and Masonry

Plain, reinforced, and prestressed concrete are used in underwater elements. Since the majority of substructures are basically compression units, concrete is a nearly ideal material choice. Some concrete damage tends to be surface damage that does not jeopardize the integrity of the system. However, concrete deterioration that involves corrosion of the reinforcement can be very serious (see Figure 11.3.25).

Typical defects include:

- Cracking
- Spalls and Delaminations
- Exposed reinforcing - Section loss
- Concrete laitance - a weak surface layer consisting only of cement paste and fine aggregates
- Sulfate attack
- Honeycombing
- Rust spots
- Grout loss
- Scaling



Figure 11.3.25 Concrete Deterioration

Timber Piles

Piles and protection systems often utilize timber. Timber pile bents are typical for short span bridges in many parts of the country, particularly for older bridges. The primary cause of timber deterioration is biological organisms, such as fungi, insects, bacteria, and marine borers. The ingredients for an attack include suitable food, water, air, and a favorable temperature. The waterline of pile structures offers all of these ingredients during at least part of the year. Since water, oxygen, and temperature generally cannot be controlled in a marine environment, the primary means to prevent a biological attack is to deny the food source through treatment to poison the wood as a food source. Timber piles are particularly vulnerable if the treatment leaches out (which happens with age) or if the core is penetrated. It is, therefore, important to carefully inspect in the vicinity of connectors, holes, or other surface blemishes (see Figure 11.3.26).



Figure 11.3.26 Deteriorated Timber Piling

Piles used in older bridges quite often were not treated if the piles were to be buried below the mud line (eliminating the source of food and oxygen). However, in some cases, streambed scour may have exposed these piles. Special care should be taken in differentiating between treated and untreated piles to ensure a thorough inspection of any exposed, untreated piles. With each inspection, the diameter or circumference should be noted for each timber pile. As a minimum, these measurements should be made at the waterline and mud line. Comparisons should be made with the original pile size.

Another primary caution for inspecting underwater timber piles is that the damage is frequently internal. Whether from fungal decay or borers, timber piles may appear sound on the outside shell but be completely hollow inside. While some sources recommend hammer soundings to detect internal damage, this method is unreliable in the underwater environment. One way to inspect for such damage is to take core samples. All bore holes should be plugged. Ultrasonic techniques for timber piling are also available.

Steel

Underwater steel structures are highly sensitive to corrosion, particularly in the low to high water zone (see Figure 11.3.27). Whenever possible, steel should be measured to determine if section loss has occurred. Ultrasonic devices are particularly useful to determine steel thicknesses.



Figure 11.3.27 Deteriorated Steel Sheet-Piling

If submerged steel elements are partially encased in concrete, the exposed steel adjacent to the encasement is particularly susceptible to aggressive corrosion.

Previous Repairs

The inspector must also be alert to note deterioration of previous member repairs or rehabilitation. The first step in the inspection of previous repairs is to review all existing bridge substructure plans prior to the actual inspection. Repair areas should be noted as important areas of inspection. Typical previous repairs might be:

- Steel cover plates
- Concrete fill repairs
- Epoxy crack repairs
- Concrete encasement or jacketing
- Limited replacement of members
- Masonry stone replacement
- Underpinning and rip rap to repair scour

Hands-on Inspection of Material Underwater

When visibility permits, the diver should visually observe all exposed surfaces of the substructure. Scraping over the surface with a sharp-tipped probe, such as a knife or ice pick, is particularly useful for detecting small cracks. With limited visibility, the diver should "feel" for damage. Because orientation and location

are often difficult to maintain, the diver should be systematic in the inspection. Regular patterns should be established from well-defined reference points.

Typical inspection patterns include:

- Circular or semicircular horizontal sweeps around piers or abutments beginning at the base, moving upward a specified increment, and repeating until complete
- Probing zones of undermining of piers by moving uniform increments from start to finish and recording the undermined penetration
- Down one side and up the other for piles (or inspecting in a spiral pattern)
- For scour surveys, record depths at regular increments adjacent to substructure (e.g., at each pile or 10 foot increments around piers), and then at each measured point extend radially from the substructure a uniform distance and repeat depth measurements

Major advantages of diver-to-surface communications are that the diver can be guided from the surface with available drawings, and that immediate recording of observations can be made topside along with the clarification of any discrepancies with plans.

Measuring Damage

Any damage encountered should be measured in detail. As a minimum for a Level II or III inspection include:

- Location of the damage zone both horizontally and vertically from a fixed reference point
- A good vertical reference point is the waterline, provided that the waterline is measured with respect to a fixed reference point on the bridge prior to the dive
- Locate the beginning and ends of cracks and intermediate points as needed to define the pattern
- Measure the maximum crack width and penetration depth
- Measure the length, width, and penetration of spalls or voids, making note of exposure and condition of any reinforcing steel
- Measure the thicknesses of all four flange tips on steel H-piles at distressed areas, and specify the vertical location
- Locate buckles, bulges, and significant loss of section in steel members - thickness of remaining sound material should be accurately measured when significant section loss is found
- For undermining of foundations, take enough measurements to define the zone no longer providing soil bearing
- If plans are not available, measure the basic dimensions of damaged members (it is also usually prudent to spot check dimensions of damaged members even if plans are available)
- Measure the diameter of timber piles – note extent and width of checks, and extent of any rot, if found
- Note the degree of scaling on concrete
- Check for displacements of major elements and whether they are plumb
- Note damage at connections

Recordkeeping and Documentation

Because of the effort spent in conducting underwater inspections, combined with the time between inspections, it is particularly important to carefully document the findings. On-site recording of all conditions is essential:

- Sketches - it is recommended that sketches be used as much as possible; providing enough detail is critical since it is difficult to go back to check items once the diving is completed. Contour and plan view sketches of the area surrounding the substructure elements allow the inspector to track any scour or streambed movement. A profile of the streambed can also provide information for tracking the development of scour.
- Notes/Logs - in addition to sketches, written notes or logs should be kept, documenting the inspection.
- Tape recordings - when significant damage is encountered, a tape recording of the diver's observations can also prove helpful.
- Underwater photographs
- Underwater videotapes

The results should also be included in a report. Drawings and text should describe all aspects of the inspection and any damage found. The report should also include recommendations on condition assessment, repairs, and time interval for the next inspection. A sample underwater inspection report can be found in Figure 11.3.28.

CONDENSED UNDERWATER BRIDGE INSPECTION REPORT

BRIDGE NUMBER	COUNTY NAME	ROAD NUMBER	ROAD NAME	DATE INSPECTED

BODY OF WATER: _____ **DIVE MODE:**

DIVING CONDITIONS

MAXIMUM CURRENT:

AIR TEMPERATURE:

AVERAGE VISIBILITY:

WATER TEMPERATURE:

BOTTOM MATERIAL:

MAXIMUM DEPTH: _____

ITEMS INSPECTED:

_____

ITEM OF INSPECTION	NCR**	REMARKS
1. PILING/SHAFTS		
2. FOOTINGS/CAISSONS/PEDESTALS		
3. COLUMNS/WALL PIERS		
4. BRACING/STRUTS/WEB WALLS		
5. ABUTMENTS/END BENTS		
6. RETAINING WALLS/WING WALLS		
7. FENDER SYSTEM/PIER PROTECTION		
8. EMBANKMENTS/SLOPES/BULKHEADS		
9. DEGRADATION/AGGRADATION		
10. OBSTRUCTION/FLOW		
11. MOVABLE BRIDGE PIERS (PIVOT, BASCULE, REST)		
12. CULVERT BARRELS		
13. CULVERT HEADWALLS		
14. SUBMARINE CABLE (S) ***		

* Deficiencies exist in this element that warrant written and/or sketched description which are provided in the "Comprehensive Report of Deficiencies" section of this report.
** NCR is an acronym for numerical condition rating, the definitions of which can be found on the back of this page.
*** Submarine Cables(s) rated using Non-Structural Features rating system [1 (Poor) to 4 (Good) or N]

INSPECTION PARTY

Name: Name:
Name: Name:

Figure 11.3.28 Sample Underwater Inspection Report

NUMERICAL CONDITION RATING DEFINITIONS FOR STRUCTURAL ITEMS

CODE	DESCRIPTION
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION-No problems noted.
7	GOOD CONDITION-Some minor problems. Minor maintenance may be needed.
6	SATISFACTORY CONDITION-Structural elements show some minor deterioration. Major maintenance is needed.
5	FAIR CONDITION-All primary structural elements are sound but may have minor section loss, cracking, spalling. Minor rehabilitation may be needed.
4	POOR CONDITION-Advanced section loss, deterioration, spalling. Major rehabilitation may be needed.
3	SERIOUS CONDITION-Loss of section, deterioration, spalling have seriously affected primary structural elements. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present. Repair or rehabilitation required immediately.
2	CRITICAL CONDITION-Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	AIMMINENT@ FAILURE CONDITION-Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION-Out of Service-beyond corrective action

NUMERICAL CONDITION RATING DEFINITIONS FOR DEGRADATION/AGGRADATION

CODE	DESCRIPTION
N	NOT APPLICABLE-Use when bridge is not over a waterway.
9	EXCELLENT CONDITION-No noticeable or noteworthy deficiencies, which affect the condition of the channel.
8	VERY GOOD CONDITION-Banks are protected or well vegetated. River control devices, such as spur dikes and embankment protection, are not required or are in stable condition. Some minor scour has occurred near bridge.
7	GOOD CONDITION-Bank protection is in need of minor repairs. River control devices and embankment protection have minor damage. There is minor streambed movement evident. Minor local scour developing near substructure.
6	SATISFACTORY CONDITION-Bank is beginning to slump. River control devices and embankment protection have considerable minor damage. There is minor streambed movement evident. Debris is restricting the waterway slightly. Scour holes deepening.
5	FAIR CONDITION-Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel. Scour holes are becoming more prominent, affecting the stability of the substructure.
4	POOR CONDITION-Bank and embankment protection undermined with corrective action required. River control devices have severe damage. Large deposits of debris in the waterway. The streambed has changed its location but is causing no problem.
3	SERIOUS CONDITION-Bank protection has failed completely. Scour holes forming in embankment. River control devices have been destroyed. Streambed aggradation or degradation has changed the waterway to now threaten the bridge and/or approach roadway.
2	CRITICAL CONDITION-Abutment has failed (portion has settled) due to undermining of footing. The waterway has changed and now threatens the bridge and/or embankment. Scour is of sufficient depth beneath footing that substructure is in near state of collapse.
1	AIMMINENT@ FAILURE CONDITION-Bridge closed. Corrective action may put the structure back into light service.
0	FAILED CONDITION-Bridge closed. Replacement necessary.

Figure 11.3.28 Sample Underwater Inspection Report (Continued)

11.3.10 Underwater Inspection Equipment

Diving Equipment

For self-contained diving, the breathing gas supply is contained within a pressurized tank, which is carried by the diver. Personal equipment includes:

- Wetsuit or drysuit (drysuit should be used when diving in water either known or suspected to be contaminated) (see Figure 11.3.29)
- Face mask or helmet (see Figure 11.3.30)
- Breathing apparatus
- Weight belt
- Swim fins
- Knife
- Wristwatch
- Buoyancy compensator (a flotation device capable of maintaining a diver face up at the surface)
- Depth gauge
- Pressure gauge

Surface-supplied air diving equipment typically includes a compressor, which supplies air into a volume tank for storage. This compressed air is then filtered and regulated to the diver's helmet or mask through an umbilical hose (see Figures 11.3.31 and 11.3.32). The umbilical is typically made up of several members, including, at a minimum, an air hose, strength member (or safety line), communication line, and pneumofathometer hose. The pneumofathometer provides diver depth measurements to the surface (see Figure 11.3.33). A reserve air tank, or bail-out bottle, should also be worn by the diver for emergency use.



Figure 11.3.29 Vulcanized Rubber Dry Suit



Figure 11.3.30 Full Face Lightweight Diving Mask with Communication System



Figure 11.3.31 Surface-Supplied Air Equipment, Including Air Compressor, Volume Tank With Air Filters, and Umbilical Hoses



Figure 11.3.32 Surface-Supplied Diving Equipment Including Helmet, or Hard Hat



Figure 11.3.33 Pneumofathometer Gauge

Equipment malfunction leading to loss of air supply must be a constant concern to the dive team. Even in shallow water, submerged drift and debris adjacent to a bridge can make an emergency ascent an arduous affair, for both the diver and the support team. As such, a reserve air supply should always be worn by the diver (see Figure 11.3.34). Other threats, such as the contamination of the air supply, are also a concern. For example, carbon monoxide poisoning can occur if the air intake of the surface supplied air compressor is located near the exhaust of other motorized equipment.



Figure 11.3.34 Surface-Supplied Diver with a Reserve Air Tank

Surface Communications

While not required in all situations, a two-way communication system linking the diver(s) and topside personnel greatly enhance the underwater inspection. Conventional hardwire (telephone) and wireless systems exist, which can even be used during self-contained diving operations. There are several advantages

provided to the underwater inspection team, through the use of direct two-way communication (see Figure 11.3.35):

- The greatest benefit to the dive team is increased safety in the event of diver entanglement or equipment malfunction.
- Allows the diver to immediately describe observations and location of deficiencies for simultaneous recording by a note taker on the surface.
- The diver can verbally interact with topside inspection personnel to clarify what is being observed, without leaving the suspect area.
- The note taker can follow drawings, verify their validity, note damage on the drawings at the proper location, and track the progress of the diver.
- Surface communication also allows an inspection team leader/engineer at the surface to discuss observations with a diver who is not yet an inspection team leader, to direct attention to specific zones, and to ensure that a satisfactory inspection is completed, according to the type and severity of damage found (see Figure 11.3.36).



Figure 11.3.35 Communication Box System



Figure 11.3.36 Surface Communication With Inspection Team Leader

Access Equipment

While inspection of short-span bridges can often be accessed from shore, many bridges require a boat or barge for access. Typically, an 5.5 m (18-foot) or larger vessel can safely handle the equipment and crew (see Figures 11.3.37 and 11.3.38). Occasionally, access is made from the bridge itself.



Figure 11.3.37 Access Barge and Exit Ladder



Figure 11.3.38 Access From Dive Boat

Tools

A number of inspection tools are available. The dive team should have access to the appropriate tools and equipment as warranted by the type of inspection being conducted.

Hand Tools

While most hand tools can be used underwater, the most useful include rulers, calipers, scrapers, probes (ice picks, dive knifes, and screwdrivers), flashlight, hammers (especially masonry and geologist's hammers), wire brushes, incremental borers, and pry bars (see Figure 11.3.39). These tools are usually tethered to the diver to prevent their loss underwater.



Figure 11.3.39 Diver-Inspector with a Pry Bar

Power Tools

Power tools include grinders, chippers, drills, hammers and saws. While pneumatic tools are sometimes used, hydraulic tools tend to be favored for heavy or extensive work.

Cleaning Tools

Light cleaning can be accomplished with scrapers and wire brushes. Heavier cleaning requires automated equipment such as grinders and chippers. One of the most effective means of cleaning is with the use of water blasters (see Figure 11.3.40). Particular care must be taken with such equipment to ensure that structural damage does not result from overzealous blasting.



Figure 11.3.40 Cleaning with a Water Blaster

Nondestructive Evaluation Equipment

- Ultrasonic measuring devices measure the thickness of steel by passing a sound wave through the member. The transducer is placed on one side only, and the thickness is displayed on an LED readout. Totally submersible or surface display units are available. They are very effective for measuring thickness.

- A V-meter is an ultrasonic device that requires two transducers and measures the distance required for the sound wave to pass through the concrete. Similar devices have also been developed for timber.

- A waterproof Schmidt hammer can be used underwater to measure concrete compression strength in-place.

- An R-meter is used to locate and measure the depth of cover and the size of reinforcing bars in concrete by inducing a magnetic field.

- Underwater magnetic particle testing equipment, typically consisting of an electromagnetic yoke and powdered metallic particles, are used to detect flaws at or near the surface of ferrous metal members. The articulating yoke is positioned on the member in question, and energized. The powdered metal particles are then sprayed over the specimen, in the area between the legs of the yoke. Discontinuities in the specimen, such as cracks, will cause a magnetic flux leakage field, which will attract the particles. As such, the inspector can readily locate deficiencies that may otherwise remain undetected.

- Parallel Seismic testing can be used to determine pile embedment lengths when as-built plans are not available. The test involves boring a hole in the vicinity of the existing pile or footing and lowering a hydrophone receiver to the bottom. While raising the receiver in small increments, a part of the foundation is struck with an instrumented hammer causing compression or shear waves to travel from the foundation into the surrounding soil. The hydrophone tracks the time it takes for the compression and shear waves to

reach the receiver. By plotting the arrival times and measuring the corresponding depth of the receiver, the pile tip location can be determined. This information is very valuable in determining a bridge's susceptibility to scour. The Parallel Seismic test can be used for steel, concrete, timber and masonry foundations (see Figure 11.3.41).

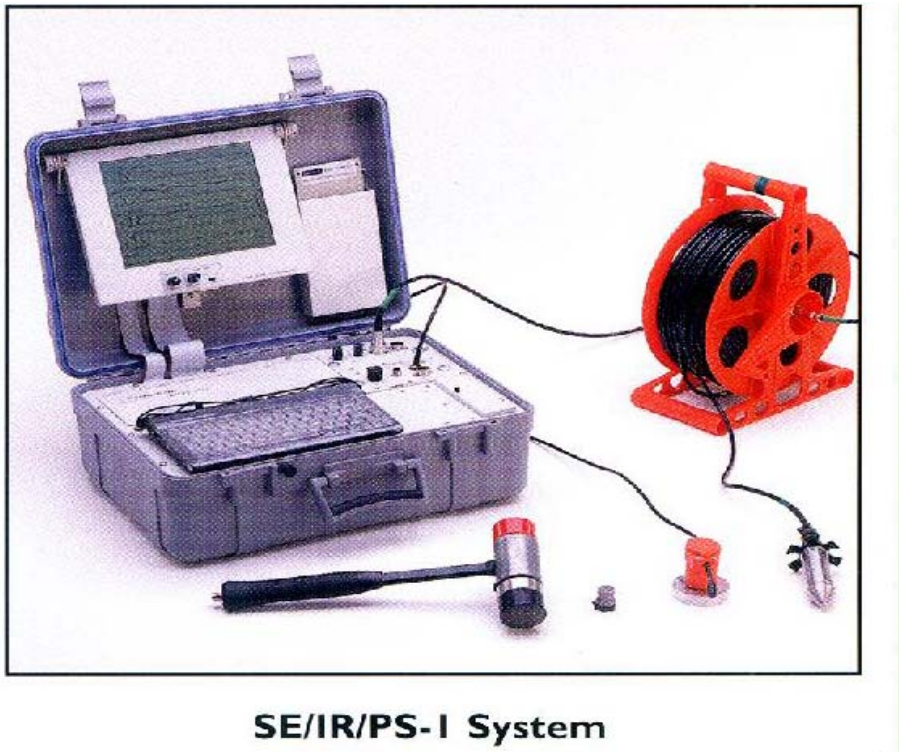


Figure 11.3.41 Parallel Seismic Testing Equipment

Coring Equipment

Coring is a partially destructive evaluation method whose use is usually limited to critical areas. Cores can be taken in either concrete or timber (see Figure 11.3.42).

Concrete coring requires pneumatic or hydraulic equipment. Deep cores (3 feet or more) can be taken to provide an interior assessment of massive substructures (see Figure 11.3.43). Two-inch diameter cores are common, but coring tools are available in other sizes (see Figure 11.3.44). Cores not only provide knowledge about interior concrete consistency but also can be tested to determine compression strength.

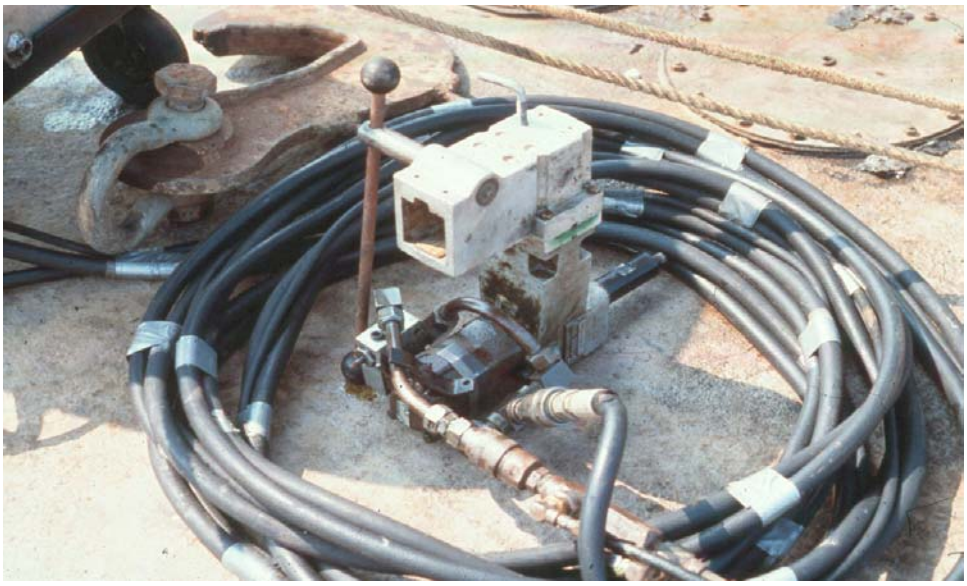


Figure 11.3.42 Coring Equipment



Figure 11.3.43 Concrete Coring Taking Place



Figure 11.3.44 Concrete Core

Timber coring is much simpler and less costly to perform than concrete coring (see Figure 11.3.45). While power tools are sometimes used, the most effective procedure is still to hand core with an increment borer. This approach preserves the core for laboratory as well as field evaluation. Examination of the core should include its compressibility, evidence of borers or other infestation, and indications of void areas. The hole should always be plugged with a treated hardwood dowel to prevent infestation.



Figure 11.3.45 Timber Core

Underwater Photography and Video Equipment

Cameras come with a variety of lens and flash units. In some cases, visibility is limited and the camera must be placed close to the subject. Wide-angle lenses are therefore most often used (see Figure 11.3.46). Suspended particles often dilute the light reaching the subject and can reflect light back into the lens. When visibility is very low, clear water boxes can be used. The boxes are constructed of clear plastic and can be filled with clean water. By placing the box against the subject area, the dirty water is displaced and the camera shot can be taken through the clear water (see Figure 11.3.47).



Figure 11.3.46 Underwater Photographer



Figure 11.3.47 Camera with a Clear Box

Video equipment is available either as self-contained, submersible units or as submersible cameras having cable connection to the surface monitor and controls (see Figure 11.3.48). The latter type allows a surface operator to direct shooting while the diver concentrates on aligning the camera only. The operator can view the monitor, control the lighting and focusing, and communicate with the diver to obtain an optimum image. Since a sound track is linked to the communication equipment, a running commentary can also be obtained.



Figure 11.3.48 Underwater Video Inspection

An extension of the video camera is a remotely operated vehicle (ROV), where the diver is eliminated and the camera is mounted on a surface controlled propulsion system. Its effectiveness diminishes substantially in stream velocities greater than 0.8 m/s (1.5 knots) and is limited by cloudy water, inability to determine the exact orientation and position of the camera, and control sensitivity. Also important to note is that an ROV cannot typically perform cleaning operations prior to photos being taken.

11.3.11 Special Considerations for Underwater Inspections

Once a diver enters the water, their environment changes completely. Visibility decreases and is often reduced to near zero, due to muddy water and depth. In many cases, artificial lighting is required. There are times when tactile (by feel) inspections are all that can be accomplished, significantly compromising the condition evaluation of the element(s) being inspected.

The diver not only has reduced perceptual capabilities but is less mobile as well. Maneuverability is essential for underwater bridge inspections. With either self-contained or surface-supplied equipment, the diver may find it useful to adjust his/her underwater weight to near buoyancy and use swim fins for propulsion.

Dealing with Current

Most waterways have low flow periods when current will not hinder an inspection. Diving inspections should be planned with this consideration in mind. Divers can work in current below 0.8 m/s (1.5 knots) with relatively little hindrance. As current increases, special precautions are required. Bottom anchors, shielding devices, and special anchoring/tethering systems may be required, depending upon the site-specific conditions encountered at the bridge. (see Figure 11.3.49).



Figure 11.3.49 Diving Inside a Cofferdam

Obviously, waterway conditions may sometimes be too swift to allow safe diving operations (see Figure 11.3.50).



Figure 11.3.50 Excessive Current

Dealing with Drift and Debris

The drift and debris that often collects at bridge substructures can be quite extensive (see Figure 11.3.51). This type of buildup typically consists of logs and limbs from trees that are usually matted or woven either against or within the substructure elements. Often this debris is located on the lower parts of the substructure and cannot be detected from the surface. The buildup can be so thick as to prevent access to major portions of the underwater substructure.



Figure 11.3.51 Debris

Hidden Costs

Since they are often hidden, drift and debris problems present the bridge owner with an unknown cost factor. The removal of the drift and debris must be provided for if an inspection of the underwater elements is to proceed. While in some cases it can be removed by the inspection divers, heavy equipment, such as a hoist or underwater cutting devices, are often required.

Past History

Generally, such buildup occurs in repetitive patterns. If previous underwater inspections have been conducted, the presence of drift can be estimated based on past history. Also, certain rivers and regions tend to have a history of drift problems, while others do not. Knowledge of this record can help predict the likelihood of drift. A separate drift removal team, working ahead of the dive inspection team, could possibly be utilized.

Safety

Divers must also have a safety concern about the buildup of debris near a bridge. Occasionally, debris can be quite extensive and can lead to entanglements or sudden shifts which might entrap the diver.

Cleaning

Bridges on many inland waterways are relatively clean and free of marine growth. In such cases, the inspection can be conducted with little extra effort from the diver other than perhaps light scraping.

In coastal waterways, the marine growth can completely obscure the substructure element and may reach several inches or more in thickness (see Figure 11.3.52). The cost of cleaning heavily infested substructures may be completely impractical. In such cases, spot cleaning and inspection may be the only practical alternative.

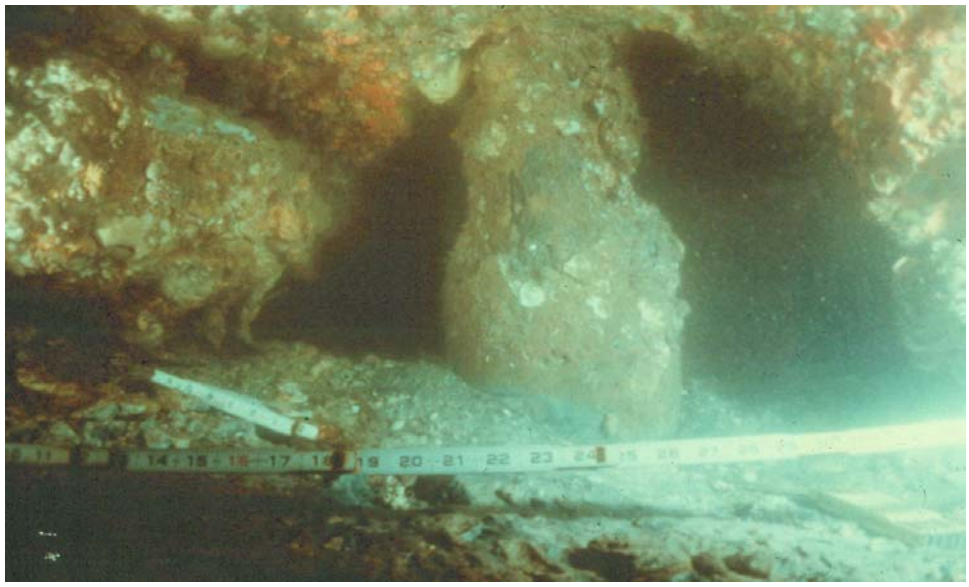


Figure 11.3.52 Cleaning a Timber Pile

Cleaning Locations and Procedures

The best approach is to restrict cleaning to small zones that are:

- Structurally critical areas
- Areas known to frequently deteriorate for that specific structural configuration
- Areas randomly selected to statistically lower the probability of overlooking damage

Typically, 150 to 300 mm (6 to 12 inch) bands or squares are recommended depending on the size and shape of the element. Highest priority should be placed on locations near the low waterline and at connecting elements. If feasible, locations should also be included at the mud line and midway between the mud line and low waterline. On small diameter elements, cleaning can be limited to bands approximately three-fourths around the circumference. For large elements, squares around the perimeter should be chosen at effective intervals.

While minor cleaning can be done with hand tools, it is often more efficient to use hydraulic grinders or high-pressure water blasting equipment.

Physical Limitations

This sometimes cold, dark, hostile environment can result in a reduced physical working capacity. The diver is also totally dependent on external life support systems, which adds psychological stress. Things that can be done intuitively above water must be conscientiously planned and executed step-by-step underwater. For example, maintaining orientation and location during an underwater inspection requires continual attention. Distractions are plentiful and range from living organisms, such as fish, snakes, and crustaceans, to environmental conditions, such as cold, high current, and debris.

Decompression Sickness

Since the majority of bridge inspections are in relatively shallow water and of relatively short duration, decompression problems rarely occur. However, multiple

dives have a cumulative effect and the no-decompression time limit decreases rapidly at depths greater than 50 feet. Therefore, divers should routinely track their time and depth as a safety precaution. OSHA requires that a decompression chamber be on-site and ready for use for any dive made outside the no-decompression limits or deeper than 100 feet of seawater.

Marine Traffic

Another hazard is vessel traffic near the dive area. There should always be someone topside with the responsibility of watching boat traffic (see Figure 11.3.53). In addition, flags should be displayed indicating that a diver is down. The international code flag "A", or “Alpha” flag (white and blue), signifies that a diver is down and to stay clear of the area. OSHA requires this flag. However, it is also prudent to display the sport diver flag (white stripe on red), since it is more likely that recreational boaters will recognize this flag (see Figure 11.3.54).



Figure 11.3.53 Commercial Marine Traffic



Figure 11.3.54 Alpha (lower) and Sport Diver (upper) Flags on Mast. See Arrow

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Section 12
Special Bridges

Topic 12.1 Cable Supported Bridges

12.1.1 Introduction

There are several bridge types which feature special elements and which require special inspection procedures. The most notable special bridge types are:

- Suspension bridges (see Figure 12.1.1)
- Cable-stayed bridges (see Figure 12.1.2)

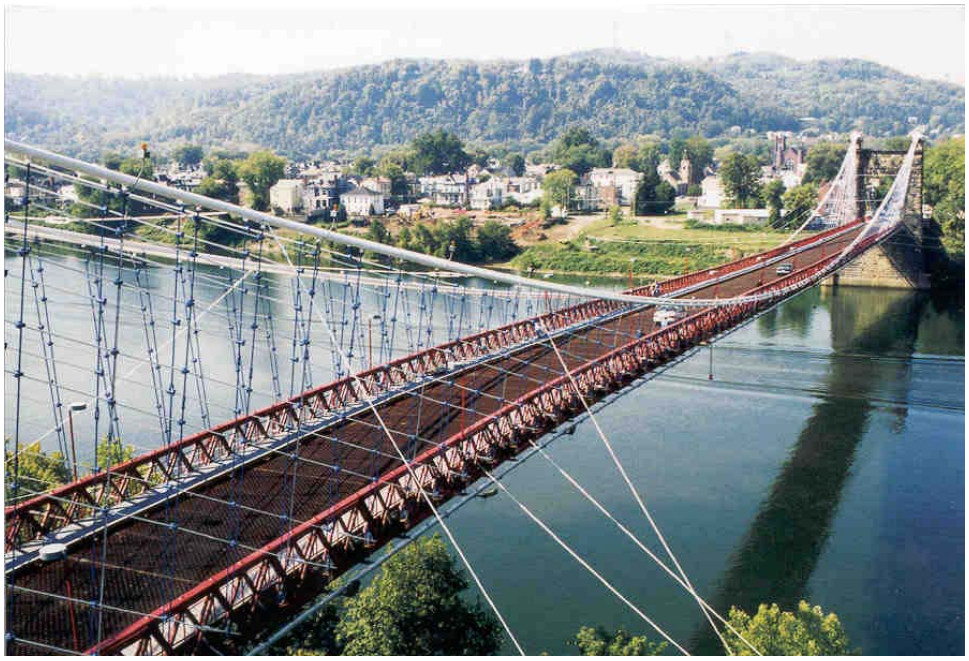


Figure 12.1.1 Wheeling Suspension Bridge – Longest in the World When Completed in 1849 (Photo Courtesy of Geoffrey H. Goldberg, 1999)



Figure 12.1.2 Maysville Cable-Stay Bridge, Built in 2000

This section is limited to the cable and its elements. All other members of a cable-supported bridge have been described in earlier sections and should be referred to for the appropriate information. For each of the above bridge types, this section provides:

- A general description
- Identification of special elements
- An inspection procedure for special elements
- Methods of recordkeeping and documentation

12.1.2 Design Characteristics

A cable-supported bridge is a bridge that is supported by or "suspended from" cables.

Suspension Bridge

A suspension bridge has a deck, which is supported by vertical suspender cables that are in turn supported by main suspension cables. The suspension cables are supported by saddles atop towers and are anchored at their ends. Suspension bridges are normally constructed when intermediate piers are not feasible because of long span requirements. Modern suspension bridge spans are generally longer than 427 m (1400 feet).

Cable-Stayed Bridge

A cable-stayed bridge is another long span cable supported bridge where the superstructure is supported by cables, or stays, passing over or anchored to towers located at the main piers. Cable-stayed bridges are the more modern version of cable-supported brides. Spans generally range from 213 to 427 m (700 to 1400 feet). Evolving for approximately 400 years, the first vehicular cable-stayed bridge in the United States was constructed in Alaska in 1972 (John O'Connell Memorial Bridge at Sitka, Alaska).

Examples

In comparison, cable-stayed bridges are much stiffer than suspension bridges. In cable-stayed bridges, the deck is supported directly from the tower with fairly taut stay cables. In suspension bridges, vertical suspender cables attach the deck to the loosely hung main cables.

- **The Wheeling Suspension Bridge** - also has stay-cables. Suspension bridges normally have an inspection walkway along the main cables. The Wheeling Suspension Bridge in Wheeling, West Virginia was originally built in 1849 (see Figure 12.1.3).

- **Footbridge** - this footbridge shows basic concepts of a suspension bridge, but on a smaller simpler structure (see Figure 12.1.4).

- **Sunshine Skyway Bridge** - multiple towers, concrete superstructure, single cable plane (see Figure 12.1.5)



Figure 12.1.3 Roebling Bridge

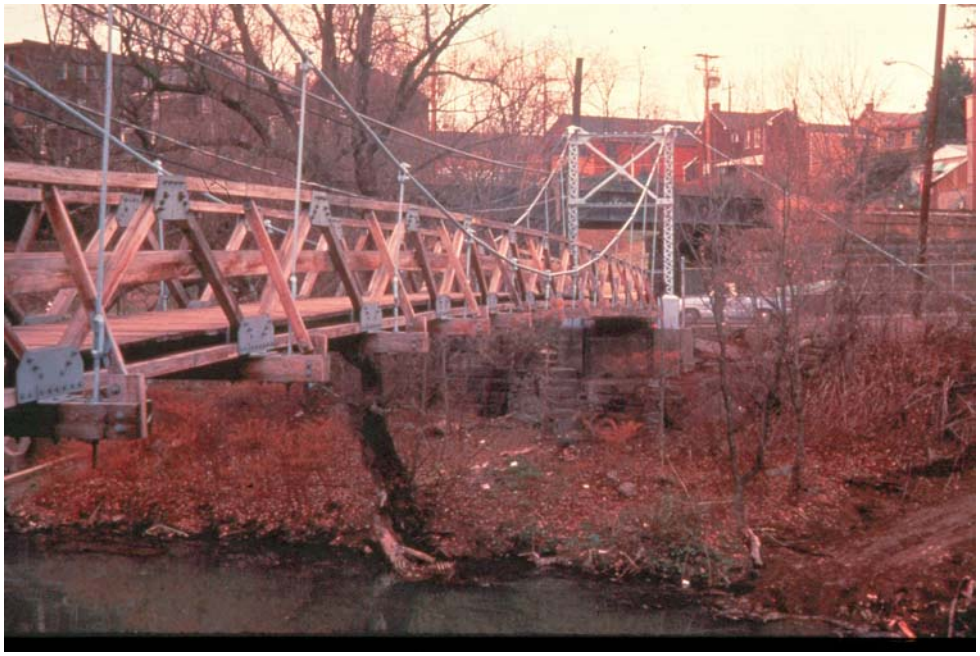


Figure 12.1.4 Footbridge Suspension Bridge



Figure 12.1.5 Sunshine Skyway Cable-Stayed Bridge in Tampa Bay, Florida

12.1.3 Suspension Bridges

In this section, only those bridge elements that are unique to suspension bridges are discussed. Refer to the appropriate section for other suspension bridge elements that are common to most bridges.

Main Suspension Cables and Suspender Cables

Main suspension cables are generally supported on saddles at the towers and are anchored at each end. Suspender cables are vertical cables that connect the deck system to the main cables (see Figure 12.1.6). The main cables are commonly composed of a number of parallel wires banded together and wrapped with a soft wire wrapping. Composition of the suspender cables varies.

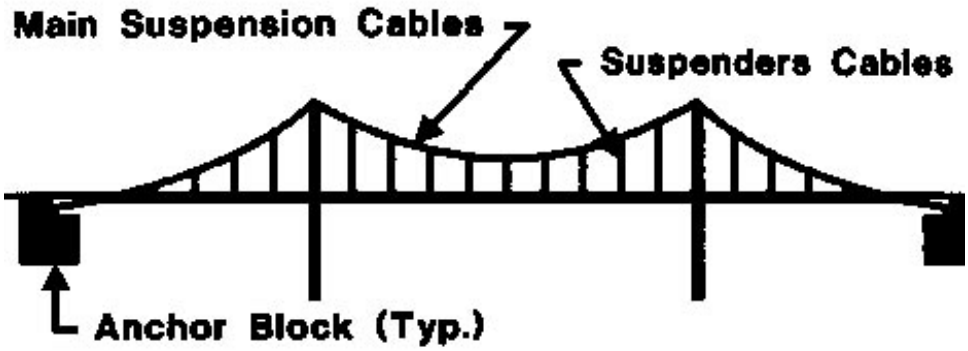


Figure 12.1.6 Three-Span Suspension Bridge Schematic

If a suspension bridge has only two main suspension cables, the cables are considered as fracture critical members. This is due to the non-redundancy (only two load paths) of the main suspension cables. Refer to Section 8.1 for a more detailed description of fracture criticality and redundancy.

Types of Cables

A cable may be composed of one or more structural wire ropes, structural wire strands, locked coil strands, parallel wire strands, or parallel wires.

- **Parallel Wire** - Parallel wire cable consists of a number of parallel wires. Size varies (see Figure 12.1.7)
- **Structural Wire Strand -** Structural wire strand is an assembly of wires formed helically around a center wire in one or more symmetrical layers. Sizes normally range from 50 to 100 mm (2 to 4 inches) (see Figure 12.1.8).
- **Structural Wire Rope -** Structural wire rope is an assembly of strands formed helically around a center strand (see Figure 12.1.9).
- **Parallel Strand Cable -** Parallel strand cable is a parallel group of strands (see Figure 12.1.10).
- **Locked Coil Strand -** Locked coil strand is a helical type strand composed of a number of round wires, then several layers of wedge or keystone shaped wires and finally several layers of Z- or S-shaped wires (see Figure 12.1.11).

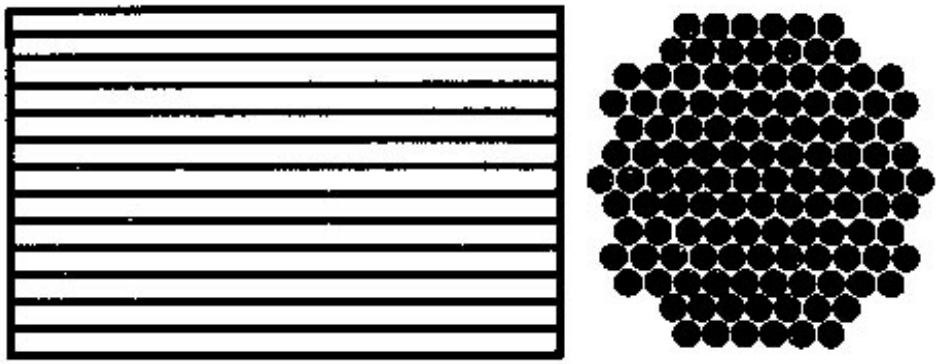


Figure 12.1.7 Parallel Wire Schematic

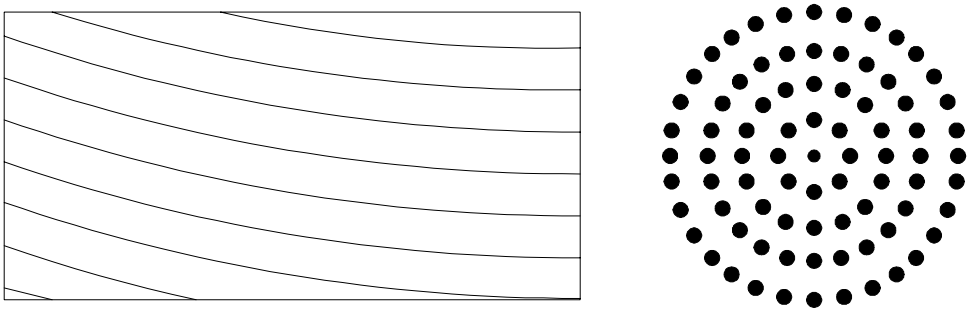


Figure 12.1.8 Structural Wire Strand Schematic

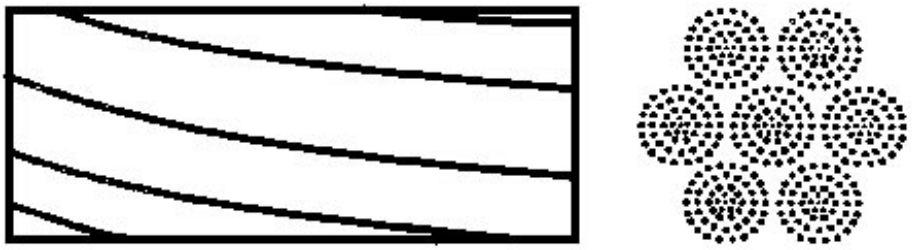


Figure 12.1.9 Structural Wire Rope Schematic

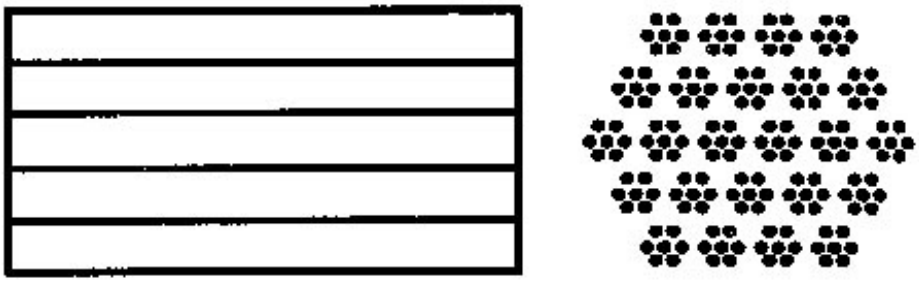


Figure 12.1.10 Parallel Strand Cable Schematic

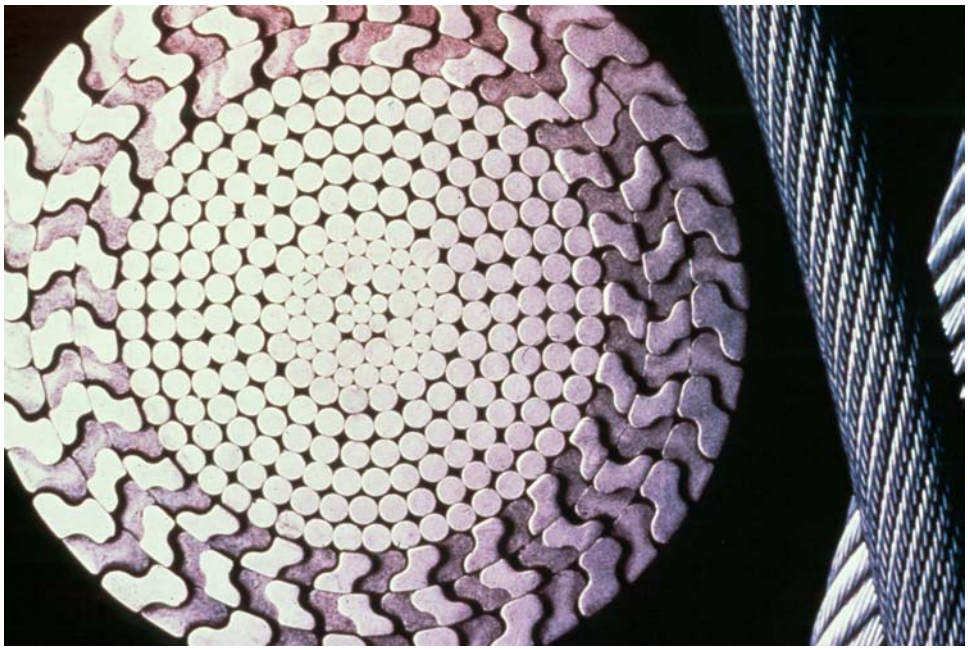


Figure 12.1.11 Locked Coil Strand Cross-Section

Corrosion Protection of Cables

Methods used for corrosion protection include:

- galvanizing the individual wires
- painting the finished cable
- wrapping the finished cable with spirally wound soft galvanized wire, neoprene, or plastic wrap type tape
- any combination of the above systems (see Figure 12.1.12).



Figure 12.1.12 Cable Wrapping on the Wheeling Suspension Bridge

Cable Anchorages

In bridges with common earth anchored cable systems, either above or below ground, the total force of the main cable has to be transferred into the anchor block (see Figure 12.1.13). Force transmission from cable to anchor block is established by anchoring the individual strands to the concrete of the block. Flaring of the strands takes place in the splay chamber. From the strand shoes or sockets the strand forces are transferred to the anchor block by steel bars, rods, pipes, or prestressed bars embedded into the concrete.

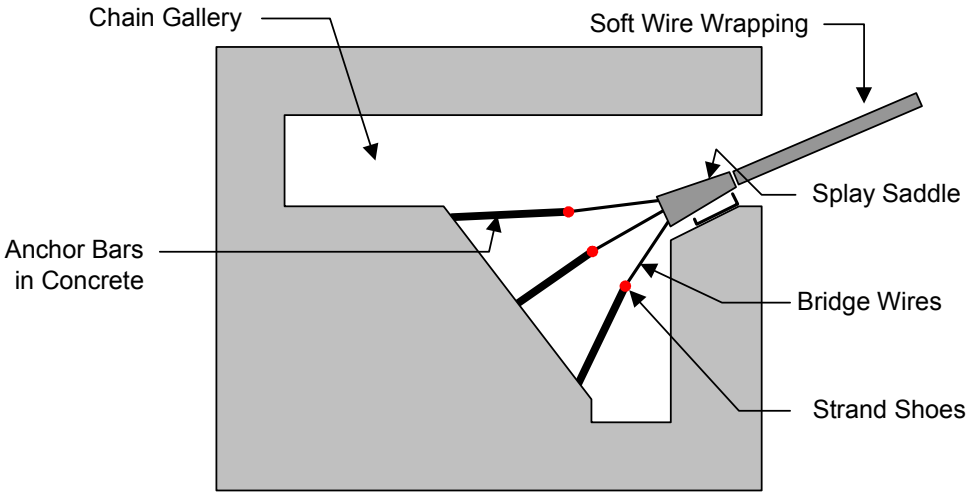


Figure 12.1.13 Gravity Anchor Schematic

Cable Saddles

The connection between cable and tower is usually made through saddles. The saddle supports the cable as it crosses over the tower. Saddles are commonly made from fabricated steel or castings (see Figure 12.1.14).



Figure 12.1.14 Cable Saddles for the Manhattan Bridge, NYC (Main Span 1,480 ft (451.1 m))

Suspender Cable Connections

The connection between the main and suspender cable is made by means of a cable band. The cable band consists of two semi-cylindrical halves connected by high-tensile steel bolts to develop the necessary friction.

- **Groove Cable Bands -** Grooved cable bands have been used in the majority of suspension bridges. The top surfaces of the bands are grooved to receive the suspender cables, which are looped over the band.

- **Open Socket -** Instead of looping the hanger cables around the band, the hanger might also be socketed at the upper end and pin connected to the cable band. This connection is called an open socket (see Figure 12.1.15). Connection to the deck and floor system can also be a similar open socket arrangement or it can be connected directly to a girder - similar to the tied arch bridge.

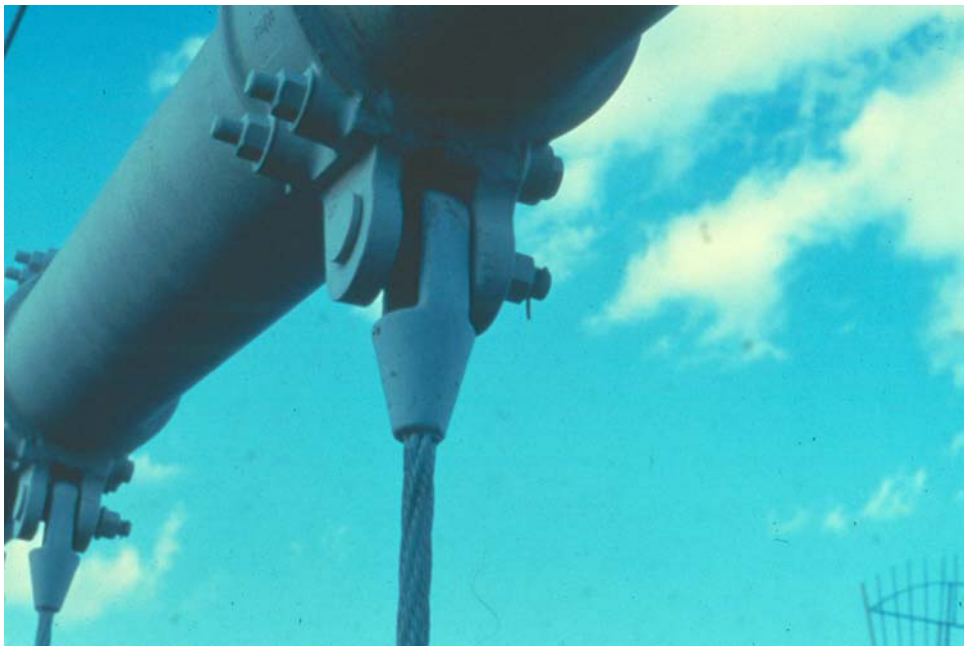


Figure 12.1.15 Open Socket Suspender Cable Connection (Brooklyn Bridge)

Vibrations

The flexibility of cable supported structures, associated with high stress levels in the main load carrying members, makes these structures especially sensitive to dynamic forces caused by earthquake, wind, or vehicular loads. The inspector should always note and describe vibrations whether local or global, while performing inspections of cable-supported structures. The term local vibration is used when dealing with the vibration in an individual member (see Figure 12.1.16). When the vibration of the entire structure as a whole is analyzed, it is known as global vibration (see Figure 12.1.17). Due to the amount of vibration in cable supported structures, it is not uncommon to see various types of damping systems attached to cables. Damping systems may be a tie between two cables, neoprene cushions, shock absorbers mounted directly to the cables, or other systems that act to dampen the cable vibrations.

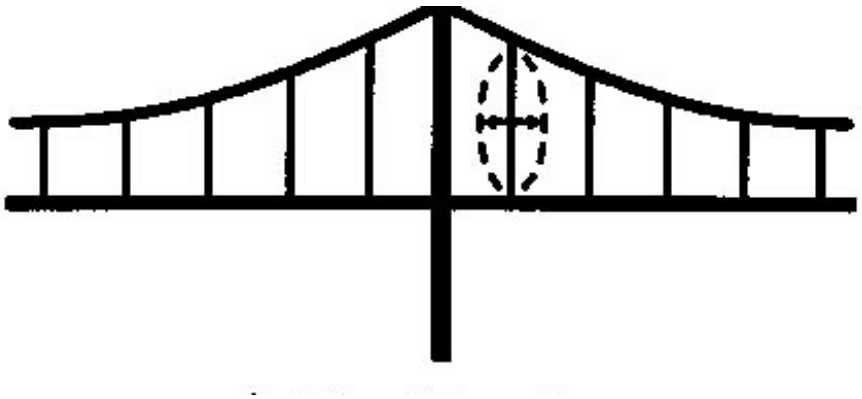


Figure 12.1.16 Cable Vibrations Local System Schematic

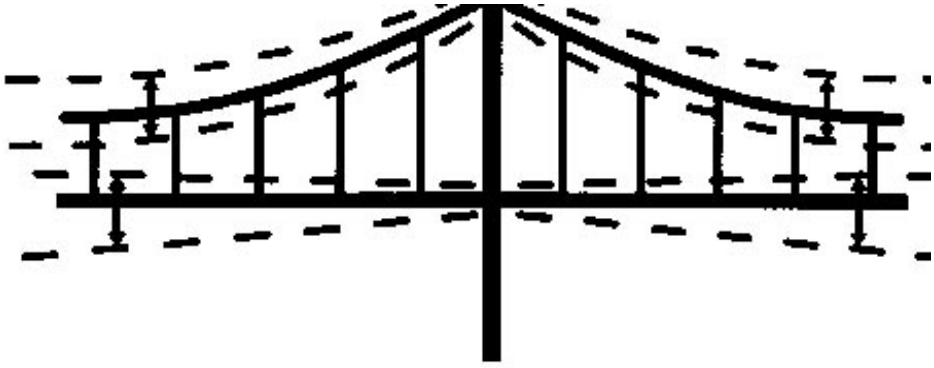


Figure 12.1.17 Cable Vibrations Global System Schematic

Effects of Vibrations

- Accelerates corrosion
- Opens cable wires allowing entry of corrosive chemicals
- Creates fretting
- Cracks grout
- Causes fatigue

John Roebling was the first to resolve the problem created by cable vibrations. His solution called for the use of diagonal cablestays (see Figure 12.1.18).

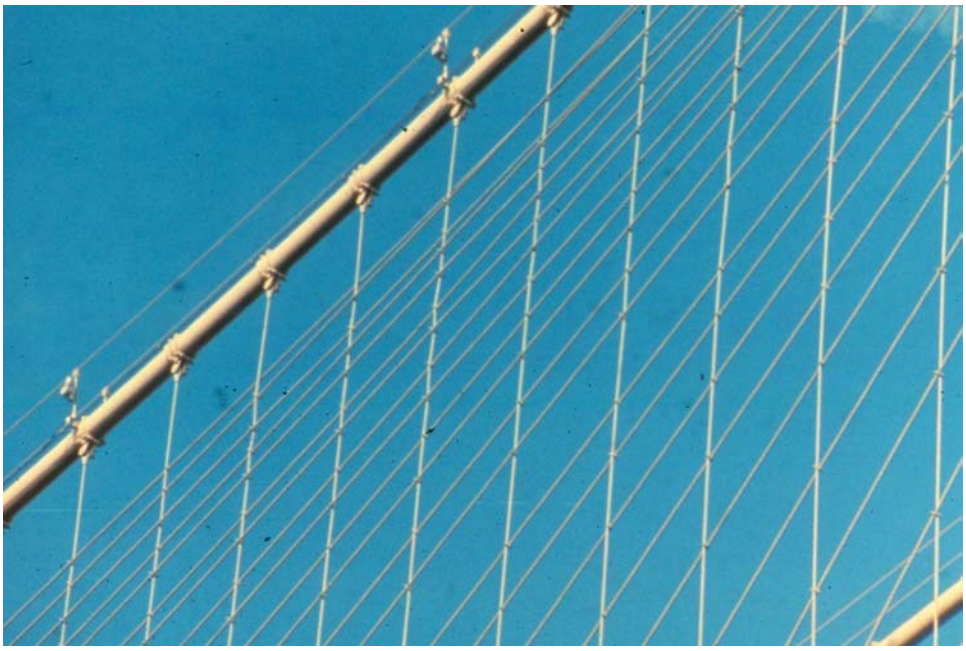


Figure 12.1.18 Close-up of Brooklyn Bridge Diagonal Cable-Stays

12.1.4 Cable-Stayed Bridges

Only the cable and its elements are described in this section. Refer to the appropriate section for detailed descriptions of other cable-stayed bridge members that are common to most structures.

Because of the complexity of the various cable arrangements and systems, fracture criticality for individual cable-stayed structures can only be determined through a detailed structural analysis.

Cable Arrangements and Systems

Cable-stayed bridges may be categorized according to the various longitudinal cable arrangements. These cable arrangements can be divided into the following four basic systems:

- Radial or Converging Cable System
- Harp Cable System
- Fan Cable System
- Star Cable System

Radial or Converging Cable System

In this system, all cables are leading to the top of the tower at a common point. Structurally, this arrangement is the most effective. By anchoring all the cables to the tower top, the maximum inclination to the horizontal is achieved (see Figure 12.1.19).

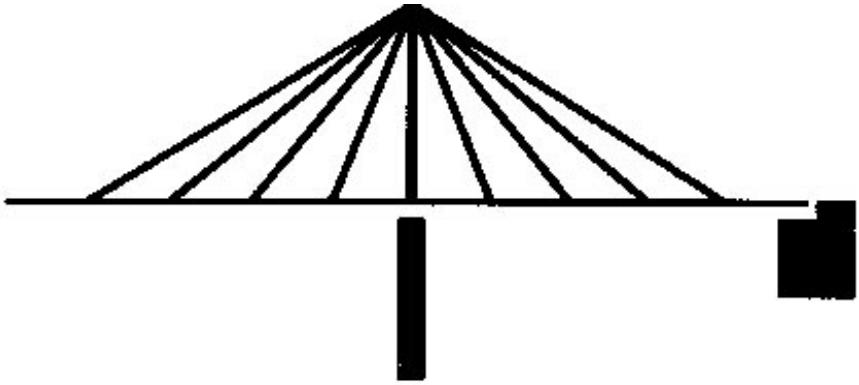


Figure 12.1.19 Radial or Converging Cable System Schematic

Harp Cable System

The harp system, as the name implies, resembles harp strings. In this system, the cables are parallel and equidistant from each other. The cables are also spaced uniformly along the tower height and connect to the deck superstructure at the same spacing (see Figure 12.1.20).

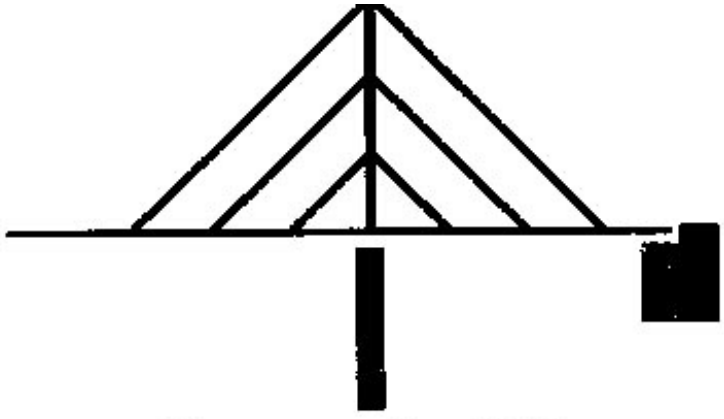


Figure 12.1.20 Harp or Parallel Cable System Schematic

Fan Cable System

The fan system is a combination of the radial and the harp systems. The cables emanate from the top of the tower at equal spaces and connect to the superstructure at larger equal spaces (see Figure 12.1.21).

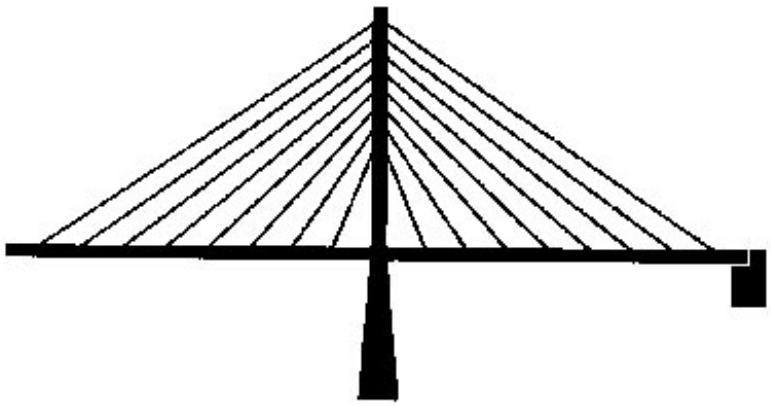


Figure 12.1.21 Fan or Intermediate Cable System Schematic

Star Cable System

In the star system, the cables intersect the tower at different heights and then converge on each side of the tower to intersect the deck structure at a common point. The common intersection in the anchor span is usually located over the abutment or end pier. The star system is rather uncommon. The star system requires a much stiffer deck structure since the cables are not distributed along the deck (see Figure 12.1.22).

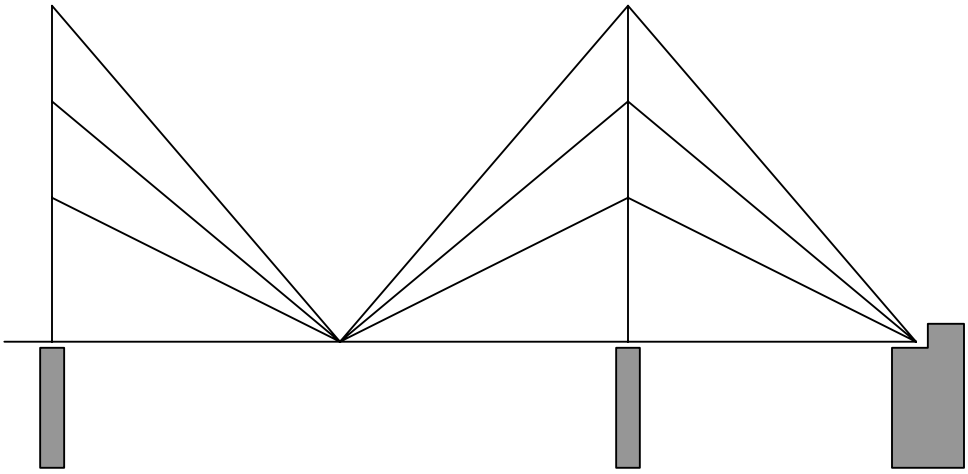


Figure 12.1.22 Star Cable System Schematic

Cable Planes

The cables may lie in either a single or a double plane, may be symmetrical or asymmetrical, and may lie in oblique or vertical planes.

Single Plane

The single-plane cable arrangement is used with a divided deck structure with the cables passing through the median area and anchored below the roadway. A single-plane cable system generally utilizes single towers (see Figure 12.1.23).



Figure 12.1.23 Single Vertical Plane Cable System

Double Vertical Plane

The double vertical plane system incorporates two vertical cable planes connecting the tower to the edge girders along the deck structure. The structure may utilize twin towers or a portal frame tower (see Figure 12.1.24). The portal frame tower is a twin tower with a connecting strut at the top. Wider bridges may utilize a triple plane system that is basically a combination of the single and double plane systems.



Figure 12.1.24 Double Vertical Plane Cable System

Double Inclined Plane

In this two plane system the cable planes are oblique, sloping toward each other from the edges of the roadway and intersecting at the tower along the longitudinal centerline of the deck (see Figure 12.1.25). Generally the tower is an A-frame type, receiving the sloping cables that intersect close to the roadway centerline on the tower.

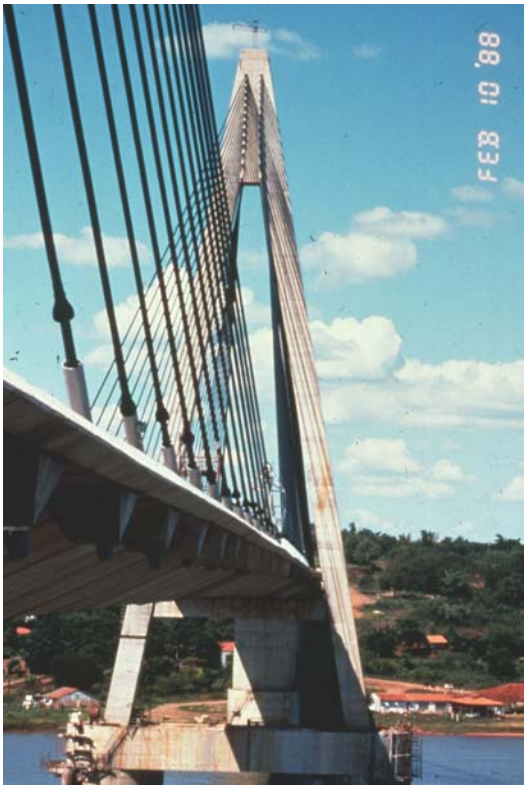


Figure 12.1.25 Double Inclined Plane Cable System

Types of Cables

Several types of cables have been used for cable-stayed bridges. The three most common are:

- Locked-coil strand
- Parallel wire
- Parallel seven-wire strand

The majority of existing cable-stayed bridges in the world, other than the United States, use preformed prestretched galvanized locked-coil strand. The cable-stayed bridges in the United States incorporate parallel wire or seven-wire prestressing strand in the cables, which are protected in a polyethylene tube filled with cement grout. The tube is commonly wrapped with a polyvinyl film.

Locked Coil Strand

This locked coil strand was previously shown for suspension bridges (refer back to Figure 12.1.11). Locked coil strand has not been used for bridges in this country. It is commonly used for cable-stayed bridges in Europe.

Parallel Wire

Parallel wire cables used in cable-stayed bridges conforms to ASTM A421, Type BA, low relaxation. It is basically stress-relieved wire used for prestressed concrete. Corrosion protection consists of polyethylene sheathing filled with cement grout. The tubing is usually wrapped with polyvinyl film tape (see Figure 12.1.26).

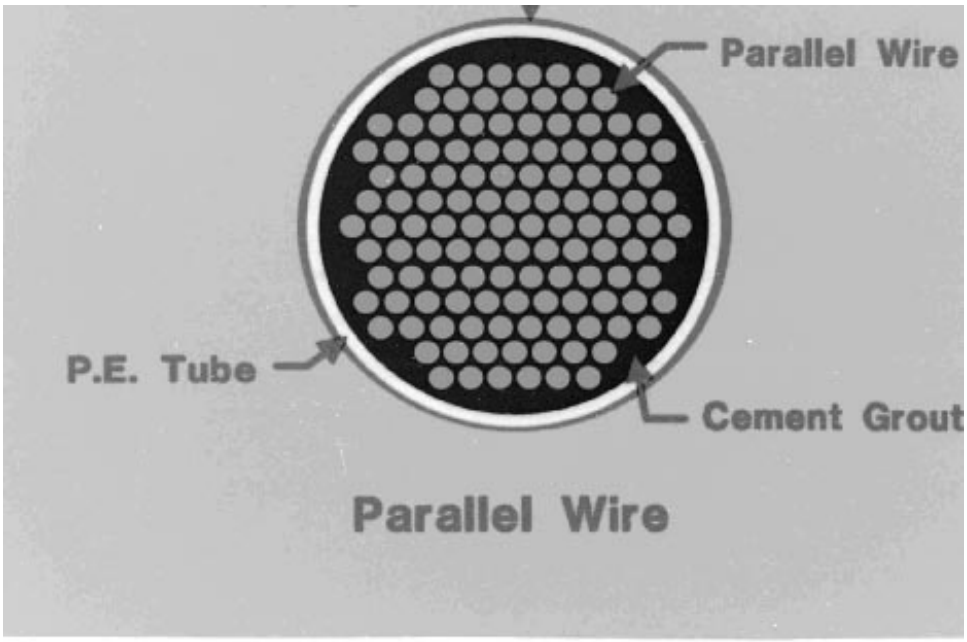


Figure 12.1.26 Parallel Wire

Parallel Strand

Seven-wire strand commonly used for cable-stayed bridges conforms to ASTM A416, weldless and low relaxation (see Figure 12.1.27). It is basically seven-wire stress-relieved strand for prestressed concrete. The corrosion protection system used for the seven-wire strand cables is similar to the system used for the parallel wire cables.



Figure 12.1.27 Parallel Strand

Anchorages and Connections

The cables may be continuous and pass through or over the tower or be terminated at the tower. If continuous across the tower, a saddle is incorporated.

Saddles

The cable saddles may be constructed from fabricated plates or steel castings with grooves through which the cables pass (see Figure 12.1.28). Between the end and center spans differential forces will occur at the cable saddles unless they are supported by rollers or rocker bearings. When the saddles are fixed, the rigidity of the system is at a maximum.

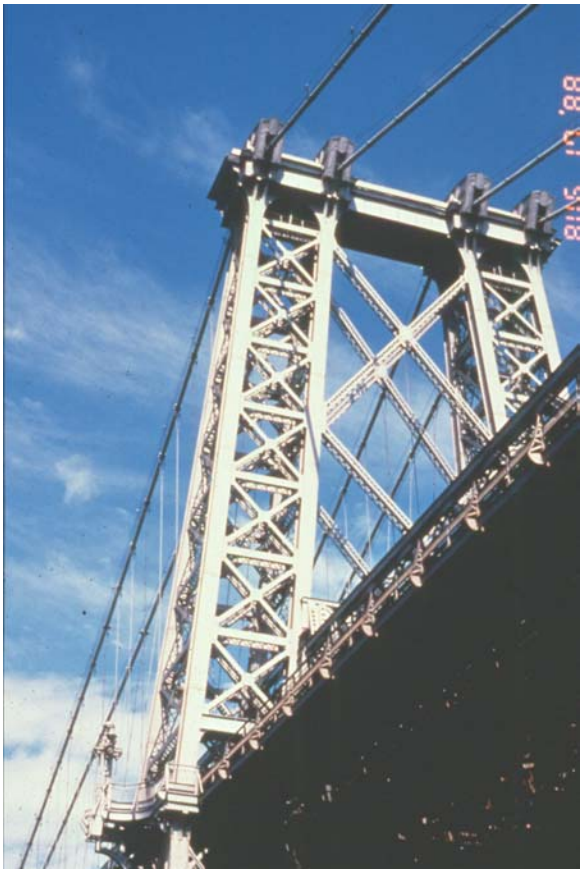


Figure 12.1.28 Cable Saddle

End Fittings

If terminated at the tower, an end fitting or anchorage is incorporated. A similar end fitting is utilized at the deck (see Figure 12.1.29).



Figure 12.1.29 Cable Deck Anchorage

Socket

A socket widely used for the anchoring of parallel-wire strands is a poured zinc socket (see Figure 12.1.30). The wires are led through holes in a licking plate at the end of the socket and have the bottom heads providing the resistance against slippage of wires. The cavity inside the socket is filled with hot zinc alloys. To improve the fatigue resistance of the anchor, a cold casing material is used.



Figure 12.1.30 Parallel Wire Zinc-Filled Socket

The problems encountered with low fatigue strength of zinc-poured sockets lead to the development of HiAm sockets in 1968 for use with parallel wire stays.

This anchorage incorporates a flat plate with countersunk radial holes to accommodate the geometry of flared wires that transition from the compact wire bundle into the anchorage. The anchorage socket is filled with zinc dust and with an epoxy binder. This method of anchoring the stays increases the magnitude of fatigue resistance to almost twice that for the zinc-poured sockets.

A common anchorage type for strands is the Freyssinet type anchor.

In the Freyssinet socket the seven wire strand is anchored to an anchor plate using wedges similar to prestressing wedges. This wedge anchor is used during erection. After application of dead load the anchor tube is filled with an epoxy resin, zinc dust, and steel ball composition. Under live load, the additional cable force will be transformed by shear from the cable strand to the tube.

Types of Towers

- Portal tower
- A-frame tower
- Single tower

Towers are constructed of reinforced concrete or steel or a combination of the two (see Figure 12.1.31 and 12.1.32).

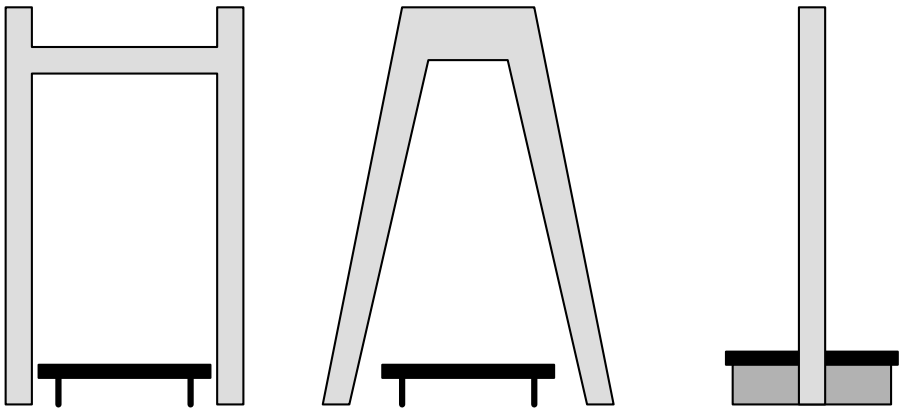


Figure 12.1.31 Tower Types Schematic



Figure 12.1.32 Inside of a Concrete Portal Tower

The deck structures are also constructed of concrete or steel.

12.1.5 Overview of Common Defects

Common defects that can occur on the cable members of a cable-supported bridge include:

- Failure of the Paint System
- Pitting
- Surface Rust
- Section Loss
- Fatigue Cracking

- Collision Damage
- Overload Damage
- Heat Damage

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

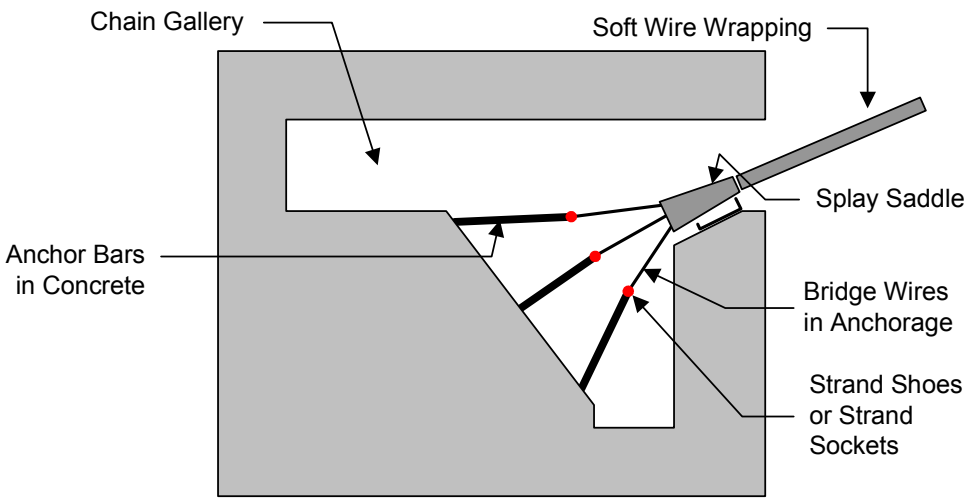
12.1.6 Inspection Locations and Procedures for Suspension Bridge Cable System Elements

The inspection and maintenance procedures presented in this Topic are not exhaustive, but are unique to the particular bridge type. Therefore, the inspection of special bridges should include both those procedures presented in this Topic and the general procedures presented previously in this manual.

Due to the specialized nature of these bridges and because no two cable supported bridges are identical, the inspection should be led by someone very familiar with the particular bridge. Many major bridges, such as cable supported bridges, will have individual maintenance manuals developed specifically for that bridge, like an "owner's" manual. If available, the inspector should use this valuable tool throughout the inspection process and should verify that specified routine maintenance has been performed. Customized, preprinted inspection forms should be used wherever possible to enable the inspector to report the findings in a rigorous and systematic manner.

Main Cable Anchorage Elements

The anchorage system, at the ends of the main cables, consists of a number of elements that require inspection (see Figure 12.1.33).



Anchor System

Figure 12.1.33 Schematic Slide Anchorage System

Splay Saddle

Inspect the splay saddles for:

- Missing or loose bolts
- Movement up the cable away from the splay; signs of this movement may be the appearance of unpainted strands on the lower side or "bunched up" wrapping on the upper side
- The presence of cracks in the casting itself

Wires in Anchorage

In parallel wire type suspension bridges, inspect the unwrapped wires between the strand shoes and the splay saddle. Carefully insert a large screwdriver between the wires and apply leverage. This will help reveal broken wires. Inspect the wires for:

- Abrasion damage
- Corrosion
- Movement.

Strand Shoes or Sockets

At the anchorages of parallel wire type suspension bridges, inspect the strand shoes for:

- Signs of displaced shims, along with movement, corrosion, misalignment, and cracks in the shoes

At the anchorages of prefabricated strand type suspension bridges, inspect the strand sockets for:

- Signs of movement
- Slack or sag
- Corrosion
- Broken sockets
- Unpainted or rusty threads at the face of the sockets; this may indicate possible "backing off" of nuts.

Anchor Bars

Inspect the anchor bars or rods for:

- Corrosion, deterioration, or movement at the face of their concrete embedment
- Corrosion or other signs of distress over the entire visible (unencased) portion

Anchorage Interior

Inspect the interior of the anchorage for:

- Corrosion and deterioration of any steel hardware
- Protection against water entering or collecting where it may cause corrosion
- Proper ventilation
- Cracks and spalls in the concrete anchor

Main Suspension Cables

The main suspension cables should be inspected as follows:

Locations

Inspect the main suspension cables for indications of corroded wires. Inspect the condition of the protective covering or coating, especially at:

- Low points of cables
- Areas adjacent to the cable bands
- Saddles over towers
- Anchorages

Cable Wrapping

Inspect the wrapping wire for:

- Cracks, staining, and dark spots (see Figure 12.1.34)
- Loose wrapping wires
- Cracks in the caulking where water can enter and cause corrosion of the main suspension cable
- Evidence of water seepage at the cable bands, saddles, and splay castings



Figure 12.1.34 Staining on Cable Wrap Indicating Internal Problems

Hand Ropes

Inspect the hand ropes and connections along the main cables for:

- Loose connections of stanchion to cable bands
- Too much slack in rope
- Bent or twisted stanchions (hand rope supports)
- Loose connections at anchorages or towers
- Corroded or deteriorated ropes or stanchions

Vibration

- Note and record all excessive vibrations.

Saddles

Inspect the saddles for:

- Missing or loose bolts
- Slippage of the main cable
- Corrosion or cracks in the casting
- Proper connection to top of tower or supporting member

Suspender Cables and Connections

Inspect the suspender cables for:

- Corrosion or deterioration
- Kinks or slack
- Abrasion or wear at sockets, saddles, clamps, and spreaders
- Broken wires
- Excessive vibrations

Sockets

Inspect the suspender rope sockets for:

- Corrosion, cracks, or deterioration
- Abrasion at connection to bridge superstructure
- Possible movement

Cable Bands

Inspect the cable bands for:

- Missing or loose bolts
- Possible slippage; signs of this movement are caulking that has pulled away from the casting or "bunching up" of the soft wire wrapping adjacent to the band
- The presence of cracks in the band itself
- Broken suspender saddles
- Corrosion or deterioration of the band
- Loose wrapping wires at the band

Recordkeeping and Documentation

A set of customized, preprinted forms should be prepared for documenting all defects encountered in the cable system of a suspension bridge. A suggested sample form is presented in Figure 12.1.35. A separate form should be used for each main suspension cable. Designations used to identify the suspender ropes and the panels provide a methodology for locating the defects in the structure.

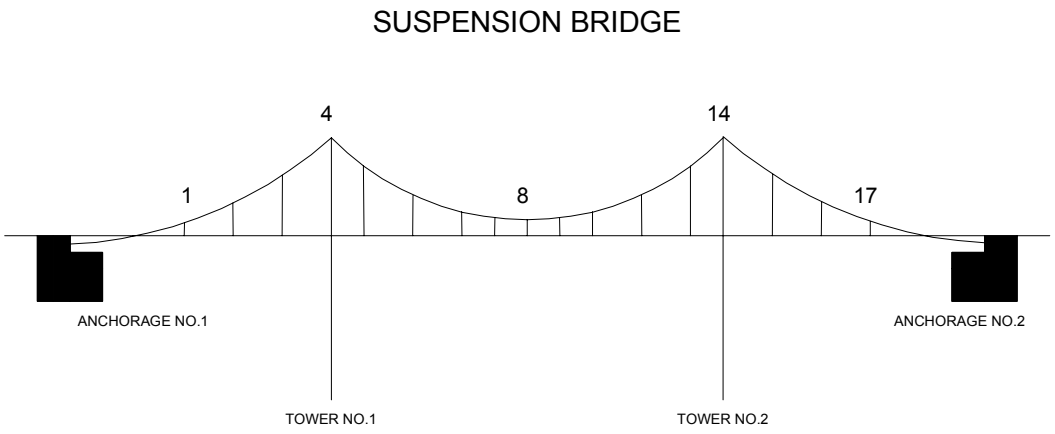


Figure 12.1.35 Form for Recording Defects in the Cable System of a Suspension Bridge

12.1.7 Inspection Locations and Procedures for Cable-Stayed Bridge Cable System Elements

A cable-stayed bridge is a bridge in which the superstructure is supported by cables, or stays, passing over or attached directly to towers located at the main piers (see Figure 12.1.36 and 12.1.37). There are several special elements that are unique to cable-stayed bridges, and the bridge inspector should be familiar with them.



Figure 12.1.36 Cable-Stayed Bridge

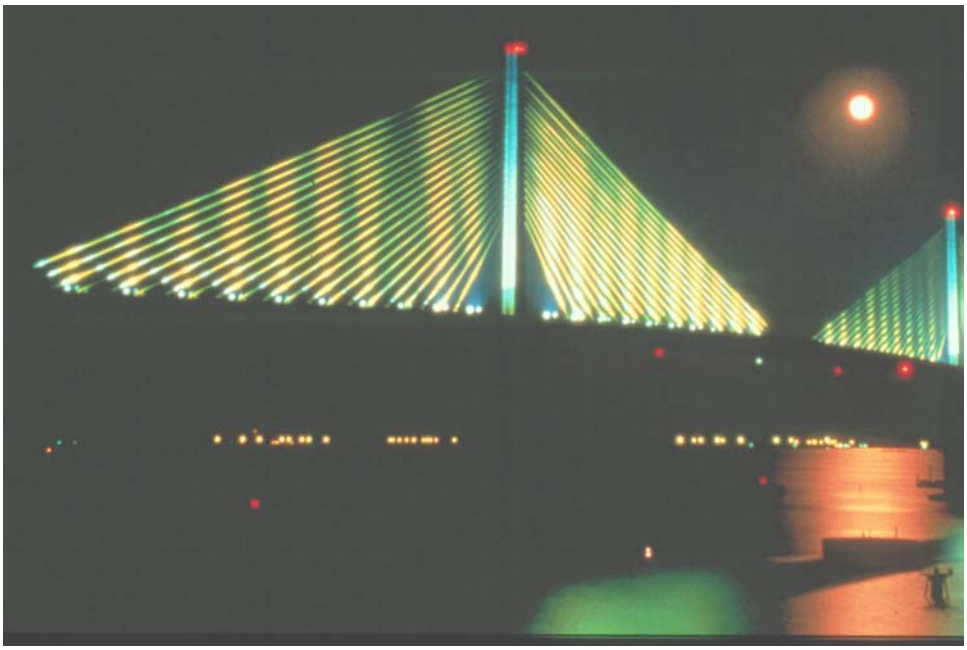


Figure 12.1.37 Cable-Stayed Bridge Cables

Inspection Elements

The inspection of the cable elements should include:

- Cable wrappings and wrap ends near the tower and deck
- Cable sheathing assembly
- Dampers
- Anchorages

Cable Wrapping

Common wrapping methods for corrosion protection of finished cables include spirally wound soft galvanized wire, neoprene, or plastic wrap type tape (see Figure 12.1.38). The wrappings should be inspected for:

- Corrosion and cracking of soft galvanized wire
- Staining and dark spots indicating possible corrosion of the cables
- Bulging or deforming of wrapping material indicating possible broken wire (see Figure 12.1.39)
- Loose wrapping wires or tape
- Evidence of water seepage at the cable bands, saddles, and splay castings



Figure 12.1.38 Cable Wrapping Placement



Figure 12.1.39 Investigation of Deformed Cable Wrapping

Cable Sheathing Assembly

The most common types of cable sheathing assemblies are steel sheathing and polyethylene sheathing.

Steel Sheathing

If steel sheathing is used, inspect the system for:

- Corrosion (see Figure 12.1.40)
- Condition of Protective coatings
- Bulging which may indicate broken wires (see Figure 12.1.41).
- Splitting which may be caused by water infiltration and corrosive action.
- Cracking which is sometimes caused by fatigue (see Figure 12.1.42).
- Weld fusion

Polyethylene Sheathing

If polyethylene sheathing is used, inspect the system for:

- Nicks, cuts, and abrasions
- Cracks and separations in caulking
- Cracks and separations in fusion welds
- Bulging which may indicate broken wires (see Figure 12.1.41).
- Splitting which is sometimes caused by temperature fluctuations (see Figure 12.1.43). Coefficient of the thermal expansion for polyethylene is three times higher than the value for steel or concrete.
- Cracking which is sometimes caused by fatigue



Figure 12.1.40 Corrosion of Steel Sheathing



Figure 12.1.41 Bulging of Cable Sheathing



Figure 12.1.42 Cracking of Cable Sheathing



Figure 12.1.43 Splitting of Cable Sheathing

Dampers

Shock Absorber Type

A variety of damper types may have been installed (see Figure 12.1.44 and 12.1.45). If shock absorber type dampers are used, inspect the system for:

- Corrosion
- Tightness in the connection to the cable pipe
- Oil leakage in the shock absorbers

- Deformations in the bushings
- Torque in the bolts



Figure 12.1.44 Shock Absorber Damper System

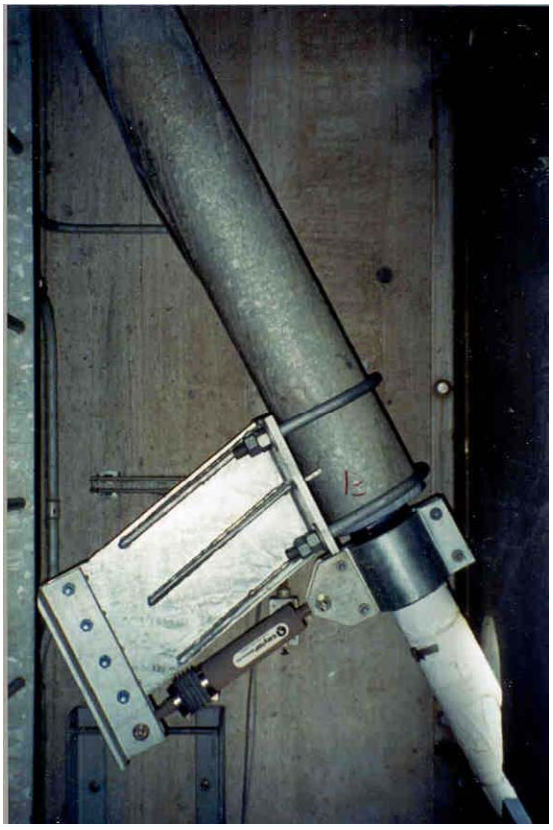


Figure 12.1.45 Shock Absorber Damper System

Tie Type
Inspect the tie type dampers for (see Figure 12.1.46):

- Corrosion
- Tightness in the connection to the cable pipe
- Deformations in the bushings
- Torque in the bolts

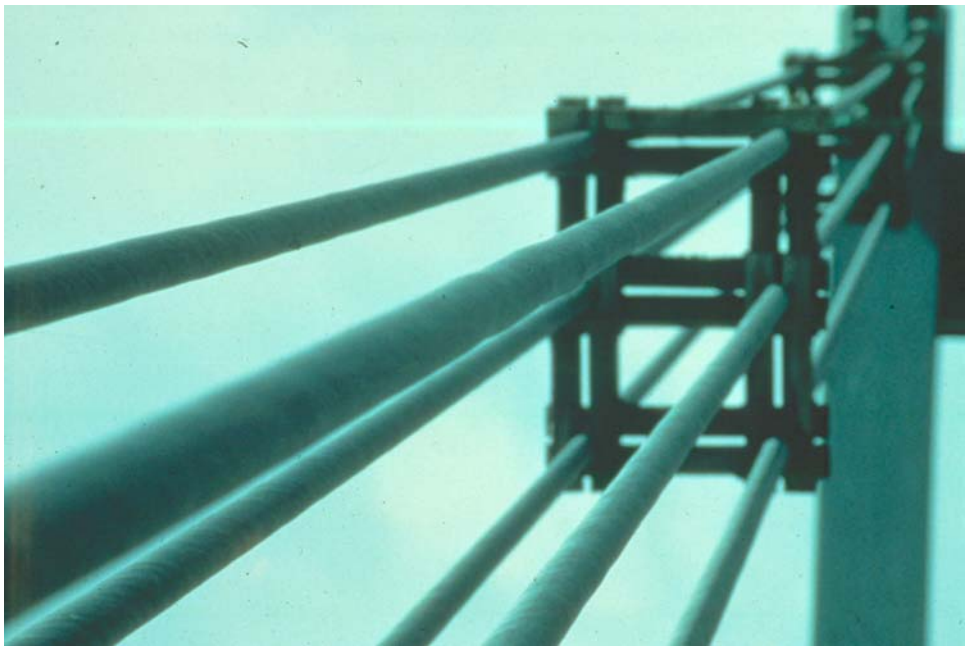


Figure 12.1.46 Cable Tie Type Damper System

Tuned Mass Type

Inspect the tuned mass dampers for (see Figure 12.1.47):

- Corrosion
- Tightness in the connection to the cable pipe
- Deformations in the bushings
- Torque in the bolts

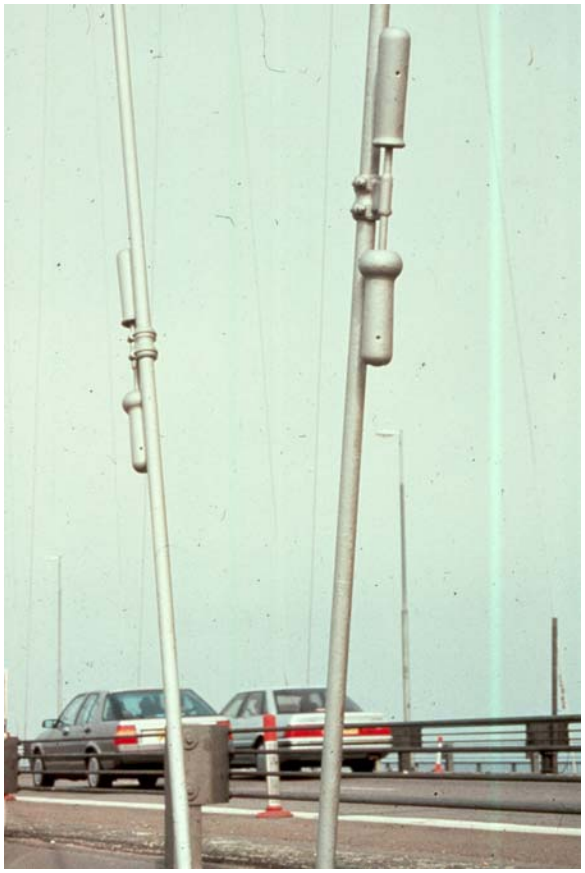


Figure 12.1.47 Tuned Mass Damper System

Anchorages

End Anchorage

Inspect the transition area between the steel anchor pipe and cable for:

- Water tightness of neoprene boots at the upper ends of the steel guide pipes (see Figure 12.1.48)
- Drainage between the guide pipe and transition pipe
- Defects, such as splits and tears, in the neoprene boots
- Sufficient clearance between the anchor pipe and cable, noting rub marks and kinks



Figure 12.1.48 Neoprene Boot at Steel Anchor Pipe Near Anchor

Tower Anchorage

Inspect the cable anchorages for:

- Corrosion of the anchor system (see Figure 12.1.49)
- Cracks and nut rotation at the socket and bearing plate
- Seepage of grease from the protective hood

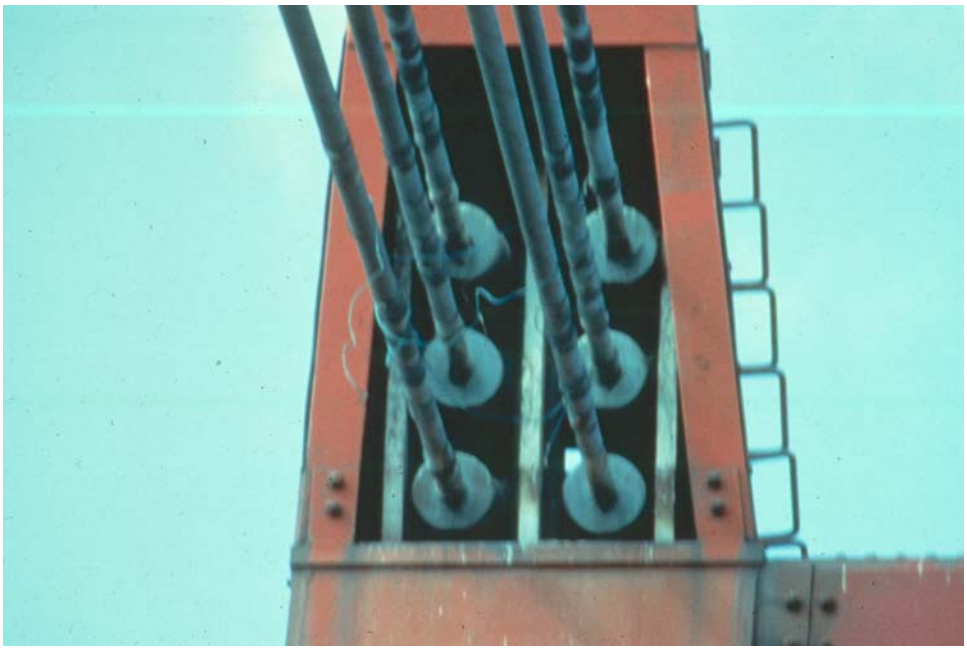


Figure 12.1.49 Corrosion of the Anchor System

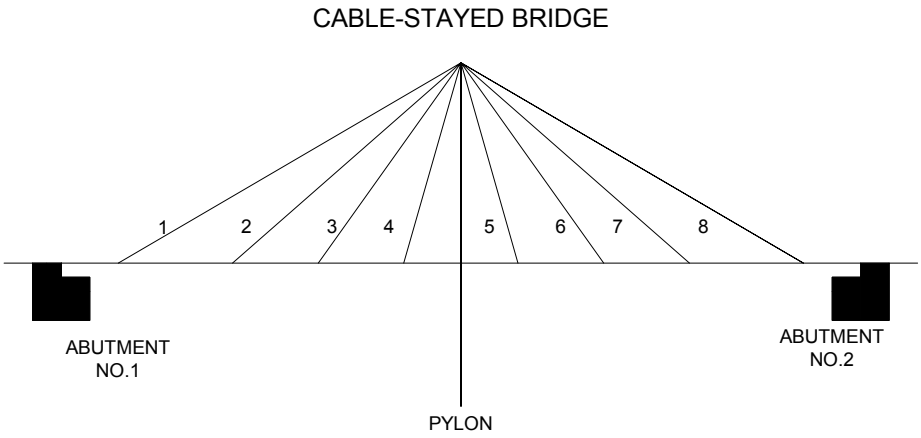
Other Inspection Items

The inspection of the cable system should also include:

- Anchor pipe clearances
- Flange joints
- Polyethylene expansion joints
- Reading the load cells and recording the forces
- Noting and recording all excessive vibrations including amplitude and type of vibration along with wind speed and direction, or other forces including vibrations such as traffic.
- Cable lighting

Recordkeeping and Documentation

A set of customized, preprinted forms should be prepared for documenting all defects encountered in the cable system of a cable-stayed bridge. A suggested sample form is presented in Figure 12.1.50. A separate form should be used for each plane or set of cables. Designations used to identify the cables and the panels provide a methodology for locating the defects in the structure.



SUSPENDER CABLE CONDITION

1.	5.
2.	6.
3.	7.
4.	8.

Figure 12.1.50 Form for Recording Defects in Cable System of a Cable-Stayed Bridge

12.1.8 Advanced Inspection Techniques

In bridge cables, whether a suspension bridge or cable-stayed bridge, the greatest problems generally occur due to corrosion and fracture of individual wires. Visual inspection of unwrapped cables is limited to the outer wires, while visual inspection of wrapped cables is limited to the protective sheathing. Therefore, advanced inspection techniques should be used to achieve a more rigorous and thorough inspection of the cables, including:

- Magnetic induction
- Electrical resistivity

- Dye penetrant
- Ultrasonic testing
- Radiographic testing
- Acoustic emission
- Accelerometers
- Strain measurements
- Vibration measurements
- Magnetic flux leakage
- Measurement of loads
- Measurement of stress ranges

See Section 13.3 for Advanced Inspection Techniques.

12.1.9 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of steel superstructures. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

Using NBIS rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Rating codes range from 9 to 0 where 9 is the best rating possible. See Section 4.2.

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the cables. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element Level Smart Flags are also used to describe the condition of the steel superstructure both unpainted and painted.

In an element level condition state assessment of a cable-supported bridge, the AASHTO CoRe element is:

Element No.	Description
146	Unpainted Steel Cable (not embedded in concrete)
147	Painted Steel Cable (not embedded in concrete)

The unit quantity for cables is each and the total number of cables must be placed in one of the four available condition states for unpainted and five available condition states for painted. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

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Topic 12.2 Movable Bridges

12.2.1 Introduction

This section serves as an introduction to the highly specialized area of movable bridge inspection (see Figure 12.2.1). It focuses on the types of movable bridges and special elements associated with the various types.



Figure 12.2.1 Movable Bridge

Movable bridges are normally constructed only when fixed bridges are either too expensive or impractical. Movable bridges are constructed across designated "Navigable Waters of the United States," in accordance with "Permit Drawings" approved by the U.S. Coast Guard. When a movable bridge is fully open, it must provide the channel width and the underclearance shown on the Permit Drawings (see Figure 12.2.2). If the bridge cannot be opened to provide these clearances, the U.S. Coast Guard should be notified immediately and action taken to restore the clearances. If that is impossible, application must be made to revise the Permit Drawings.

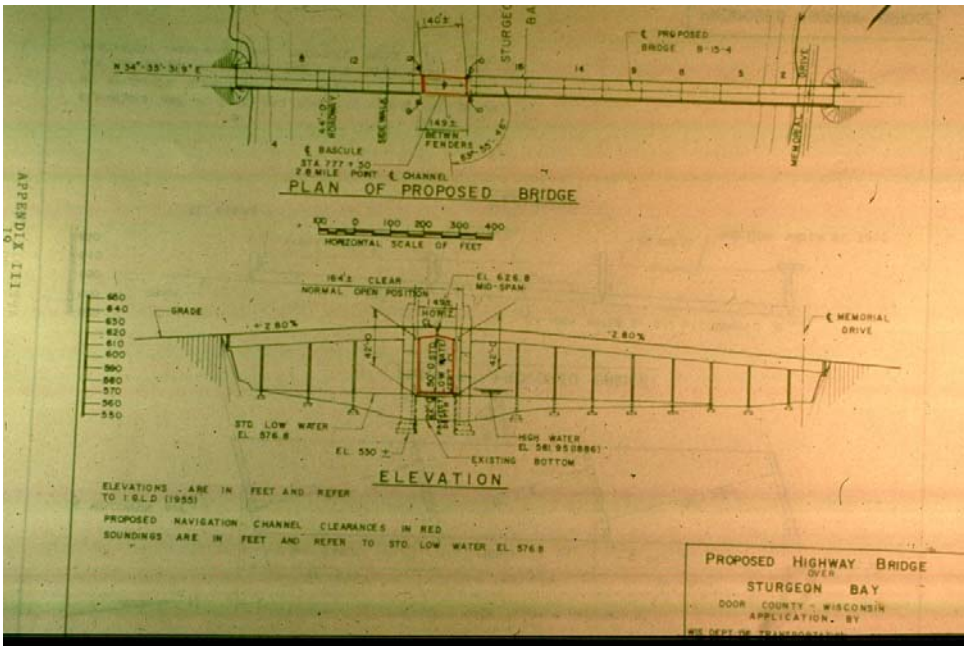


Figure 12.2.2 Typical "Permit Drawing" Showing Channel Width and Underclearance, Both Closed and Open, that Must be Provided

If any work is to be done in the channel or on the movable span to reduce the clearances from those shown on the Permit Drawing, an additional permit, covering the scheduled time for the work, must be obtained from the U.S. Coast Guard District that has jurisdiction.

The U.S. Coast Guard publishes Local Notices to Mariners to keep waterway users informed of work in progress that may affect navigation. The permittee must keep the U.S. Coast Guard informed of all stages of construction.

An inspection of the bridge should verify that the bridge conforms to the Permit Drawing and that the operator is instructed to open the bridge to the fully open position every time the bridge is operated. Failure to do this would establish a precedent that a vessel is expected to proceed before the green navigation lights have turned "on." Any accident caused as a result of this practice could be ruled the fault of the bridge owner.

Early America's engineering literature did not establish where the first iron drawbridge was built. The first all-iron movable bridge in the Midwest was completed in 1859 carrying Rush Street over the Chicago River (see Figure 12.2.3). The bridge was a rim bearing swing span and was probably operated by steam. It was destroyed November 3, 1863 when it was opened while a drove of cattle was on one end. It was rebuilt and finally destroyed by the great Chicago fire of 1871.

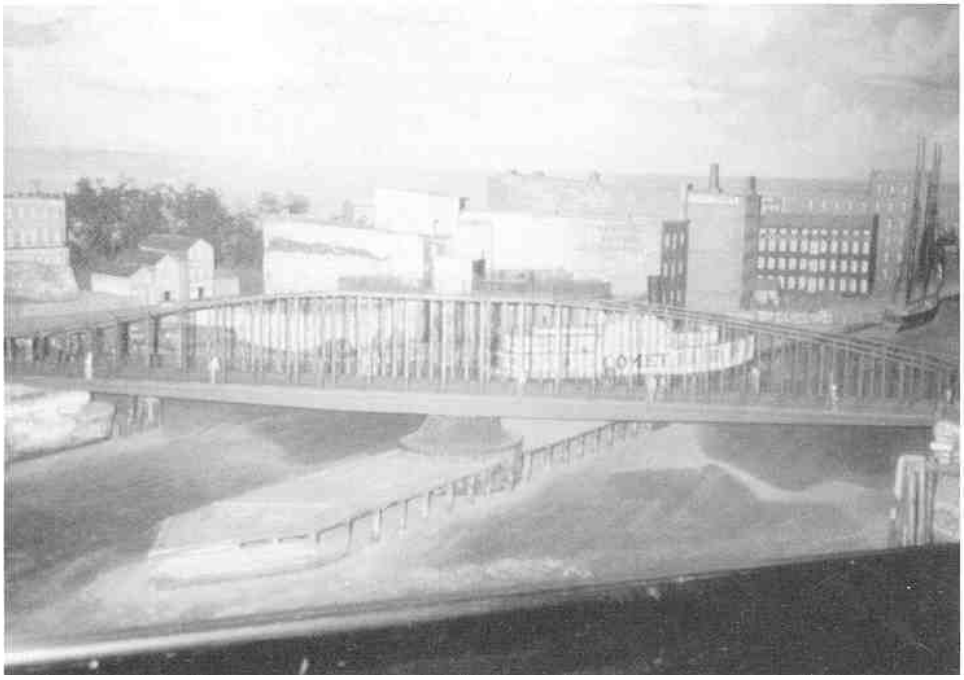


Figure 12.2.3 The First All-Iron Movable Bridge in the Midwest was Completed in 1859 (Photo on File at the Chicago Historical Society

All categories of movable bridges are powered by electric-mechanical or hydraulic-mechanical drives with power driven pinions operating against racks, or by hydraulic cylinders. A small number are hand powered for normal operation, and a few use it for standby operation. Three categories of movable bridges comprise over 95 percent of the total number of movable bridges within the United States. These categories are:

- Swing bridges
- Bascule bridges
- Vertical lift bridges

12.2.2 Swing Bridges Design Characteristics

Swing bridges consist of two-span trusses or continuous girders, which rotate horizontally about the center (pivot) pier (see Figure 12.2.4). The spans are usually, but not necessarily, equal. When open, the swing spans are cantilevered from the pivot (center) pier and must be balanced longitudinally and transversely about the center. When closed, the spans are supported at the pivot pier and at two rest (outer) piers or abutments. In the closed condition, wedges are usually driven under the outer ends of the bridge to lift them, thereby providing a positive reaction sufficient to offset any possible negative reaction from live load and impact in the other span. This design feature prevents uplift and hammering of the bridge ends under live load conditions.



Figure 12.2.4 Center-Bearing Swing Bridge

Swing spans are subdivided into two types:

- Center-bearing
- Rim-bearing

Center-Bearing

Center-bearing swing spans carry the entire load of the bridge on a central pivot (usually metal discs). Balance wheels are placed on a circular track around the outer edges of the pivot pier to prevent tipping (see Figures 12.2.5 and 12.2.6). When the span is closed, wedges similar to those at the rest piers are driven under each truss or girder at the center pier. This relieves the center bearing from carrying any live load. However, these wedges should not raise the span at the pivot pier, but should merely be driven tight.

The latest swing spans built are nearly all of the center-bearing design. Center-bearing swing spans are less complex and less expensive to build than rim-bearing swing spans.

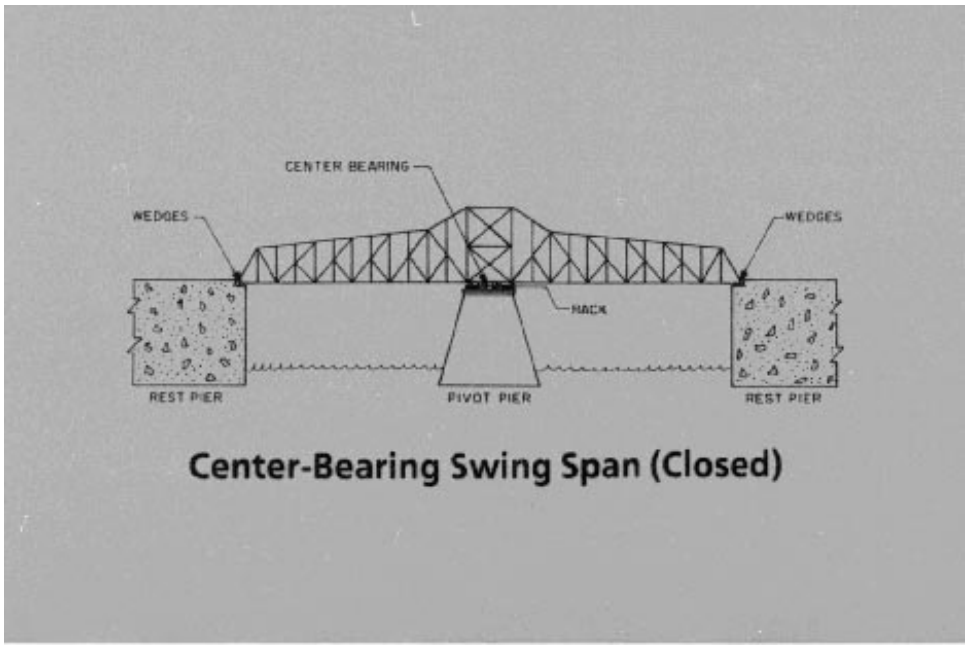


Figure 12.2.5 Center-Bearing Swing Span in Closed Position

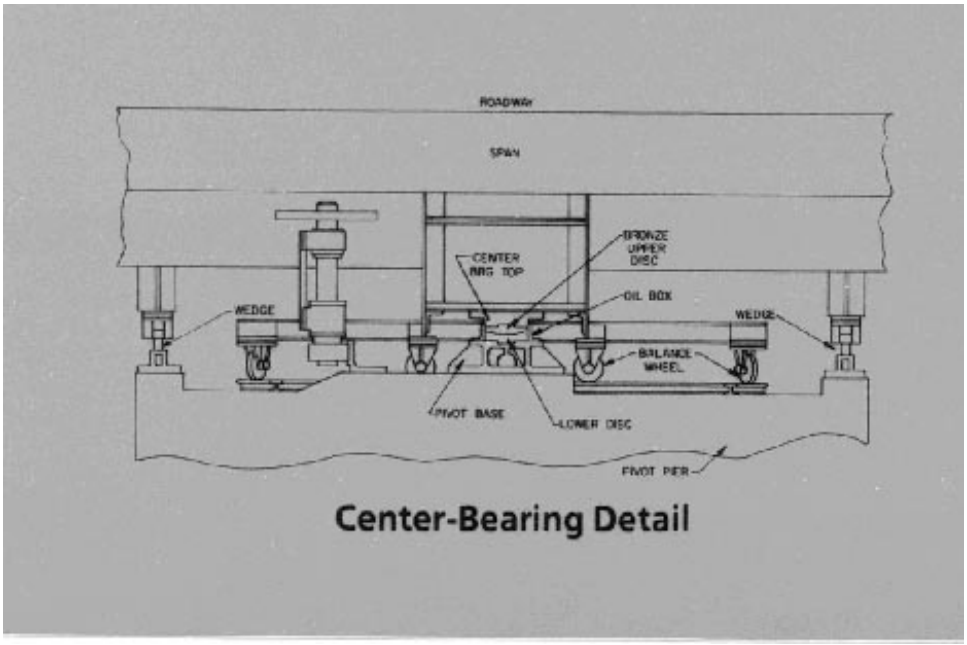


Figure 12.2.6 Layout of Center-Bearing Type Swing Span with Machinery on the Span

Rim-Bearing

Rim-bearing swing spans transmit all loads, both dead and live, to the pivot pier through a circular girder or drum to beveled rollers. The rollers move on a circular track situated inside the periphery of the pier. The rollers are aligned and spaced on the track by concentric spacer rings. This type of swing span bridge also has a central pivot bearing which carries part of the load. This pivot bearing is connected to the rollers by radial roller shafts and keeps the span centered on the circular track.

On both types of swing bridges, the motive power is usually supplied by an electric motor(s), hydraulic motor(s), or hydraulic cylinder(s), although gasoline engines or manual power may also be used. The bridge is rotated horizontally by a circular rack and pinion arrangement, or cylinders.

12.2.3 Bascule Bridges Design Characteristics

Bascule bridges open by rotating a leaf or leaves (movable portion of the span) from the normal horizontal position to a point that is nearly vertical, providing an open channel of unlimited height for marine traffic (see Figure 12.2.7).



Figure 12.2.7 Bascule Bridge in the Open Position

If the channel is narrow, a single span may be sufficient. This is called a single-leaf bascule bridge. For wider channels, two leaves are used, one on each side of the channel. When the leaves are in the lowered position, they meet at the center of the channel. This is known as a double-leaf bascule bridge.

A counterweight is necessary to hold the raised leaf in position. In older bridges, the counterweight is usually overhead, while in more modern bascule bridges, the counterweight is placed below the deck and lowers into a pit as the bridge is opened.

The leaf lifts up by rotating vertically about a horizontal axis. The weight of the counterweight is adjusted by removing or adding balance blocks in pockets to position the center of gravity of the moving leaf at the center of rotation. When the bridge is closed, a forward bearing support located in front of the axis is engaged and takes the live load reaction. On double-leaf bascule bridges, a tail-lock behind the axis and a shear lock at the junction of the two leaves are also engaged to stiffen the deck.

There are many types of bascule bridges, but the most common are the following

three types:

- Rolling lift (Scherzer) bridge
- Simple trunnion (Chicago) bridge
- Multi-trunnion (Strauss) bridge

Rolling Lift (Scherzer) Bridge

The first rolling lift bridge, completed in January 1895 in Chicago, was designed by William Scherzer. The entire moving leaf, including the front arm with the roadway over the channel and the rear arm with the counterweight, rolls away from the channel while the moving leaf rotates open (see Figure 12.2.8). On this type of bridge, curved tracks are attached to each side of the tail end of the leaf. The curved tracks roll on flat, horizontal tracks mounted on the pier. Square or oblong holes are machined into the curved tracks. The horizontal tracks have lugs (or teeth) to mesh with the holes preventing slippage as the leaf rolls back on circular castings whose centerline of roll is also the center of gravity of the moving leaf. Double-leaf bascule bridges are also constructed (see Figure 12.2.9).

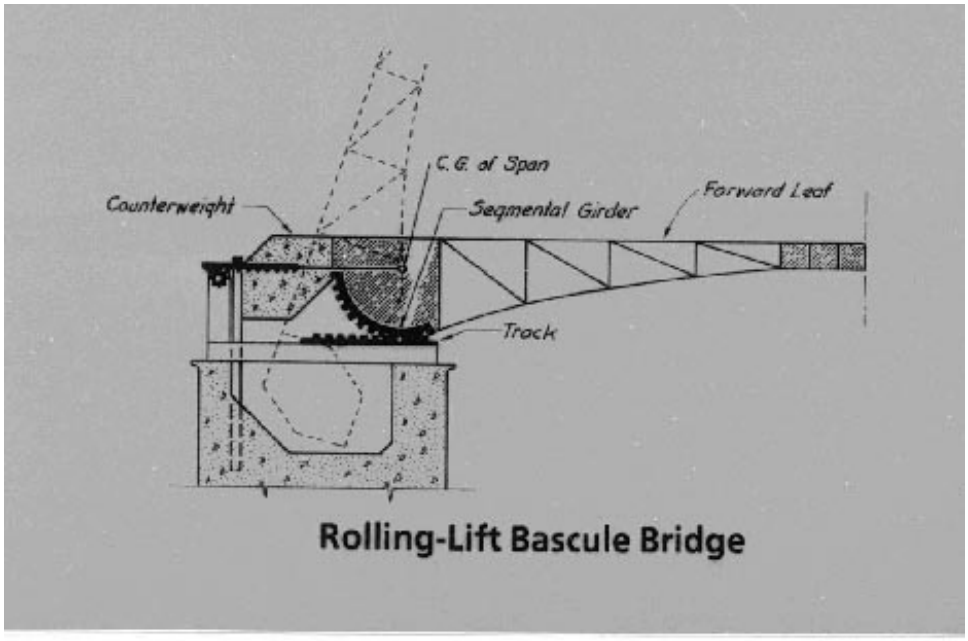


Figure 12.2.8 Rolling Lift Bascule Bridge Schematic



Figure 12.2.9 Double-Leaf Rolling Lift Bascule

The simple principal of this type of bridge can be seen easiest with a railroad bridge. The dead load of the bridge is balanced about the centerline of the drive pinion (center of roll). The pinion teeth are engaged with the teeth on the rack casting. When the pinion turns it moves along on the fixed rack and causes the span to rotate on the circular tread casting as it rolls back on the track casting.

The weight of the leaf, including the superstructure and counterweight, is supported by the curved tracks resting on the horizontal tracks. The counterweight is positioned to balance the weight of the leaf.

On one variation of this type, the trusses on the two leaves acted as three-hinged arches when closed. There is a 310 feet (94.5 m) span between the centerline of bearings. This bridge was built across the Tennessee River at Chattanooga in 1915, and it is believed to be the third longest double-leaf bascule in the world. It provides a 295-foot (89.9 m) channel, which is the widest channel spanned by a bascule bridge.

Simple Trunnion (Chicago) Bridge

The Chicago Bridge Department staff of Engineers designed the first Chicago type simple trunnion bascule bridge, completed during 1902. This type of bascule bridge consists of a forward cantilever arm out over the channel and a rear counterweight arm (see Figure 12.2.10). The leaf rotates about the trunnions. Each trunnion is supported on two bearings, which in turn, are supported on the fixed portion of the bridge such as a trunnion cross-girder, steel columns, or on the pier itself (see Figures 12.2.11, 12.2.12, 12.2.13). Forward bearing supports located in front of the trunnions are engaged when the leaf reaches the fully closed position. They are intended to support only live load reaction. Uplift supports are located behind the trunnions to take uplift until the forward supports are in contact (if misadjusted) and to take the live load uplift that exceeds the dead load reaction at the trunnions. If no forward live load supports are provided or if they are grossly misadjusted, the live load and the reaction at the uplift supports are added to the

load on the trunnions. A double-leaf bascule bridge of this type in Lorain, Ohio has101.5 m (333 feet) between trunnions. It was built on a skewed crossing of a river, and it is believed to be the second longest double-leaf bascule in the world. Of the three types of movable bridges, the simple trunnion is by far the most popular.

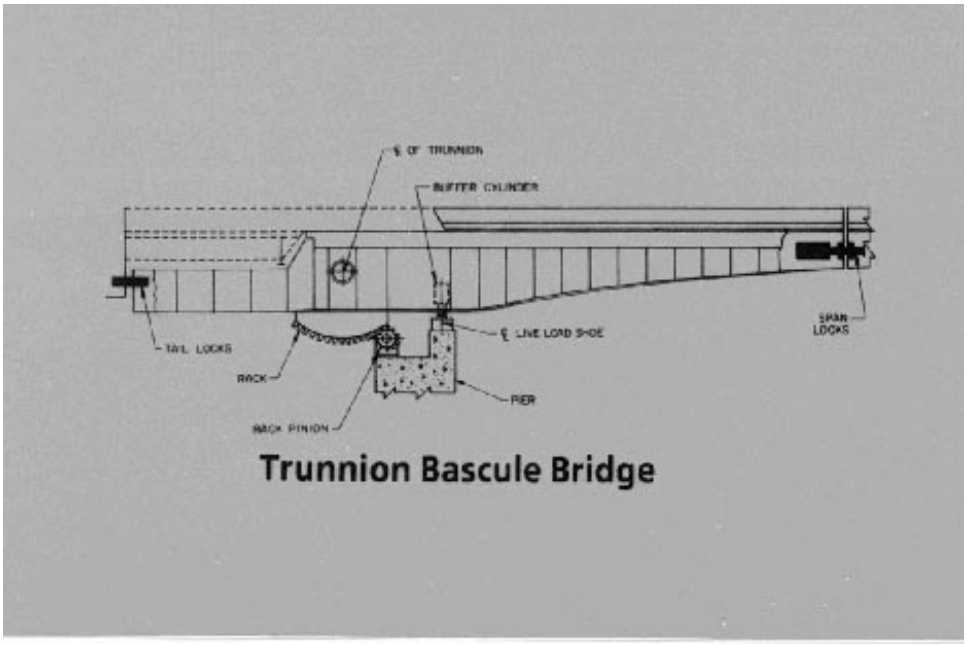


Figure 12.2.10 Trunnion Bascule Bridge Schematic



Figure 12.2.11 Double-Leaf Trunnion Bascule Bridge

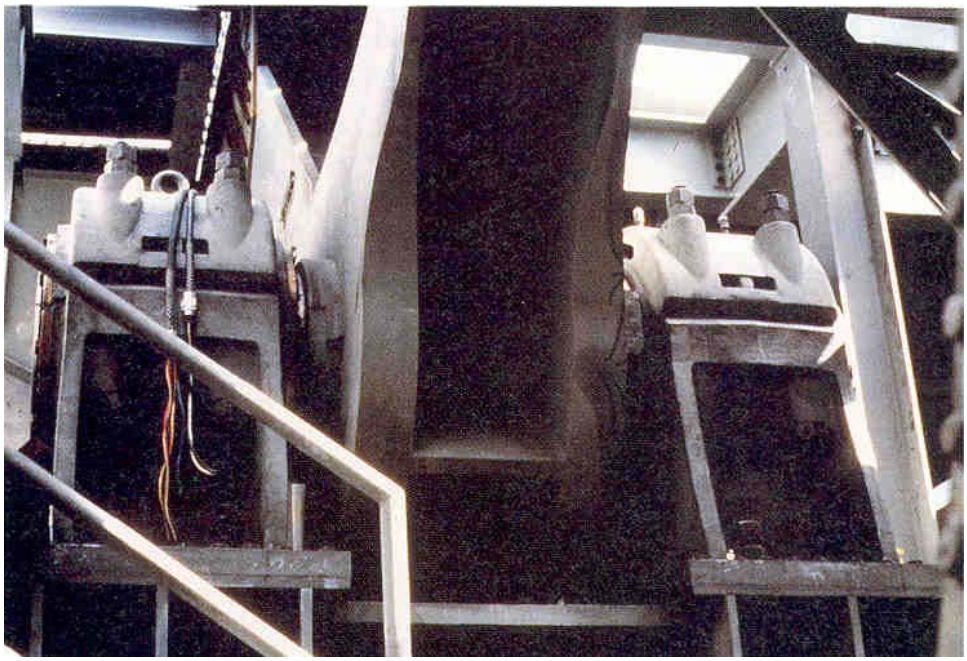


Figure 12.2.12 Each Trunnion is Supported on Two Bearings, Which in Turn are Supported on a Fixed Cross-Girder

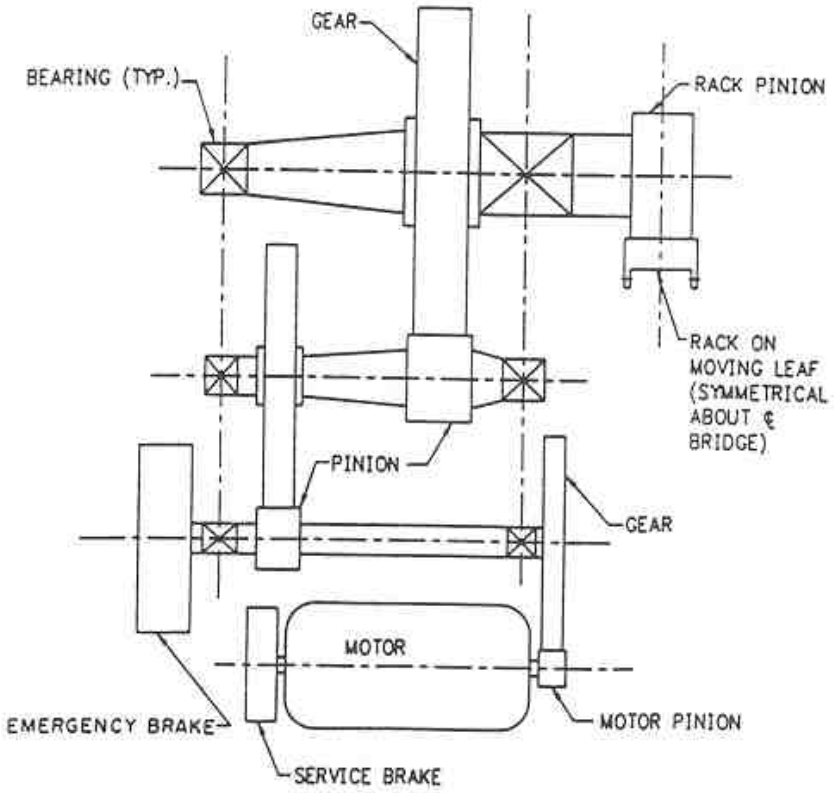


Figure 12.2.13 The Trunnion Bascule Bridge Machinery (One Quarter Shown) is Located Outside of the Bascule Trusses on the Pier; the Leaf Rotates about the Centerline on the Trunnions

Multi-Trunnion (Strauss Bridge)

The first multi-trunnion (Strauss) bascule bridge was designed by J.B. Strauss and completed during 1905 in Cleveland, Ohio. There are many variations of multi-trunnion bascule bridges, but basically one trunnion supports the moving span, one trunnion supports the counterweight, and two link pins are used to form the four corners of a parallelogram-shaped frame that changes angles as the bridge is operated (see Figure 12.2.14). One variation of this parallelogram layout is the heel trunnion. A double-leaf bascule bridge of this type in Sault St. Marie, Michigan has 336 feet (102.4 m) between the span trunnions. It was built across the approach to a lock in 1914, and it is believed to be the longest double-leaf bascule in the world.

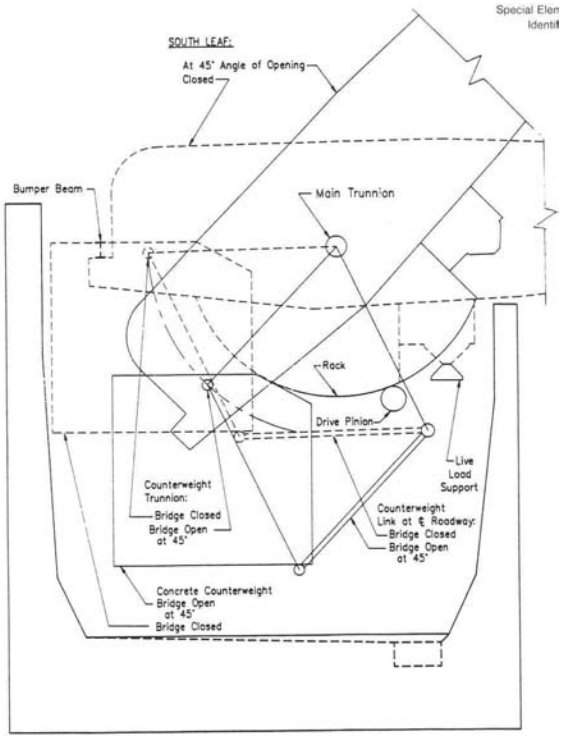


Figure 12.2.14 Multi-Trunnion, Strauss Type Bascule Bridge, in which the Counterweight Link Keeps the Counterweight Hanging Vertically from the Counterweight Trunnions while the Moving Leaf Rotates about the Main Trunnions

12.2.4 Vertical Lift Bridges Design Characteristics

Vertical lift movable bridges have a movable span with a fixed tower at each end. The span is supported by steel wire ropes at its four corners. The ropes pass over sheaves (pulleys) atop the towers and connect to counterweights on the other side. The counterweights descend as the span ascends (see Figure 12.2.15).

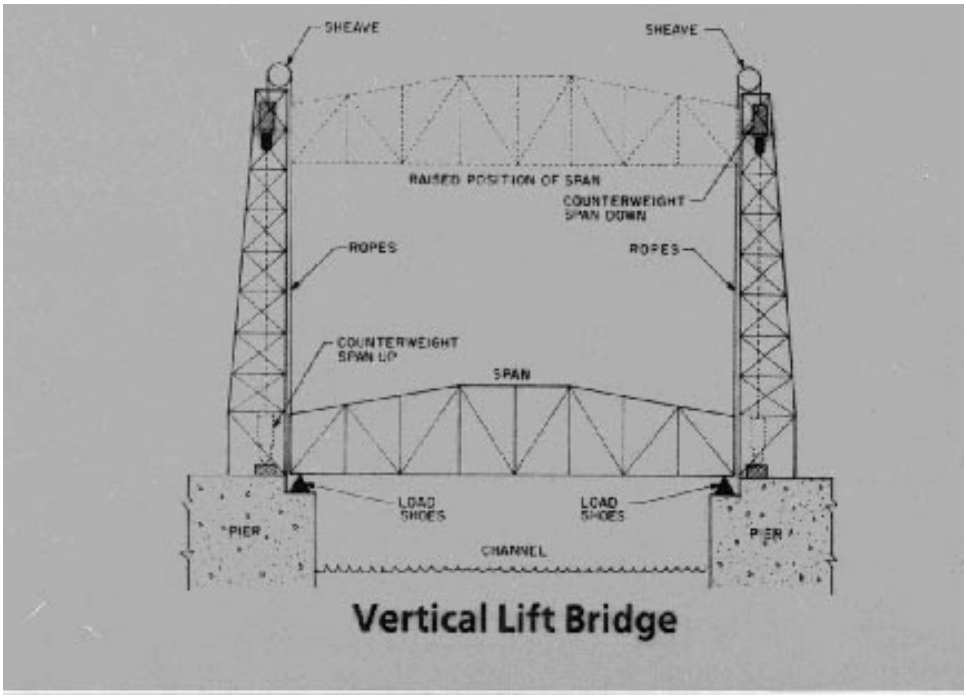


Figure 12.2.15 Vertical Lift Bridge Schematic

There are two basic types of vertical lift bridges:

- Power and drive system on lift span
- Power and drive system on towers

Power and Drive System on Lift Span

The first vertical lift bridge completed during 1894 in Chicago was designed by J.A.L. Waddell. This type locates the power on top of the lift truss span. The actual lifting is accomplished using "up-haul and down-haul ropes" where turning drums wind the up-haul (lifting) ropes as they simultaneously unwind the down-haul ropes (see Figures 12.2.16). A variation of this type provides drive pinions at both ends of the lift span which engage racks on the towers.

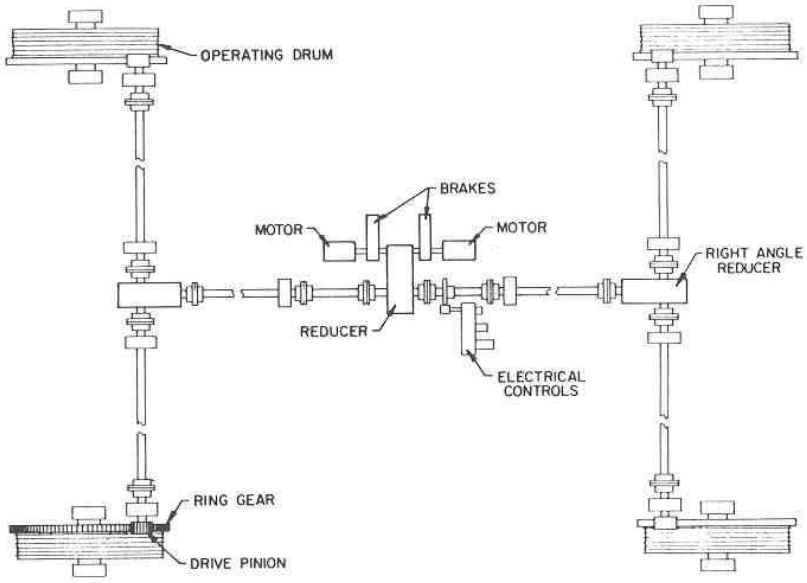


Figure 12.2.16 Vertical Lift Bridge Machinery is Located on Top of the Lift Truss Span, and the Operating Drums Rotate to Wind the Up-Haul (Lifting) Ropes as They Simultaneously Unwind the Down-Haul Ropes

Power and Drive System on Towers

The other basic type of vertical lift bridge locates the power on top of both towers, where drive pinions operate against circular racks on the sheaves. The lifting speed at both towers must be synchronized to keep the span horizontal as it is lifted (see Figures 12.2.17 and 12.2.18).

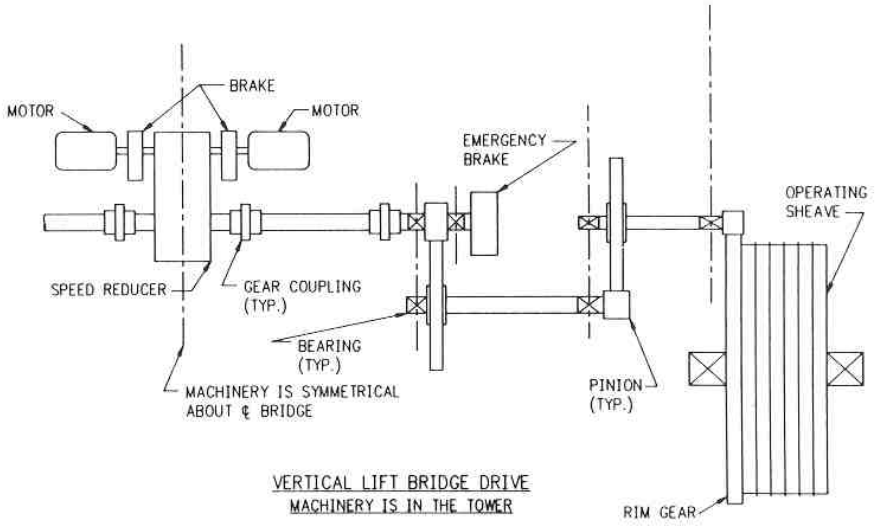


Figure 12.2.17 Vertical Lift Bridge Machinery is Located on the Towers, and the Rim Gears (and Operating Sheaves) are Rotated to Raise and Lower the Bridge



Figure 12.2.18 Vertical Lift Bridge with Power and Drive System on Towers

12.2.5 Special Elements Common to All Movable Bridges

Particular attention should be given to the special elements found in swing bridges, bascule bridges, and vertical lift bridges during inspection. These elements are commonly found on all types of movable bridges.

Open Gearing

Open gearing is used to transmit power from one shaft to another and to alter the speed and torque output of the machinery. Beveled gears are also used to change direction (see Figure 12.2.19).

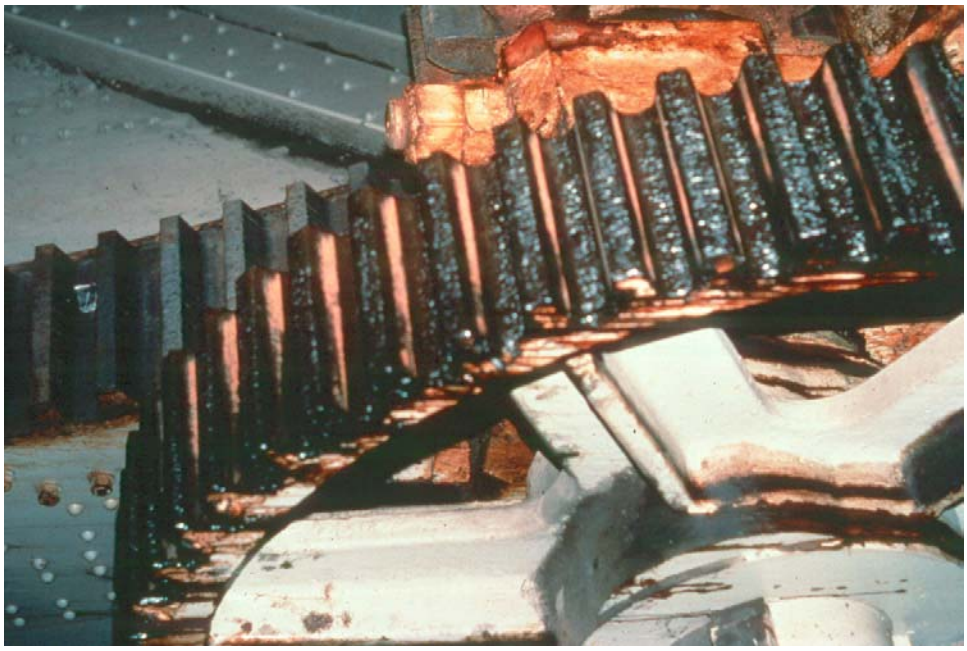


Figure 12.2.19 Open Gearing

Speed Reducers Including Differentials

Speed reducers including differentials serve the same function as open gearing (see Figure 12.2.20). However, they may contain several gear sets, bearings, and shafts to provide a compact packaged unit, which protects its own mechanical elements and lubrication system with an enclosed housing. Differential speed reducers also function to equalize torque and speed from one side of the mechanical operating system to the other.

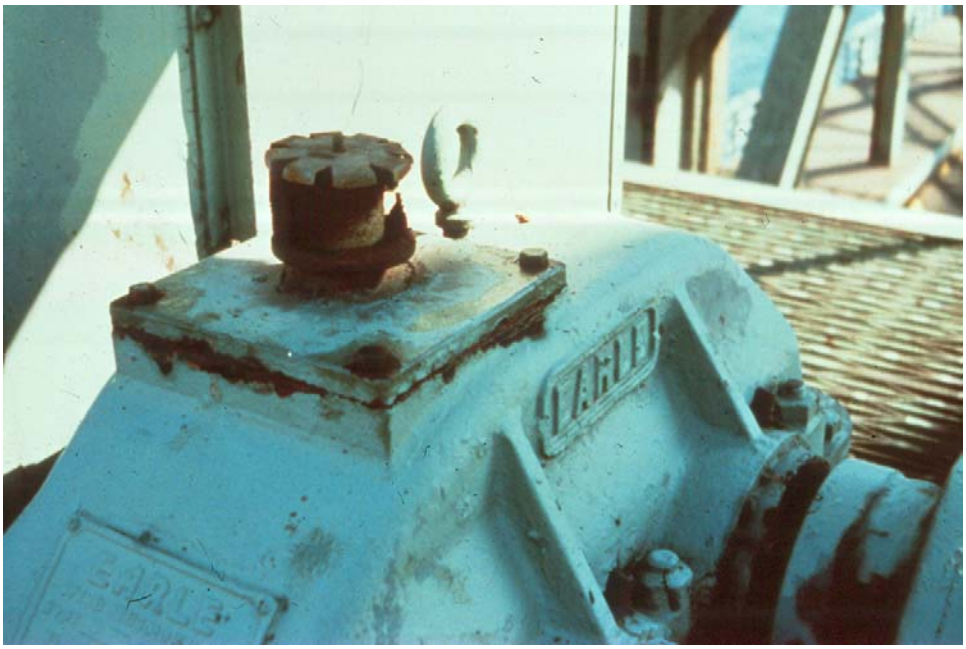


Figure 12.2.20 Speed Reducer

Shafts and Couplings

Shafts transmit mechanical power from one part of the machinery system to another. Couplings transmit power between the ends of shafts in line with one another, and several types can be used to compensate for slight imperfections in alignment between the shafts (see Figure 12.2.21).

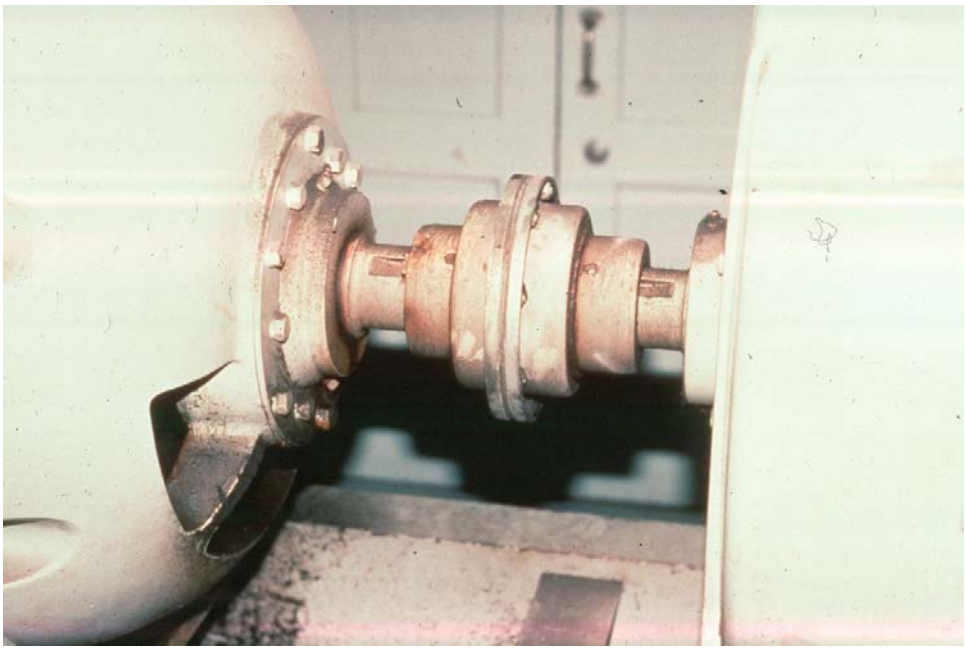


Figure 12.2.21 Coupling

Bearings

Bearings provide support and prevent misalignment of rotating shafts, trunnions, and pins (see Figure 12.2.22).

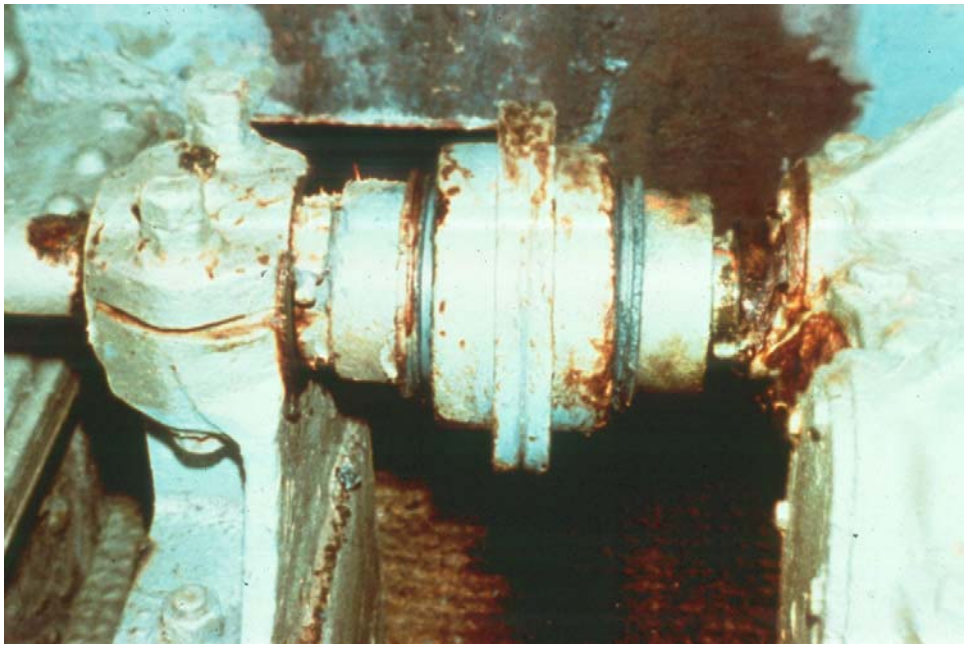


Figure 12.2.22 Bearing

Brakes

Brakes can be of either the shoe type or disc type, and can be released manually, electrically, or hydraulically (see Figure 12.2.23 and 12.2.24). They are generally spring applied for fail safe operation. Motor brakes are located close to the drive to provide dynamic braking capacity, except that some types of drives can provide their own braking capability, thereby eliminating the need for separate motor brakes. Machinery brakes are located closer to the operating interface between movable and fixed parts of the bridge and are used to hold the span statically, in addition to serving as emergency brakes in many cases. Supplemental emergency brakes are sometimes also provided.

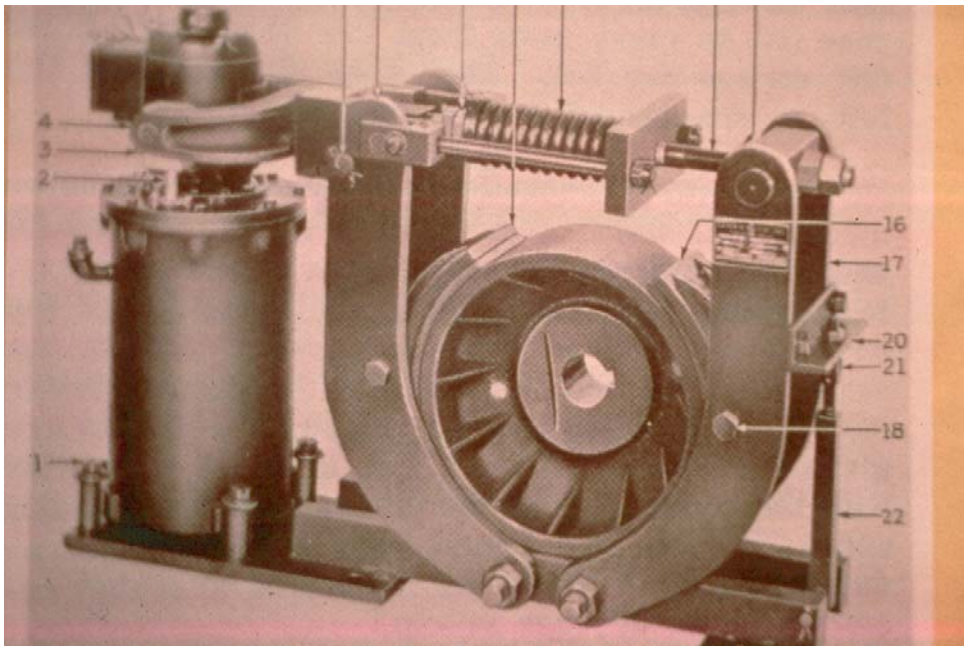


Figure 12.2.23 Shoe Type Break

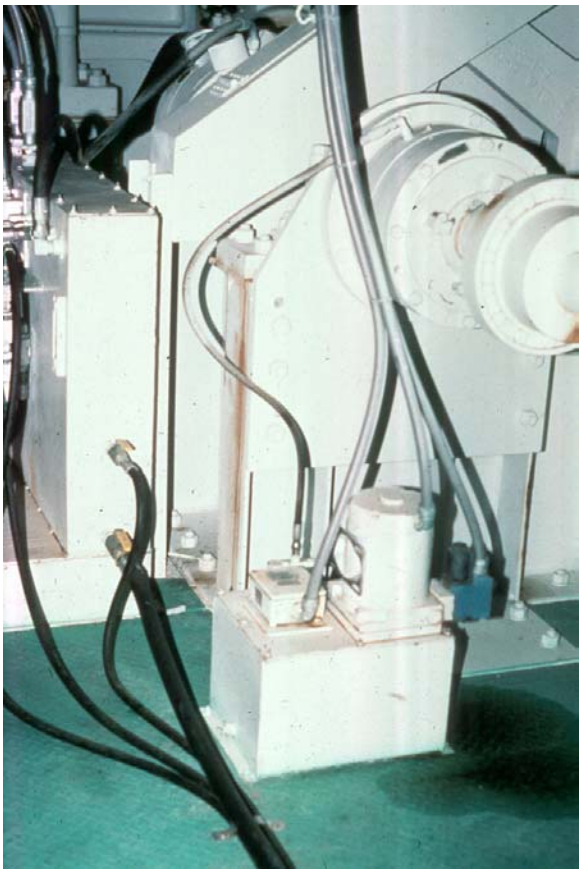


Figure 12.2.24 Spring Set Hydraulically Released Disc Break

Drives

Drives can consist of electric motors, hydraulic equipment, or auxiliary drives.

- Electric Motors - For electric motors, either AC or DC power may be used. AC power is often used to power wound rotor motors with torque controllers on older bridges, while new bridges may utilize squirrel cage induction motors with adjustable frequency speed control. DC motors can also provide speed control.

- Hydraulic Equipment - For hydraulic equipment, prime movers may include either large actuating cylinders or hydraulic motors (see Figure 12.2.25). Either type of drive must be supplied with pressure to provide force and fluid flow to provide speed to the operating system. Electrically operated hydraulic power units consisting of a reservoir and pump, with controls, provide power to the operating systems.

- Auxiliary Drives - For auxiliary drives, emergency generators are provided to serve in the event of power failure. Auxiliary motors and hand operators, with their clutches and other mechanical power transmission components, are provided to serve in the event the main drive fails (see Figure 12.2.26). In some cases, to prevent the need for larger auxiliary generators, the auxiliary motors are required for use any time the auxiliary generators are used, requiring increased time of operation.



Figure 12.2.25 Low Speed High Torque Hydraulic Motor

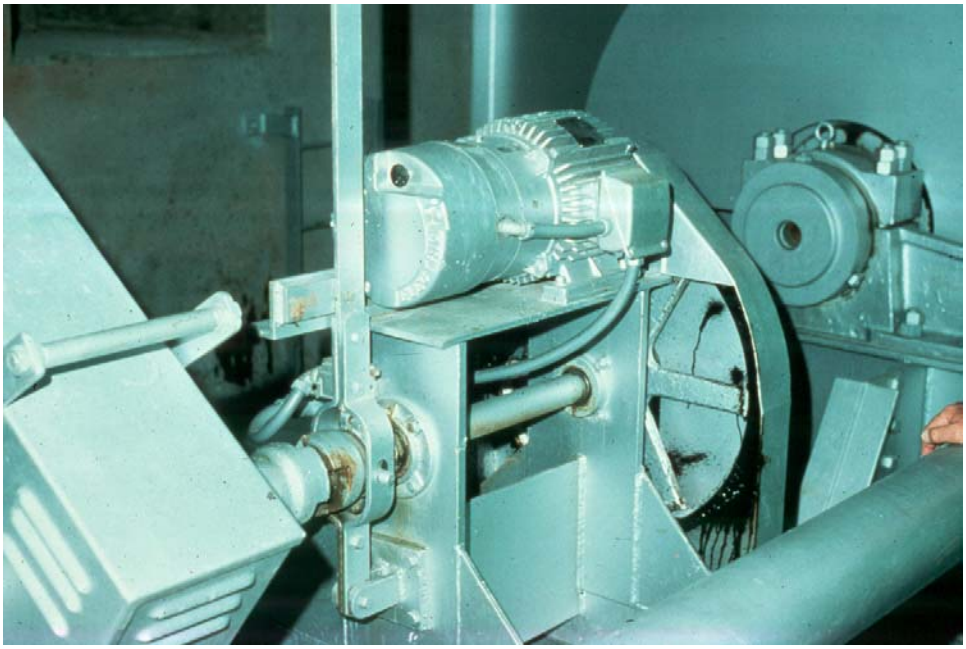


Figure 12.2.26 AC Emergency Motor

Air Buffers and Shock Absorbers

Air buffers and shock absorbers are located between the span and the pier at points where impact may occur between the two (see Figures 12.2.27 and 12.2.28). A

cross section of the buffer shows the air chamber and seals on the piston. As the span lowers, the rod is pushed in, causing the air inside to be compressed (see Figure 12.2.29). A pressure relief valve allows the air to escape beyond the pressure setting. Force is required to build-up and keep the pressure of the air at the movement of the span for a "soft" touchdown on the bearings. Shock absorbers provide the same purpose as the air buffers. However, they are completely self-contained and, therefore, require very little maintenance.



Figure 12.2.27 Air Buffer



Figure 12.2.28 Shock Absorber

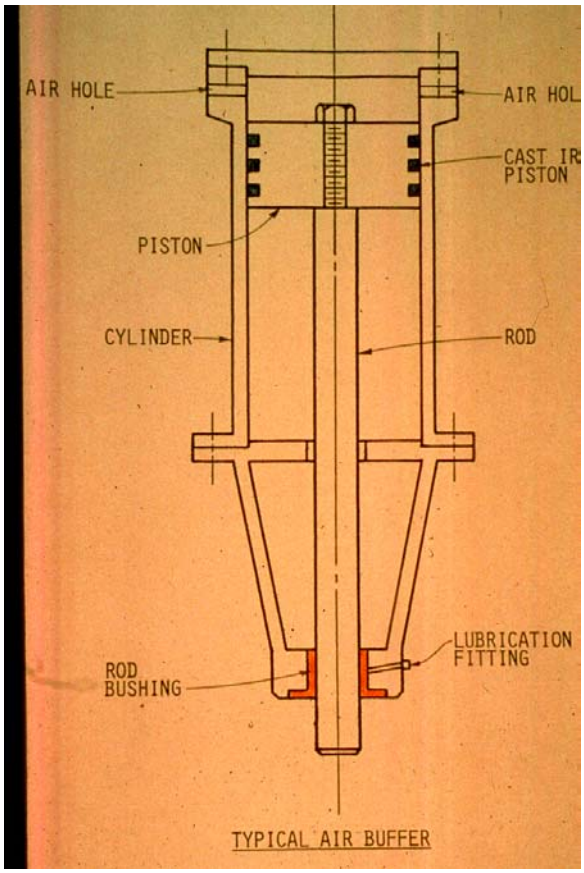


Figure 12.2.29 Typical Air Buffer Schematic

Span Locks

Span lock bars at the end of the span are driven when the span is fully closed to prevent movement under live load. Span locks may also be provided at other locations on the span to hold the span in an open position against strong winds or to prevent movement from an intermediate position. They can be driven either mechanically or hydraulically (see Figure 12.2.30 and 12.2.31).

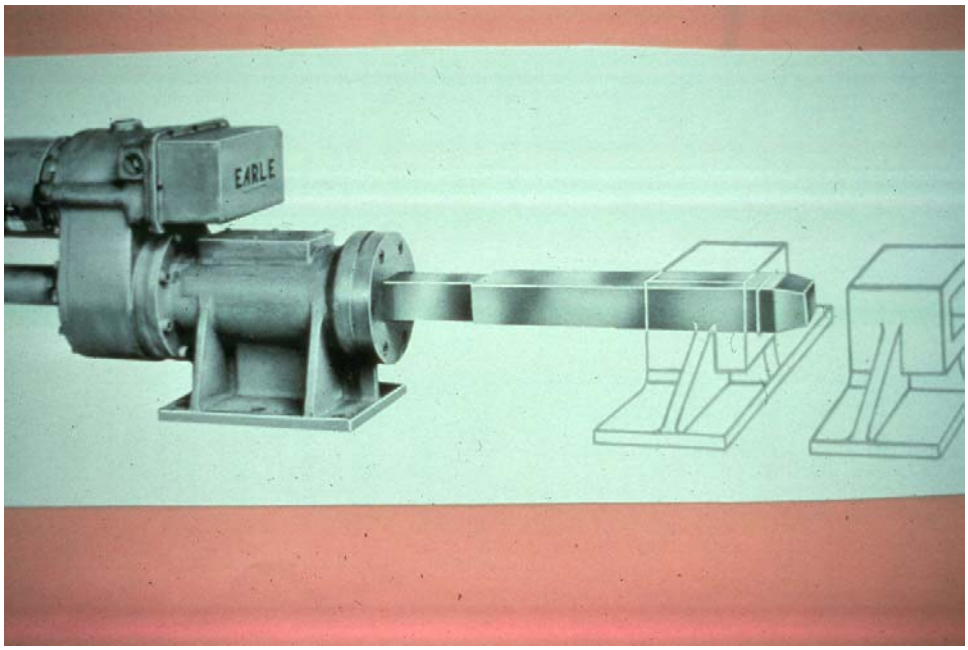


Figure 12.2.30 Typical Mechanically Operated Span Lock

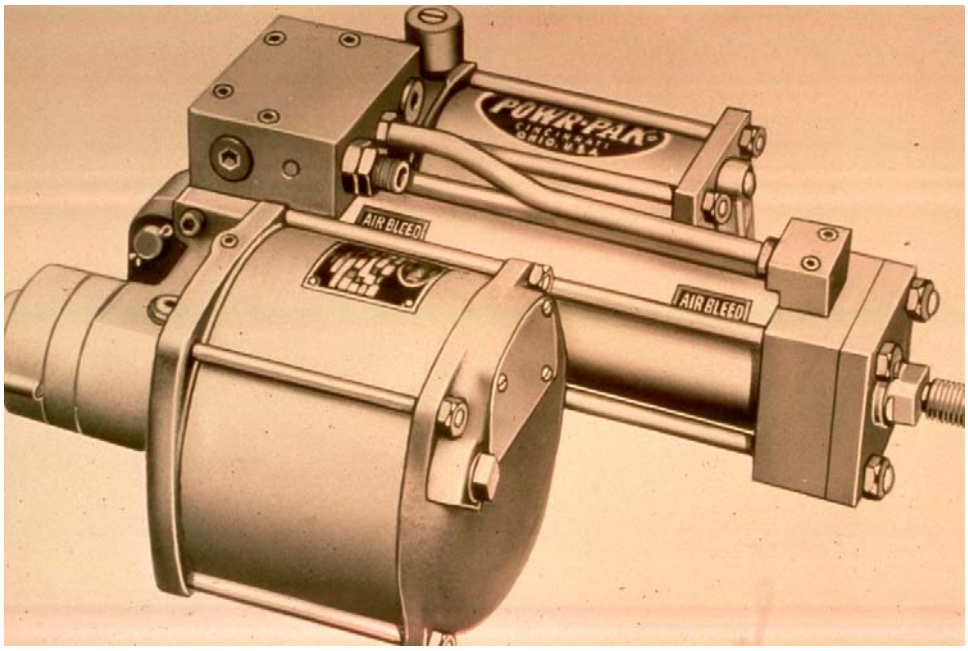


Figure 12.2.31 Hydraulic Cylinder that Drives Lock Bars

Counterweights

Adjustable quantities of counterweight blocks are provided in addition to the permanent counterweight, which is part of the structure so that adjustments may be made from time to time due to changes in conditions (see Figure 12.2.32 and 12.2.33). A movable span is designed to function in a balanced condition, and serious unbalanced conditions will cause overstress or even failure of the mechanical or structural elements.



Figure 12.2.32 Concrete Counterweight on a Single-Leaf Bascule Bridge



Figure 12.2.33 Concrete Counterweight on a Vertical Lift Bridge

Live Load Shoes and Strike Plates

Live load shoes and strike plates between the movable and fixed portions of the bridge are designed to bear most or all of the live load when the bridge is carrying traffic (see Figure 12.2.34).

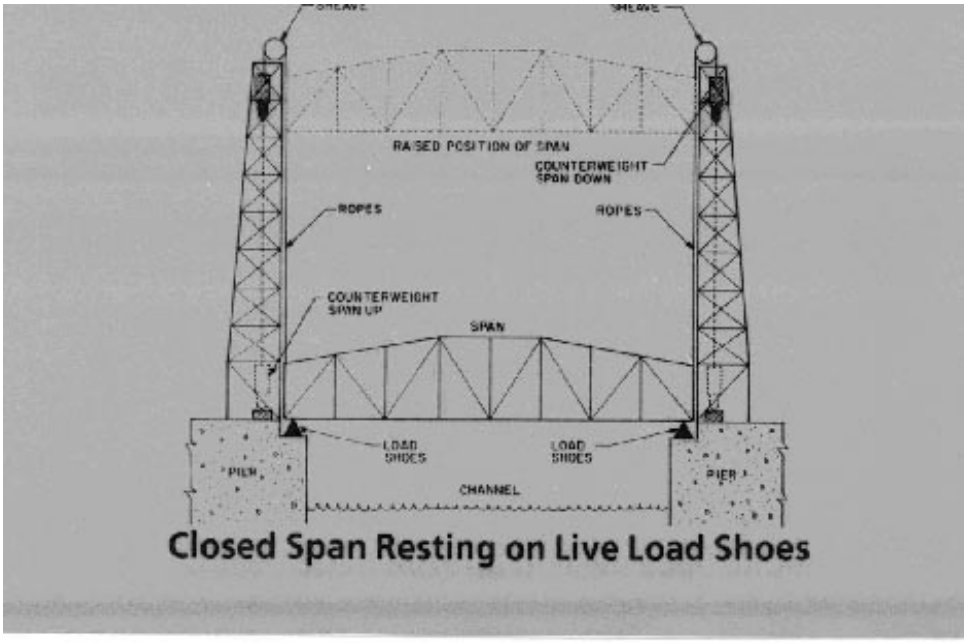


Figure 12.2.34 Closed Span Resting on Live Load Shoes

Traffic Barriers

Traffic barriers are heavyduty movable gates or posts that are designed to prevent a vehicle from plunging from the roadway into the draw or into the pit below the bridge (see Figure 12.2.35. Their operation is important for public safety. They are used mainly in situations where a large opening exists between the approach span and the movable span when it is open.



Figure 12.2.35 Traffic Barrier

12.2.6 Swing Bridge Special Elements

Swing bridges are designed utilizing the following special elements.

Pivot Bearings

In center-bearing types (with balance wheels), the axially loaded thrust bearing is usually composed of spherical discs, attached to top and bottom bases, enclosed in an oil box to provide lubrication and prevent contamination (see Figure 12.2.36). In rim-bearing types, the pivot bearing is also enclosed but will be radial loaded, maintaining the position of the pivot shaft or king pin.



Figure 12.2.36 Center Pivot Bearing

Balance Wheels

On center-bearing types only, non-tapered balance wheels bear on the circular rail concentric to the pivot bearing only when the span is subjected to unbalanced loading conditions (see Figure 12.2.37). At other times, when the span is not subjected to unbalanced loads, a gap should be found between each wheel and the rail.



Figure 12.2.37 Balance Wheel in-place over Circular Rack

Rim-Bearing Rollers

Usually tapered to allow for the differential rolling distance between the inside and outside circumferences of the rail circle, rim-bearing rollers should bear at all times.

Wedges

End wedges are used to raise the ends of the span and support live load under traffic (see Figure 12.2.38). The end wedge bearings are under all four corners of the span. Center wedges are used to stabilize the center of the span and to prevent the center bearing from supporting live load. Wedges may be actuated by machinery and linkage, which connects wedges to actuate together, or each wedge may have its own actuator (see Figure 12.2.39).

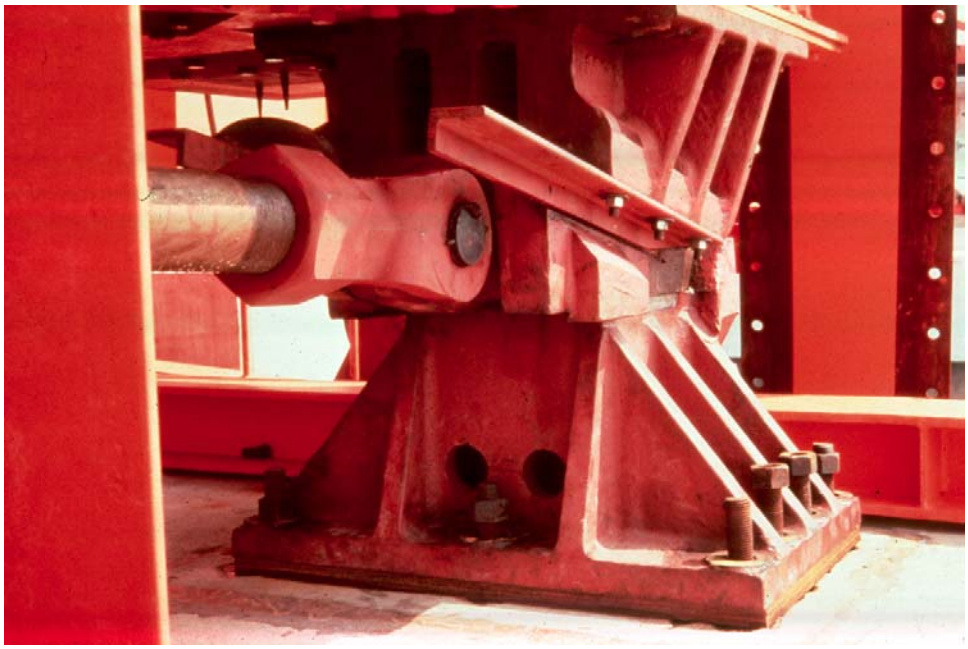


Figure 12.2.38 End Wedge

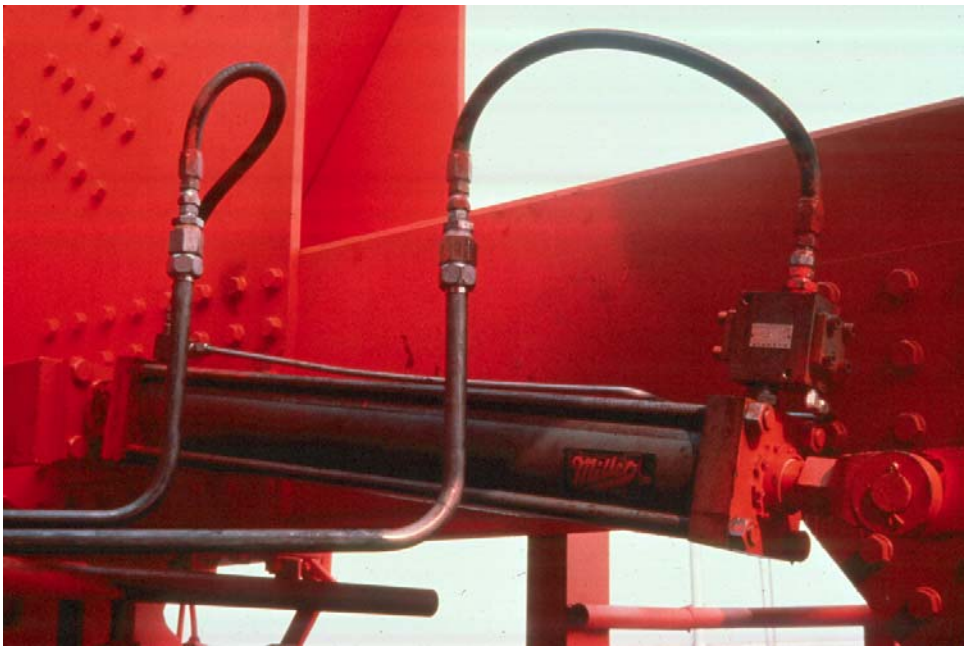


Figure 12.2.39 Hydraulic Cylinder Actuator

End Latches

Located at the center of one or both rest piers, end latches generally consist of a guided tongue with roller mounted on the movable span that occupies a pocket mounted on the rest pier when the span is in the closed position. To open the span, the tongue is lifted until it clears the pocket at the time the wedges are withdrawn (see Figure 12.2.40). As the span is swung open, the latch tongue is allowed to lower or fall into a position in which the roller may follow along a rail or track mounted on the pier. When closing, the tongue rolls along the rail or track and up a ramp which leads to the end latch pocket where the tongue is allowed to drop to center the span.

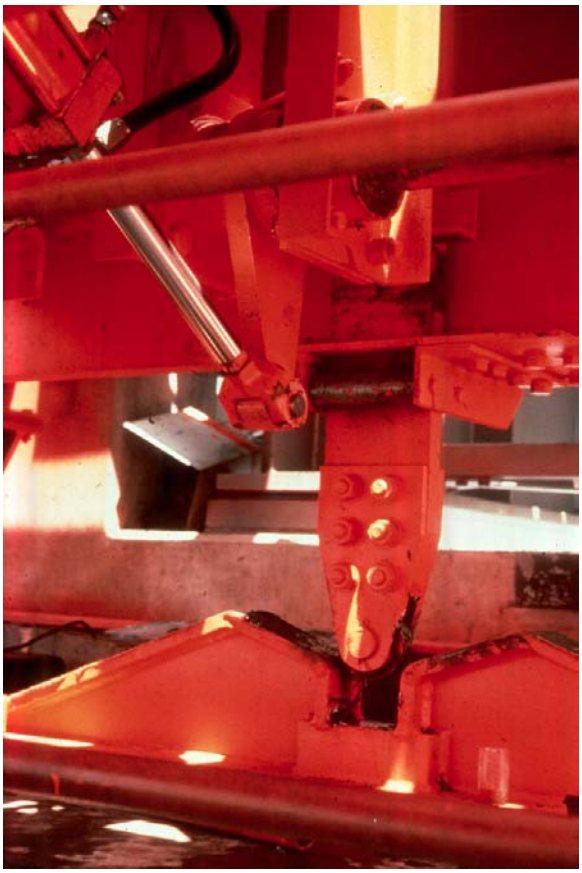


Figure 12.2.40 End Wedges Withdrawn and End Latch Lifted so the Roller on the Bottom can Roll up the Slope on the Inside

12.2.7 Bascule Bridge Special Elements

Bascule bridges utilize the following elements peculiar to their design.

Rolling Lift Tread and Track Castings

Rolling lift tread and track castings are rolling surfaces which support the bascule leaves as they roll open or closed (see Figure 12.2.41). Tread sockets and track teeth prevent transverse and lateral movement of the span due to unbalanced conditions, such as wind, during operation and especially when held in the open position.

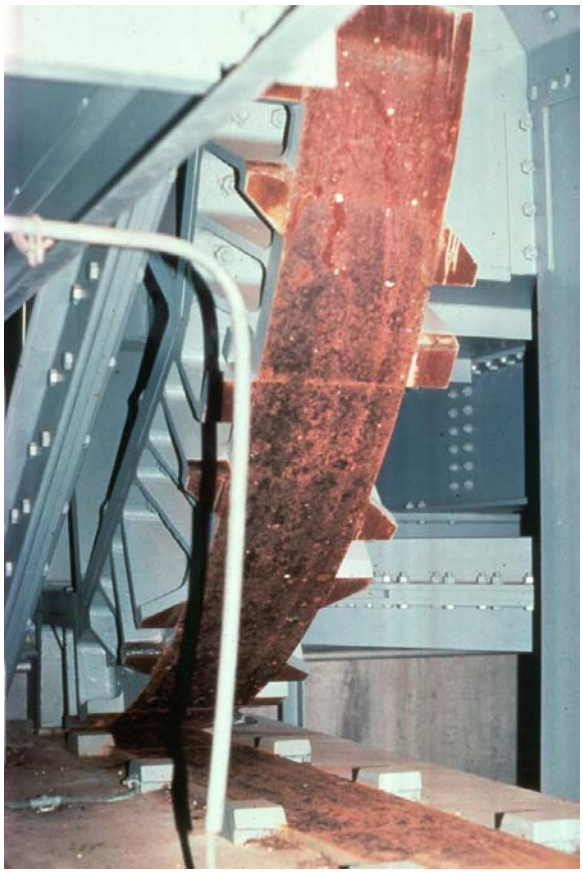


Figure 12.2.41 Circular Lift Tread and Track Castings

Racks and Pinions

- Rolling lift rack and pinion - The driving pinion engages the rack teeth at the centerline of the roll (see Figure 12.2.42).

- Trunnion rack and pinion - The circular rack casting are attached in the plane of the truss (or girder) in front of the counterweight (see Figures 12.2.43 and 12.2.44).

The drive pinions are overhung in order to engage the rack teeth. A cover is placed over the pinions for safety and to keep debris from falling on it.

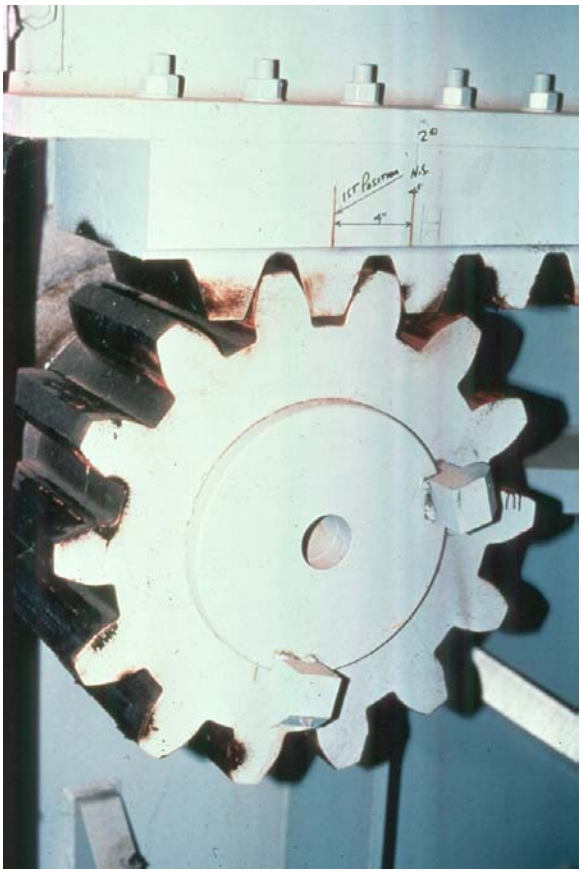


Figure 12.2.42 Rack Casting and Pinion

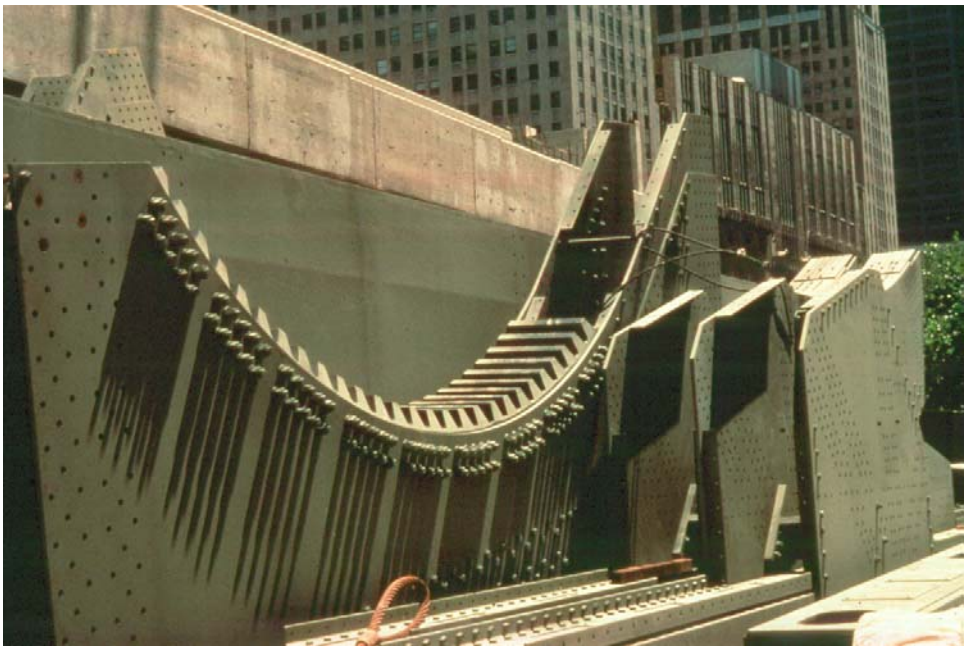


Figure 12.2.43 Rack Casting Ready for Fabrication

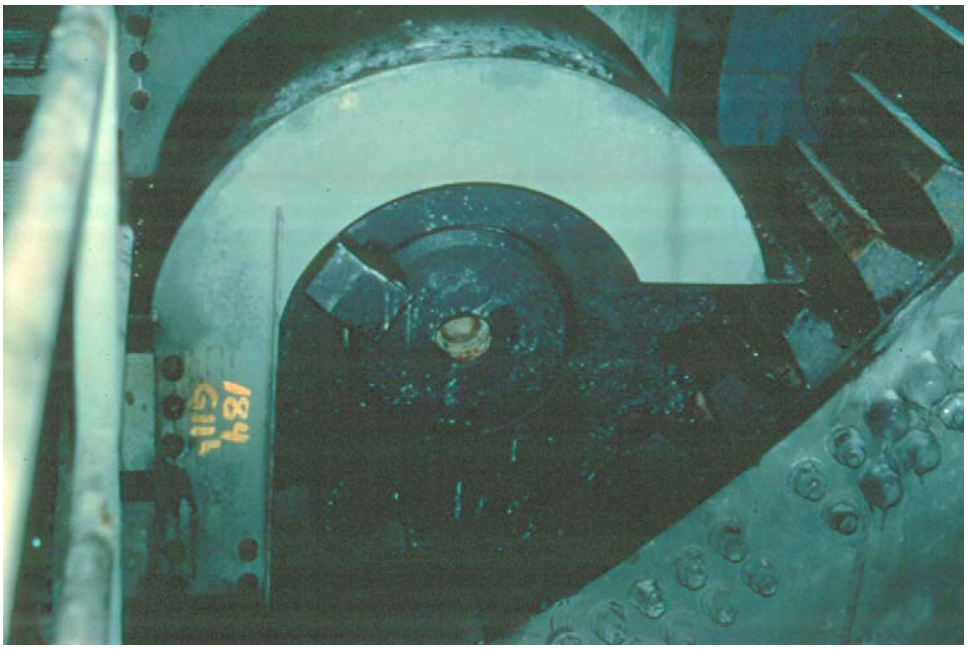


Figure 12.2.44 Drive Pinion

Trunnions and Trunnion Bearings

Trunnions and trunnion bearings (see Figure 12.2.45), which are large pivot pins or shafts and their bearings, entirely support the leaf as it rotates during operation as well as supporting dead load when the bridge is closed. Some designs, which do not provide separate live load supports, also require the trunnions to carry live load (see Figure 12.2.46).

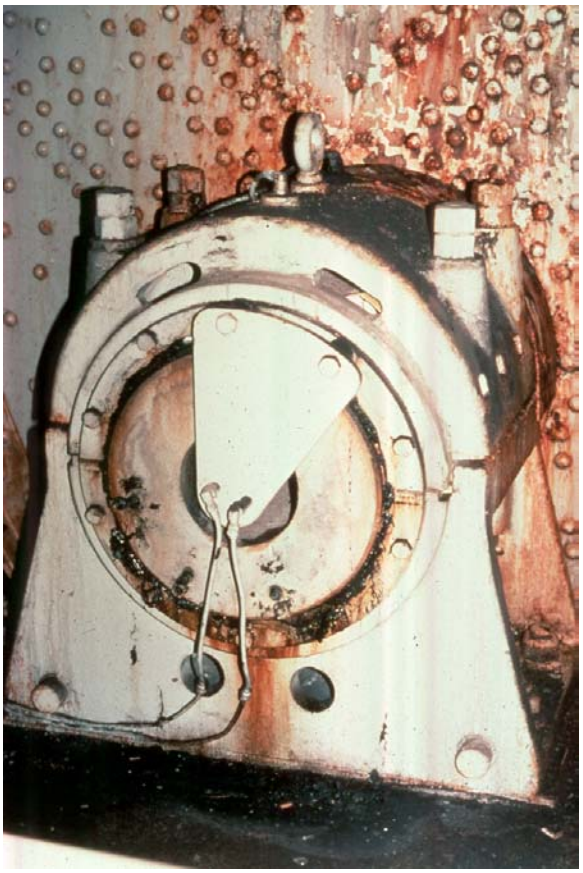


Figure 12.2.45 Trunnion Bearing

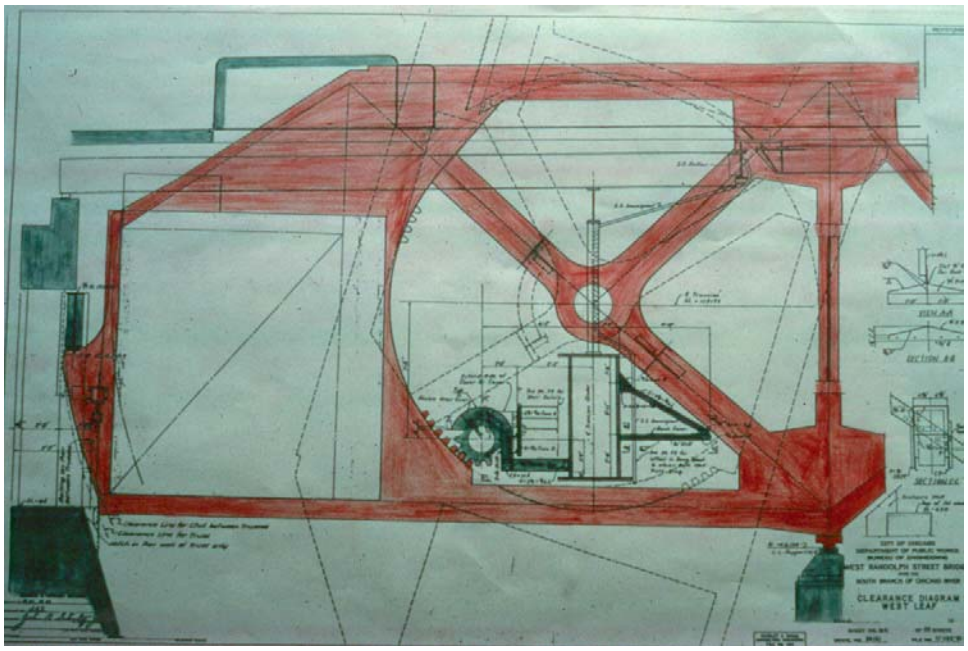


Figure 12.2.46 Trunnion Design Drawing

Hopkins Frame

A Hopkins frame machinery arrangement is provided on some trunnion bascule bridges. The main drive pinion locations are established in relationship to their

circular racks by a pivot point on the pier and pinned links attached to the trunnions.

Tail (Rear) Locks

Located at the rear of the bascule girder on the pier, tail locks prevent inadvertent opening of the span under traffic or under a counterweight-heavy condition should the brakes fail or be released (see Figure 12.2.47).

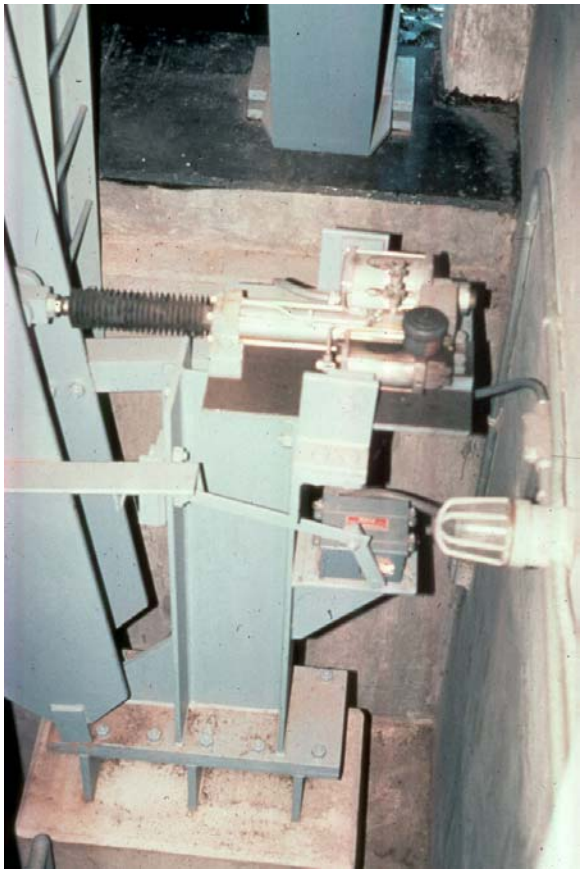


Figure 12.2.47 Rear Lock Assembly

Center Locks

Center locks are provided to transfer shear load from one leaf to the other when the bridge is under traffic. Center locks may consist of a driven bar or jaw from one leaf engaging a socket on the other leaf, or may be a meshing fixed jaw and diaphragm arrangement with no moving parts (see Figure 12.2.48). Without the center lock engaged, a double-leaf bascule functions as a cantilevered span, experiencing four times the bending moment, with proportional increases in stresses, at the pier.

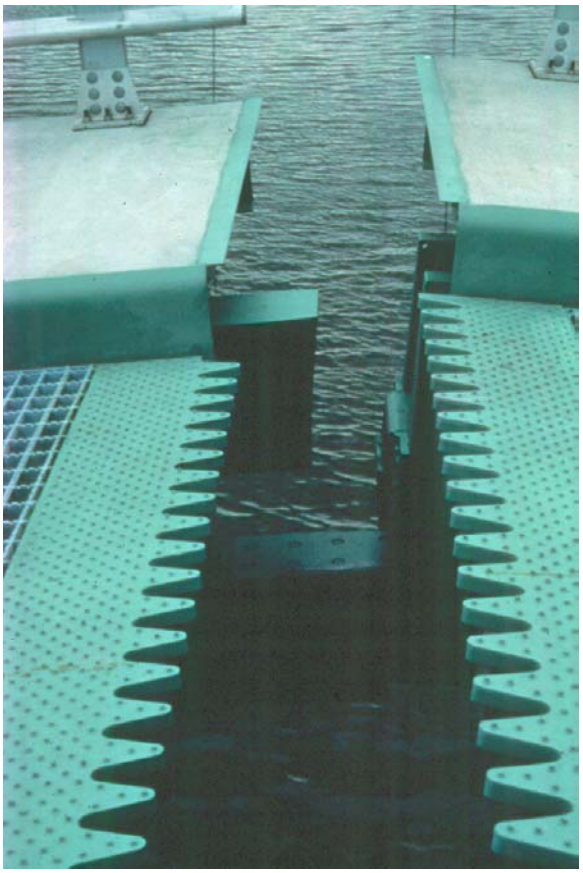


Figure 12.2.48 Center Lock Jaws

Transverse Locks

In twin bascule bridges that are split longitudinally to allow flexibility during construction, repair, or rehabilitation, transverse locks between the inside girders are used to keep the pairs together during operation (see Figure 12.2.49). These are usually operated manually, as they are not normally used for long periods of time.



Figure 12.2.49 Transverse Locks on Underside can be Disengaged

12.2.8 Vertical Lift Bridge Special Elements

Vertical lift bridges may utilize the following elements peculiar to their design.

Wire Ropes and Sockets

Wire ropes and sockets include up-haul and down-haul operating ropes and counterweight ropes (see Figure 12.2.50 and 12.2.51). Ropes consist of individual wires twisted into several strands that are wound about a steel core. Fittings secure the ends of the rope and allow adjustments to be made.

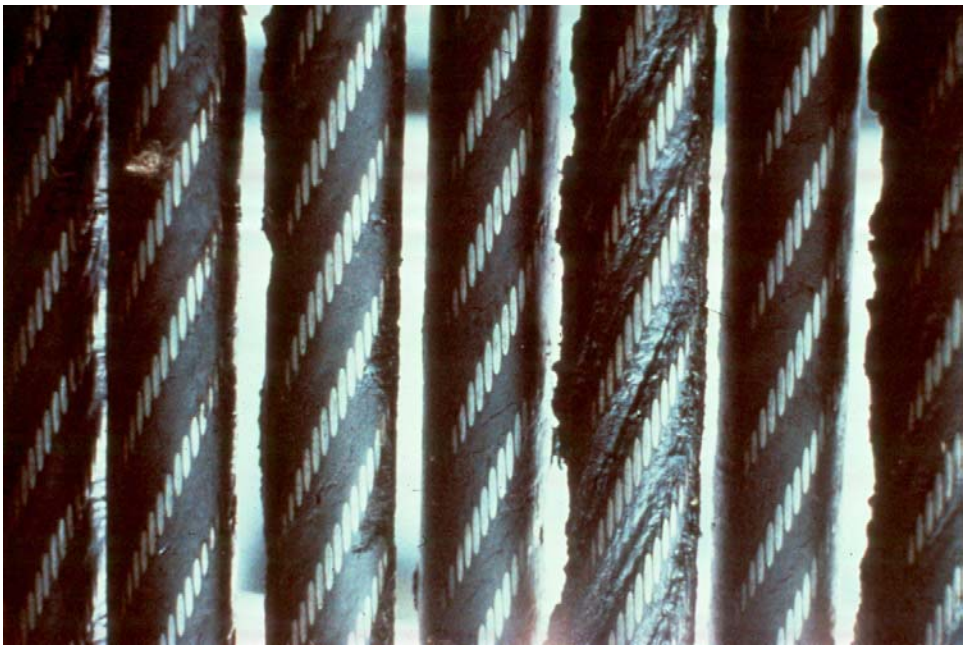


Figure 12.2.50 Wire Rope

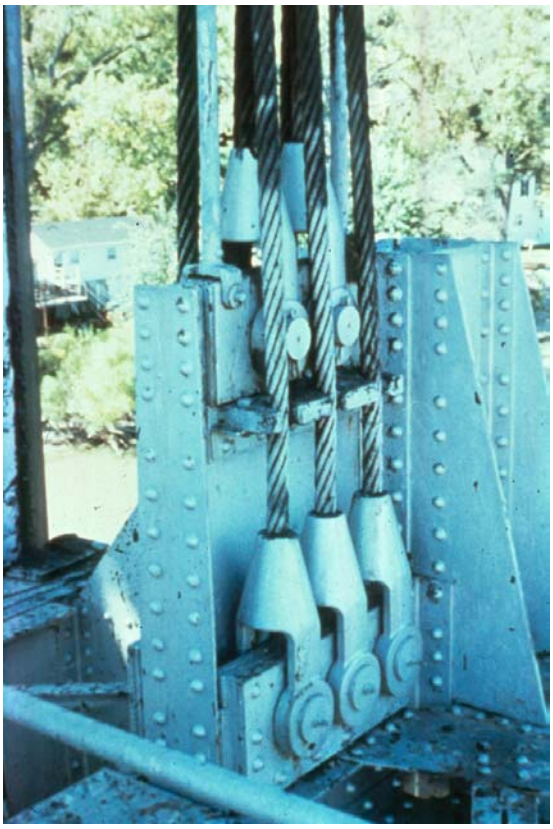


Figure 12.2.51 Wire Rope Sockets and Fittings

Drums, Pulleys, and Sheaves

Drums are used to wind a rope several times around to extend or retract it (see Figure 12.2.52). Pulleys and sheaves change the direction of the rope or guide it at intermediate points between ends of the rope.

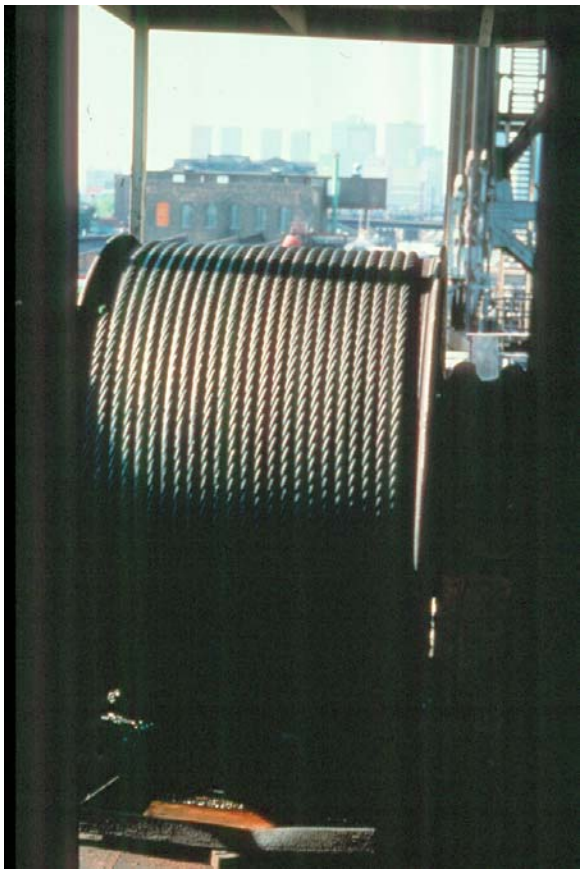


Figure 12.2.52 Drums Wind Up the Up-Haul (Lifting) Ropes as they Simultaneously Unwind the Down-Haul Ropes

Span and Counterweight Guides

Span and counterweight guides are located between tower and span or counterweight to prevent misalignment.

Balance Chains

Balance chains are provided to compensate for the weight of counterweight rope that travels from the span side to the counterweight side of the sheaves at the top of the tower as the span is raised. Weight of chain is removed from the counterweight and is supported by the tower as rope weight is increased on the counterweight side of the sheaves on the tower.

Span Leveling Devices

Whether mechanical or electrical, span leveling devices compensate and adjust the movement of the two ends of the span during operation to prevent unsynchronized movement.

12.2.9 Overview of Common Defects

Steel

Common defects that can occur to steel members of movable bridges include:

- Failure of the Paint System
- Pitting
- Surface Rust

- Section Loss
- Fatigue Cracking
- Collision Damage
- Overload Damage
- Heat Damage

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

Concrete

Common defects that occur to concrete members of movable bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion
- Stress corrosion

Refer to Topic 2.2 for a more detailed presentation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

12.2.10 Inspection Locations and Procedures - Safety

Movable Bridge Inspector Safety

It is imperative that all movable bridge inspectors coordinate their work with the Bridge Operator and emphasize the need for advance warning of a bridge opening. The Bridge Operator should not operate the bridge until being notified by all inspectors that they are ready for an opening. There are many ways that this can be accomplished, such as placing a warning note on the control console or opening the circuit breakers and locking the compartment to the equipment that they will be inspecting.

Inspection Considerations

Important considerations for a movable bridge inspector include observing and making comments in the inspection report on the following.

Public Safety

Public safety considerations include:

- Good visibility of roadway and sidewalk for Bridge Operator (see Figure 12.2.53).
- Adequate time delay on traffic signals for driver reaction.
- Adequate time delay before lowering gates.
- Interlock - all "gates down" before raising bridge (bypass available if traffic signals are on).
- Interlock - bridge must be closed before gates can be raised (bypass

available if locks are driven).

- Interlock - traffic signals do not turn off until all gates are fully raised (bypass available).
- Observe the location of the bridge opening in relation to the gates, traffic lights and bells, and determine whether approaching motorists can easily see them. Check their operation and physical condition to determine if they are functioning and well maintained. Recommend replacement when conditions warrant it.
- Unprotected approaches, such as both ends of a swing bridge and vertical lift bridge and the open end of a single-leaf bascule bridge, should preferably have positive resistance barriers across the roadway, with flashing red lights as provided on the gate arms (see Figure 12.2.54).
- High-speed roadways and curved approaches to a movable bridge should preferably have advanced warning lights (flashing yellow).



Figure 12.2.53 Operator's House with Clear View of Traffic Signals and Lane Gates



Figure 12.2.54 Traffic Control Gate

Navigational Safety

Navigational safety considerations include:

- Minimum underclearance designated on the permit drawing should be provided. Compliance with minimum channel width with any restriction on vertical clearance when span is open for navigation.
- All navigation lights should have a relay for backup light, and red span lights should not change to green until both leaves are fully open (see Figure 12.2.55).
- Marine radio communication (depends on the need) (see Figure 12.2.56).
- Operator should be able to automatically sound the emergency signal to navigation vessels if bridge cannot be opened.
- Navigation lights are very important and should be checked for broken lenses, deteriorated insulation of wiring and cable, and dry and clean interior.
- Underclearance gauges for closed bridges must be inspected for accuracy, visibility, and legibility.

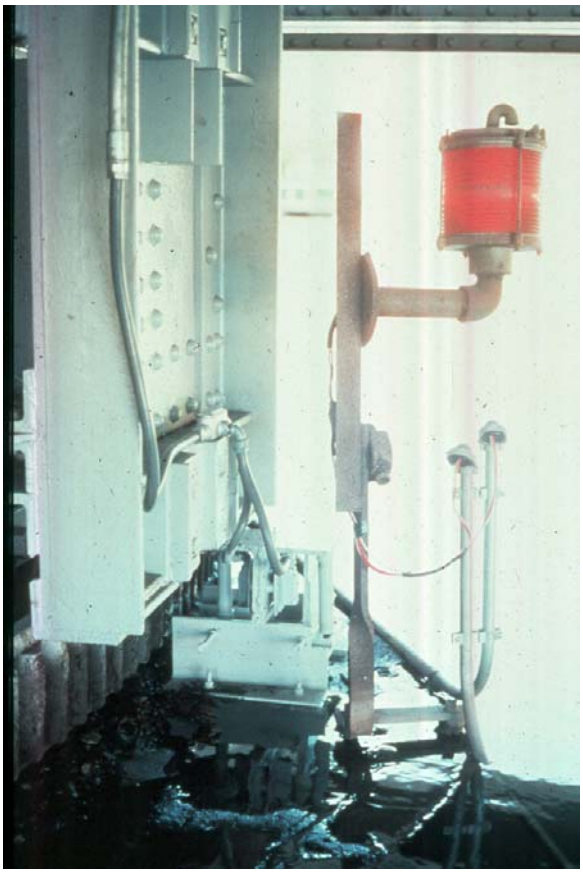


Figure 12.2.55 Navigational Light



Figure 12.2.56 Marine Two-Way Radio Console

Structure Safety

Structure safety considerations include:

- Structural ability to carry the anticipated loads.
- Presence of pressure relief valves on hydraulic power units to limit hydraulic forces applied to machinery and structure.
- Keeping horsepower applied to machinery and structure within design limits by limiting speed.

Dependable Operation

The movable bridge should be operated in the normal and emergency modes to check all interrelated interlocks and to be sure every component is operational.

12.2.11 Inspection Locations and Procedures of Movable Bridge Opening and Closing Sequences

Interlocking for Normal Operation

During normal operation, the inspector should verify that each interlock functions properly and can be bypassed (when provided). The controls for the traffic signals, traffic gates, center or rear locks, emergency brakes, and the bridge operation should be so interlocked that they can only be operated in the following sequences.

Opening Sequence

The bridge opening sequence should be as follows:

1. Activate traffic signals.
2. Lower oncoming gates and, when traffic has cleared, lower off-going gates. "All gates down" interlocked for withdrawing locks (bypass provided).
3. Press "raise" button if automatic operation is provided or, if manual operation is provided, proceed as follows:

    a. Withdraw locks - "Locks Withdrawn." Interlocked for bridge operation (no bypass).
    b. Release emergency brakes - no interlock provided. Warning buzzer sounds if brakes are not released when power is applied to motors to move bridge.
    c. Accelerate leaves to full speed.
    d. When advanced to nearly open position, decelerate leaves to slow speed and stop at nearly open position.
    e. At nearly open position, with reduced power, lower leaves to stop at fully open position.
    f. Set emergency brakes.

Closing Sequence

The bridge closing sequence should be as follows:

1. Press "lower" button if automatic operation is provided or, if manual operation is provided, proceed as follows:

a. Release emergency brakes.
b. Accelerate leaves to full speed.
c. For all types of bridges with lock bars:

(1) At advanced nearly closed position, decelerate leaves to slow speed. Leaves should stop at nearly closed position by action of the bridge limit switch.
(2) At nearly closed position with reduced power, lower leaves to stop at fully closed position.
(3) With machinery wound up (bascule bridges and counterweight heavy vertical lift bridges) or when span is fully closed (swing bridges and span heavy vertical lift bridges), set the brakes and drive lock bars.

d. For rolling lift bridges having jaw and diagram shear locks with no moving parts:

(1) At advanced nearly closed position, decelerate to slow speed. The jaw leaf should stop at the "locking position" (within the "window" to receive the diaphragms) by action of the bridge limit switch.
(2) At advance nearly closed position, decelerate to slow speed. The diaphragm leaf should stop in the "clear position" (where the lower jaw will clear the diaphragm) by action of the bridge limit switch.
(3) Foot switch must be depressed to provide reduced power from this point until both leaves are closed.
(4) Lower the diaphragm leaf to make "soft" contact with lower jaw.
(5) Close both leaves together with diaphragm castings against lower jaws.
(6) When leaves are fully closed, drive the rear locks. "Fully closed" interlock provided for rear lock operation (no bypass).
(7) Set emergency brakes with reduced power applied to motors to hold machinery wound up.

2. Deactivate automatic traffic control, or manually raise gates:

a. All gates raise, off-going gates should start up before oncoming gates raise.
b. Warning signals and red lights should not turn off until all gates are raised, even if the power switch is turned "off" (bypass should be provided), after which the green traffic lights are turned "on."

Bypass Note: All bypass switches should have handles that are spring returned to "off." When the switch is turned to bypass momentarily, a holding relay should hold the bypass activated until power is removed from the controls or the switch is turned to cancel bypass. These circuits should be provided in order to prevent inadvertent use of any bypass. Until a malfunction is corrected, the operator must therefore initiate the use of any

bypass switch that is needed every time the bridge is operated.

12.2.12 Inspection Locations and Procedures for the Control House

The operator is responsible for public and navigational safety during operation and, together with maintenance personnel, should be most familiar with any known structural or operational defects. Operational and maintenance log books should be kept in the control house for reference. The resources within the control house can therefore provide a great deal of general information, through the knowledge of its personnel and the records stored there. The position of the control house should provide the best general view of the bridge itself.

Inspection of the control house should include:

- Consult with the bridge operators to ascertain whether there are any changes from the normal operation of the bridge.
- Note where the control panel is located in relation to roadway and waterway, and also whether the bridge operator has a good view of approaching boats, vehicles, and pedestrians (see Figure 12.2.57). Check operation of all closed circuit TV equipment, and evaluate its position for safe operation.
- Note whether the structure shows cracks, and determine whether it is windproof and insulated.
- If controls are in more than one location, note description of the other locations and include their condition as well as the information about the control house.
- Note whether all Coast Guard, Corps of Engineers, and local instructional bulletins are posted.
- Note whether alternate warning devices such as bullhorns, lanterns, flasher lights, or flags are available.
- Check for obvious hazardous operating conditions involving the safety of the operator and maintenance personnel.
- Check for any accumulations of debris, which may be readily combustible.
- Check controllers while bridge is opening and closing. Look for excess play and for sparking during operation.
- Note whether the submarine cables are kinked, hooked, or deteriorated, especially at the exposed area above or below the water. In tidal areas, check for marine and plant growth. Note if the ends of the cable have been protected from moisture.



Figure 12.2.57 Control Panel

12.2.13 Inspection Locations and Procedures for Structural Members

Defects, Damage, and Deterioration

Defects, damage, and deterioration, typically detrimental to all steel and concrete structures, must be noted during the inspection of all types of movable bridges. Most of the bridge structure defects and deterioration listed elsewhere as potential problems apply to movable spans also.

Fatigue

Fatigue can be a problem with movable bridges due to the reversal or the fluctuation of stresses as the spans open and close (see Figure 12.2.58). Any member or connection subject to such stress variations should be carefully inspected for fatigue failure.



Figure 12.2.58 Normal Operations can Cause Stress Reversals in Members, Leading to Fatigue

Counterweights and Attachments

Inspection of counterweights and attachments should include:

- Inspect the counterweights to determine if they are sound and are properly affixed to the structure. Also check temporary supports for the counterweights that are to be used during bridge repair and determine their availability should such an occasion arise.
- Where steel members pass through or are embedded in the concrete check for any corrosion of the steel member and for rust stains on the concrete.
- Look for cracks and spalls in the concrete.
- Check for debris, birds, animals, and insect nests in the counterweight pockets.
- Where lift span counterweight ropes are balanced by chains (or other means), make sure the links hang freely, and check these devices along with slides, housings, and storage devices for deterioration and for adequacy of lubrication, where applicable (see Figure 12.2.59).
- Determine whether the bridge is balanced and whether extra balance blocks are available. A variation in the power demands on the motor, according to the span's position, is an indication of an unbalanced leaf or span. If the controls provide a "drift" position, it should be used to test the balance.
- Paint must be periodically removed from a lift span properly; otherwise, the counterweights will eventually be inadequate.
- Determine whether the counterweight pockets are properly drained. On

vertical lift bridges, be sure that the sheaves and their supports are well drained. Examine every portion of the bridge where water can collect. All pockets that are exposed to rain and snow should have a removable cover.



Figure 12.2.59 Counterweights on Vertical Lift Bridge

Piers

Inspection of piers on movable bridges should include:

- Check the pier protection system (see Figure 12.2.60).
- Take notice of any rocking of the piers when the leaf is lifted. This is an indicator of a serious deficiency and should be reported at once.
- Check the braces, bearings, and all housings for cracks, especially where stress risers would tend to occur.
- Inspect the concrete for cracks in areas where machinery bearing plates or braces are attached (see Figure 12.2.61). Note the tightness of bolts and the tightness of other fastening devices used.
- Survey the spans including towers to check both horizontal and vertical displacements. This will help to identify any foundation movements that have occurred.

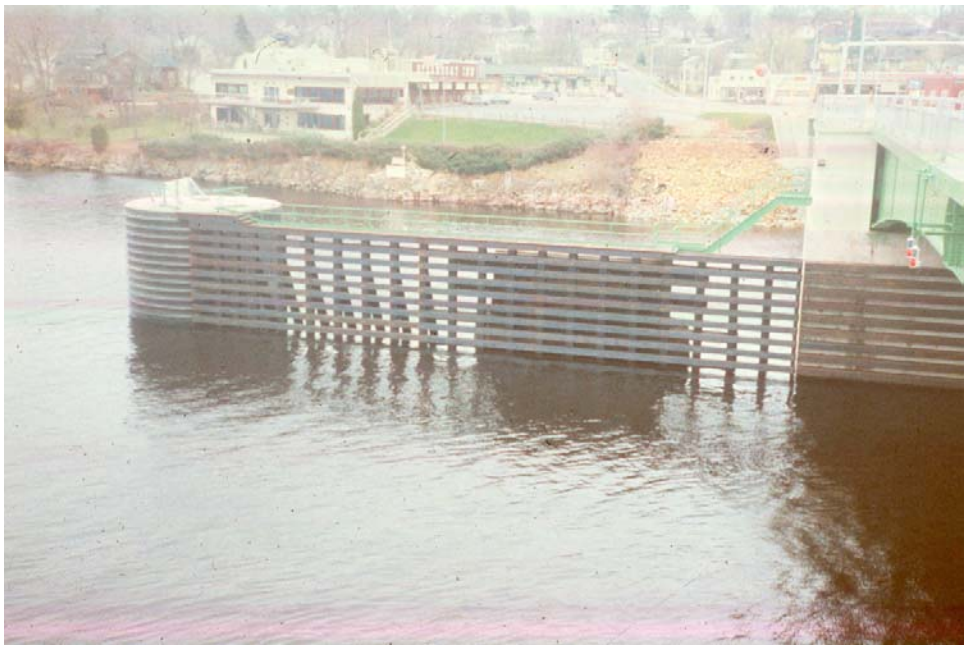


Figure 12.2.60 Pier Protection Systems – Dolphins and Fenders



Figure 12.2.61 Concrete Bearing Areas

Steel Grid Decks

Structural welds should be sound and the grid decks should have adequate skid resistance. Check the roadway surface for evenness of grade and for adequate clearance at the joints where the movable span meets the fixed span. For more information on steel grid decks, see Topic 5.3.

Concrete Decks

A solid concrete roadway is used over the pier areas (pivot or bascule pier) to keep water and debris from falling through onto the piers. Since the machinery room is usually under the concrete deck, check the ceiling for leaks or areas that allow debris and rust to fall on the machinery. For more information of concrete decks,

see section 5.2.

Other Structural Considerations

Other structural considerations include:

- On swing bridges, check the wedges and the outer bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of span or uplift when load comes upon the other span.
- Examine the live load bearings and wedges located under the trusses or girders at the pivot pier for proper fit alignment and amount of lift.
- On double-leafed bascule bridges, measure the differential vertical movement at the joint between the two leaves under heavy loads. On other types, check for this type of movement at deck joints (breaks in floor) between movable and fixed portions of the structure. This can indicate excessive wear on lock bars or shear lock members.
- Inspect the joint between the two leaves on double-leaf bascule bridges, or the joints between fixed and movable portions of the structure for adequate longitudinal clearance for change in temperature (thermal expansion).
- On bascule bridges, see if the front live load bearings fit snugly. Also observe the fit of tail locks at rear arm and of supports at outer end of single-leaf bridges.
- Inspect the fully open bumper blocks and the attaching bolts for cracks in the concrete bases.
- Examine the counterweight pit for water. Check the condition of the sump pump, the concrete for cracks, and the entire area for debris.
- See if the shear locks are worn. Measure the exterior dimensions of the lock bars or diaphragm casting and the interior dimensions of sockets or space between jaws to determine the amount of clearance (wear). Excessive movement should be reported and investigated further.
- On rolling lift bascule bridges, check the segmental and track castings and their respective supporting track girders (if used) for wear on sides of track teeth due to movement of sockets on segmental castings. Compare all wear patterns for indications of movement of the leaves. Check for cracking at the fillet of the angles forming the flanges of the segmental and track girders, cracking in the flanges opposite joints in the castings, and cracking of the concrete under the track. Inspect rack support for lateral movement when bridge is in motion.
- On multi-trunnion (Strauss) bascule bridges, check the strut connecting the counterweight trunnion to the counterweight for fatigue cracks. On several bridges, cracking has been noted in the web and lower flanges near the gusset connection at the end nearer the counterweights. The crack would be most noticeable when the span is opened.

12.2.14 Inspection Locations and Procedures for Machinery Members

Mechanical, electrical, and hydraulic equipment includes specialized areas, which are beyond the scope of this manual. Since operating equipment is the heart of the movable bridge, it is recommended that expert assistance be obtained when conducting an inspection of movable spans. It should be noted that in many cases, the owners of these movable bridges follow excellent programs of inspection, maintenance, and repair. However, there is always the possibility that some important feature may have been overlooked.

Trial Openings

Conduct trial openings as necessary to insure proper operational functioning and that the movable span is properly balanced. Trial openings should be specifically for inspection. During the trial openings, the safety of the inspection personnel should be kept in mind.

Machinery Inspection Considerations

On all movable structures, the machinery is so important that considerable time should be devoted to its inspection. The items covered and termed as machinery include all motors, brakes, gears, tracks, shafts, couplings, bearings, locks, linkages, over-speed controls, and any other integral part that transmits the necessary mechanical power to operate the movable portion of the bridge. Machinery should be inspected not only for its current condition, but operational and maintenance procedures and characteristics of operation should also be analyzed. The items listed below and items similar to them should be inspected and analyzed by a machinery or movable bridge specialist. Refer to FHWA-IP-77-10, Bridge Inspector's Manual for Movable Bridges and the AASHTO Movable Bridge Inspection, Evaluation and Maintenance Manual, for further information on inspecting these items. The FHWA-IP-77-10 manual is published by the Federal Highway Administration (FHWA), but is currently out of print.

Operation and General System Condition

Observe the general condition of the machinery as a whole, and its performance during operation. Check for smoothness of operation, and note any abnormal performance of components. Noise and vibration should also be noted, and the source determined. Unsafe or detrimental procedures followed by the operator should be noted to prevent injury to the public or to personnel, or damage to the equipment. The condition of the paint system should also be noted.

Maintenance Procedures

An evaluation of maintenance procedures in light of design details for the equipment should be done. Application methods and frequency of lubrication should be checked in the maintenance logbook, if available. General appearance of existing applied lubricant should be noted.

Mechanical Elements

The following are condensed guidelines for various mechanical elements.

Open Gearing

Check open gearing for tooth condition and alignment including over- and under-engagement. The pitch lines should match. Excessive or abnormal wear should be noted. Inspect the teeth, spokes, and hub for cracks. Observe and note the general appearance of the applied lubricants on open gearing. If the lubricant has been contaminated, especially with sand or other gritty material, it should be removed and new lubricant applied. If there is a way to prevent future contamination, it should be recommended in the inspector's comments in the report. Check the teeth of all gears for wear, cleanliness, corrosion, and for proper alignment.

Speed Reducers Including Differentials

The exterior of the housing and mountings should be examined for cracks and damage (see Figure 12.2.62 and 12.2.63). Check bolts for tightness and note any corrosion. The interior of the housing should be inspected for condensation and corrosion. Check the condition of gears. Watch for abnormal shaft movement during operation, indicating bearing and seal wear. Oil levels and condition of lubricant should be checked periodically through the use of sampling and analysis techniques. Circulating pumps and lubricating lines should be observed for proper operation. Abnormal noise should be noted.



Figure 12.2.62 Cracked Speed Reducer Housing

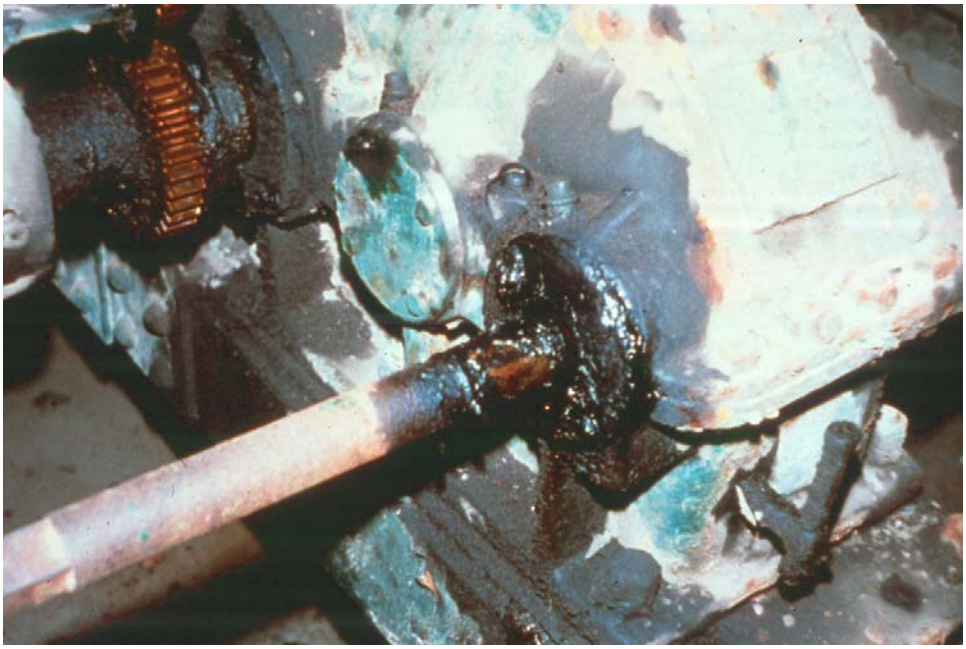


Figure 12.2.63 Leaking Speed Reducer

Shafts and Couplings

Shafts should be examined for damage, twisting, and strain. Cracks, if suspected, may be detected using dye penetrant (see Figure 12.2.64). Misalignment with other parts of the machinery system should be noted. Cracks in shafts should be measured and the exact location recorded. Consideration should be given to replacement of the shaft. Other shafts should be examined in the same locations as they were probably made from the same material and fabricated to the same details. They have also been exposed to the same magnitude and frequency of loading. Coupling hubs, housings, and bolts should be checked for condition. Seals and gaskets should be inspected for leaks. Internal inspection of couplings is warranted if problems are suspected and can be used to determine tooth wear in gear couplings.

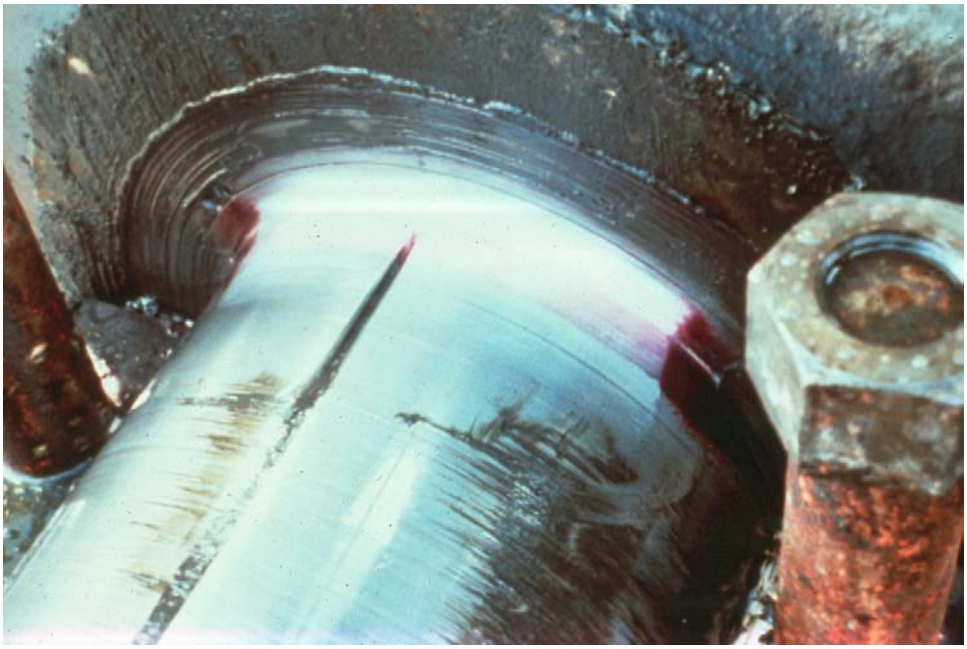


Figure 12.2.64 Hairline Crack Revealed on Shaft from Dye Penetrant Test

Bearings

Bearing housings, pedestals, and supports should be examined for external condition. Any cracks should be noted. Bolts in housings and those used for anchors should be checked for tightness, damage, and corrosion. Apparent lubrication characteristics should be noted. Grinding noises can be caused by dry bearings (unlubricated) (see Figure 12.2.65). In sleeve bearings, the bushings should be inspected for damage and excessive wear. Evidence of seal damage in anti-friction bearings should be noted. Unusual noise should be investigated. Check the trunnion bearings for excessive wear, lateral slip, and loose bolts.

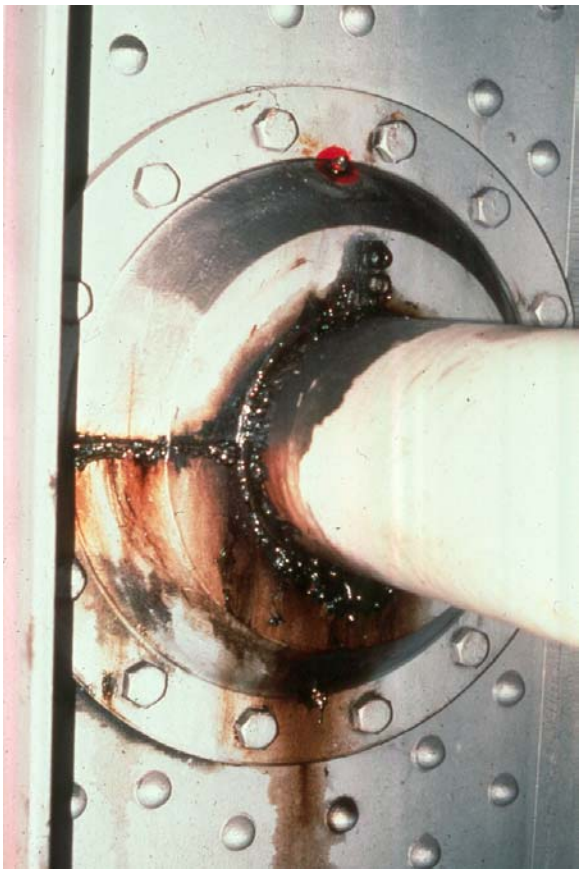


Figure 12.2.65 Dry Bearing

Brakes

Inspect all braking devices for proper setting of braking torque and for complete release of the brakes when actuated. On shoe brakes, check drums and shoes for wear, damage, and corrosion, for misalignment of shoes with drums, and for clearance when released. Determine if worn linings need replaced. Check for proper actuation without leakage by actuators. Linkages and hand releases should be free but not sloppy. On enclosed hydraulic disc brakes, make certain there is proper actuation without leakage at connections or seals. Check the brakes, limit switches, and stops (cylinders and others) for excessive wear and slip movement. Note whether the cushion cylinder ram sticks or inserts too easily. The brake limit switches should be inspected for proper setting. Observe the surface of the brake drum for indications of contact with the brake shoes. Check the pressure developed by each disc brake power unit to be sure the brakes are releasing. Also check the manual release on all of the brakes.

Drives - Electric Motors

Check the housing and mountings for damage, corrosion, and fastener condition. Inspect bearings for lubrication and note indications of wear (movement) and seal leakage at shaft extensions.

Drives - Hydraulic Equipment

Look for any leakage at connections and seals. Note any corrosion on the cylinder rods. Listen to motors and pumps, and note any unusual noise. Power units should be checked to make sure all components are functioning and that pressures are properly adjusted. Fluid should be sampled periodically and examined for contamination and wear metal. Check all main hydraulic power units for charge pressure setting and maximum pressure that can be developed by the unit. All

filters must be checked routinely and replaced as needed. The level of fluid in the vertical reservoir should also by checked.

Auxiliary Drives

Check emergency generators for operation and readiness. There should be no oil leaks or abnormal noises. Mechanical service specialists and electrical inspectors are required for more thorough inspections. Auxiliary motors and hand operators, with their clutches and other transmission components, should be checked for adjustment and readiness to perform when called upon.

Drives - Internal Combustion Engines

The detailed inspection of internal combustion engines should be made by mechanical engine specialists. Inspection should also include but not be limited to the checking of the following conditions:

- If a belt drive is used, look for any wear or slippage. Note the condition of all belts and the need for replacement, if any.
- If a friction drive is used, all bracing and bearings should be tight.
- If a liquid coupling is used, make sure that the proper quantity of fluid is used. Look for leaks.

Locks

Examine the center locks and tail locks (if used) on double-leafed bascule spans, and the end locks on single-leaf bascule bridges, swing bridges, and vertical lift bridges. Note whether there is excessive deflection at these joints or vibration on the bridge. Inspect the locks for fit and for movement of the span or leaf (or leaves). Check lubrication and for loose bolts. The lock housing and its braces should have no noticeable movement or misalignment. The paint adjacent to the locks will have signs of paint loss or wear if there is movement. Check lock bars, movable posts, linkages, sockets, bushings, and supports for damage, cracks, wear, and corrosion.

Check all rear locks in the withdrawn position for clearance from the path of the moving leaf as it opens and for full engagement when the leaf is closed. The gap, if any, should be measured between the lock plate and the moving leaf bearing plate. Check each rear lock hydraulic drive unit for leakage of oil and operation for correct length of movement of the lock.

On bascule bridges, see if the front live load bearings fit snugly. Also observe the fit of tail locks at the rear arm and of supports at the outer end of single-leaf bridges.

Actuators should be examined for operational characteristics, including leakage if hydraulic. The quantity and quality of lubricant should be noted. Check for alignment, and analyze the type of wear that is occurring. Note condition of movable operators.

Live Load Shoes and Strike Plates

The fasteners and structure should be inspected for defects and corrosion. Contact surface conditions should be noted. Check for alignment and movement under load.

Air Buffer Cylinders and Shock Absorbers

Note indications of lack of pressure or stickiness during operation. Check piston rod alignment with strike plate. Note the condition of the rod and housing. There should be no hydraulic leakage. Check the air filter and function of any pressure reading or adjusting devices and the operating pressure, if possible. The air buffers

should have freedom of movement and development of pressure when closing. Inspect the fully open bumper blocks and the attaching bolts for cracks in the concrete bases.

Machinery Frames, Supports, and Foundations

There should be no cracking in steel or concrete. Note corrosion and damage. Check for deflection and movement under load. The linkages and pin connections should have proper adjustment and functional condition. Check motor mounting brackets to ensure secure mounting.

Fasteners

Inspect the fasteners for corrosion, loss of section, and tightness.

Wedges

Check the wedges and the outer bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of span or uplift when load comes upon the other span.

Examine the live load bearings and wedges located under the trusses or girders at the pivot pier for proper fit alignment and amount of lift.

Special Machinery for Swing Bridges

Check center bearings for proper and adequate lubrication, oil leaks, and noise. Examine the housing for cracking, pitting, fit of joints, and note indications of span translation (irregular rotation) at racks and track. Measure for proper clearance of balance wheels above track. The tracks and balance wheels should be free of wear, pitting, and cracking. Check for proper and adequate lubrication at all lubrication points.

Balance characteristics should be noted as indicated by loads taken by balance wheels, and by drag on the rest pier rail.

Check the rim bearing for wear on tracks and rollers, particularly at rest positions where the bridge is carrying traffic. Examine the center pivots and guide rings for proper fit, and for wear, pitting, and cracking. Check for proper and adequate lubrication at all lubrication points.

The center (live load) wedges located under the trusses or girders at the pivot pier must be examined for proper fit (no lifting) and alignment. Check end wedges and bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of the span or uplift when live load crosses the other span. The end lift jacks, shoes, and all linkages must be inspected for wear, proper bearing under load, and proper adjustment.

Note the condition of end latches, including any modification that adversely affects their functional design.

Special Machinery for Bascule Bridges

On rolling lift bascule bridges, check the segmental and track castings and their respective supporting track girders (if used) for wear on the sides of track teeth due to movement of sockets on segmental castings. The trunnion assemblies must be inspected for deflection, buckling, lateral slip, and loose bolts. The trunnions themselves should have no corrosion, pitting, or cracking, particularly at stress risers. Check the balance of each leaf. Compare all wear patterns for indications of movement of the leaves. Check for cracking at the fillet of the angles forming the flanges of the segmental and track girders, cracking in the flanges opposite joints in the castings, and cracking of the concrete under the track. Inspect rack support for lateral movement when bridge is in motion.

Check trunnion bearings for lubrication of the full width of the bearing. Verify that extreme pressure (EP) lubrication oil of the proper grade is used.

Special Machinery for Vertical Lift Bridges

The condition of wire ropes and sockets, including wire rope lubrication, is important. Look for flattening or fraying of the strands and deterioration between them. This is reason for replacement. Similarly, check the up-haul and down-haul ropes to see if they are winding and unwinding properly on the drums. The need for any tension adjustments in up-haul and down-haul ropes should be noted. Determine whether ropes have freedom of movement and are running properly in sheave grooves. Look for any obstructions to prevent movement of the ropes through the pulley system, and check the supports on span drive type bridges. Check rope guides for alignment, proper fit, free movement, wear, and structural integrity of the longitudinal and transverse grooved guide castings. The grooved guide castings must be inspected closely for wear in the grooves. The cable hold-downs, turnbuckles, cleats, guides, clamps, splay castings, and the travel rollers and their guides must be examined.

Check for damage, including cracking, at drums and sheaves. Note the condition and alignment of span guides.

Check that balance chains hang freely, that span leveling devices are functioning, and that span and counterweight balance closely. Observe if span becomes "out of level" during lifting operation. Inspect spring tension, brackets, braces, and connectors of power cable reels.

12.2.15 Electrical Inspection Considerations

An electrical specialist should be available for the inspection of the electrical equipment. The inspection should be made using FHWA-IP-77-10, *Bridge Inspector's Manual for Movable Bridges* (FHWA-IP-77-10 is currently out of print). AASHTO Movable Bridge Inspection, Evaluation and Maintenance Manual, may aid in the movable bridge inspection. The inspector should observe the functional operation of the bridge and look for abnormal performance of the equipment. Check the operational procedures and safety features provided. Evaluate the maintenance procedures being followed and check the frequency of services performed.

Power Supplies

The normal power supply, standby power supply, and standby generator set (for emergency operation of bridge and service lighting) should be examined and the following noted:

- Take megger readings on the cable insulation values, noting the weather conditions, namely temperature and humidity.
- Make sure all cable connections are properly tightened.
- Measure the voltage and the current to the motors at regular intervals during the operation of the bridge.
- Check the collector rings and windings on the generator set.
- Test starting circuitry for automatic starting and manual starting.
- See if the unit is vibrating while running under load.

If the power cable has been repaired with a splice, note the condition of the splice box seal.

If no standby power supply has been provided, determine whether a portable generator could be used. A manual transfer switch would be a convenient way of connecting it.

Motors

Span drive motors, lock motors, brake thrustor motors, and brake solenoids should be examined for the same items as given for power supplies.

Transformers

Check dry transformer coil housings, terminals, and insulators, including their temperature under load. Observe the frames and supports for rigidity to prevent vibration. The liquid filled transformer should be checked in the same way, and the oil level should be checked while looking for leakage. Examine oil insulation test records.

Circuit Breakers

Check circuit breakers (e.g., air, molded case, and oil) and fuses, including the arc chute, contact surfaces, overload trip settings, insulation, and terminal connections. Examine oil insulation test records, and observe the closing and tripping operation. Record all fuse types and sizes being used.

Wires and Cables

Examine the wiring and cables for both power and control. Note whether the submarine cables are kinked, hooked, or deteriorated, especially at the exposed area above and below the water. In tidal areas, look for marine and plant growth. Note if the ends of the cable have been protected from moisture. Record the insulation value of each wire as measured by megger. Look for cracking, overheating, and deterioration of the insulation. Check for wear against surfaces and especially sharp edges. Check the adequacy of supports and that dirt and debris do not accumulate against the conduit and supports. Terminal connections, clamps, and securing clips should be checked for tightness, corrosion, and that there are wire numbers on the end of each wire. The weight of the wires or cables must be carried by the clamps and not by the wire connections at the terminal strips.

Cabinets

Examine the programmable logic controller (PLC) cabinets, control consoles and stations, switchboards (see Figure 12.2.66), relay cabinets, motor control centers (MCC), and all enclosures for deterioration, debris inside, drainage, operations of heater to prevent condensation, and their ability to protect the equipment inside. Check the operation of all traffic signals, traffic gates, traffic barriers, and navigation lights. Verify that the bridge is open to provide the clearance shown on the permit drawing before the green span light turns on. Check the traffic warning equipment and control circuits, including the advanced warning signals (if used), traffic lights/signals, gates, barriers, and the public address and communication equipment.

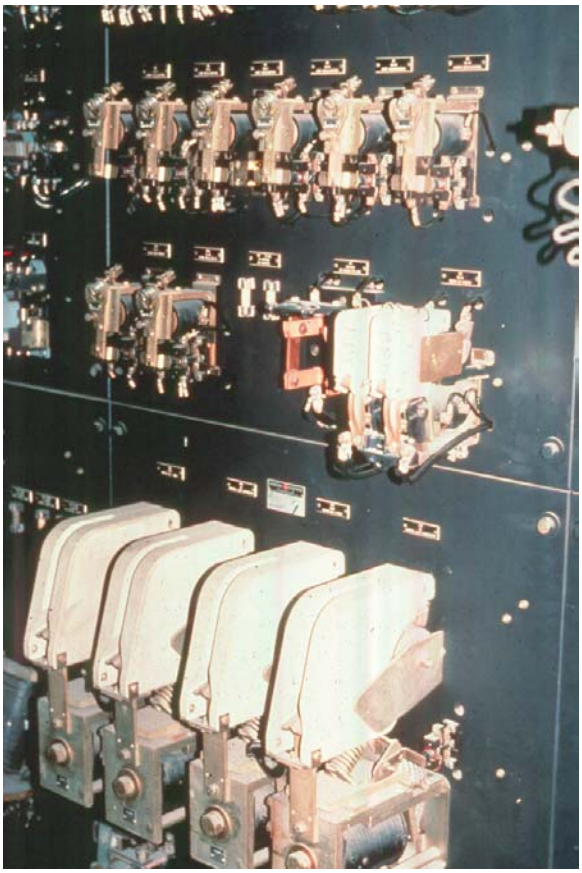


Figure 12.2.66 Open Switchboard

Conduit

See if all conduit is far enough away from all surfaces to avoid debris from collecting against it. Note if it is adequately supported and pitched to drain away from junction boxes and pull boxes, so that water is not trapped within. Also, note if all conduits have covers with seals. Report deteriorated conduit so that it can be replaced with new conduit. The connectors at the ends of all PVC coated conduit must be sealed and re-coated after all fittings are installed.

Junction Boxes

The covers on all junction boxes (JB's) should be examined for an effective seal, dry interior, functioning breather-drains, heaters having enough power to prevent condensation inside, and terminal strips all secured to the bottom of horizontal JB's or to the back of vertical JB's.

Meters

Observe if all voltmeters, ammeters, and watt meters are freely fluctuating with a change in load. All switches for meters should be operable.

Control Starters and Contactors/Relays

Check the operation of this equipment under load, and watch for arcing between contacts, snap action of contacts, deterioration of any surfaces, and drainage of any moisture. Look for signs of corrosion and overheating.

Limit Switches

All limit switches should be set so they do not operate until they are intended to stop the equipment or complete an interlock. The interior should be clean and dry, with all springs active.

Selsyn Transmitters and Receivers

Check for power to the field and signal being sent from the transmitter to the receiver. Observe the receiver tracking the rotation of the bridge as it operates. Observe the mechanical coupling between the driving shaft and the transmitter, checking for damage and misalignment.

Service Light and Outlet

Power should be going to each light and outlet. Note if there is a shield or bar for protecting each bulb and socket. It is desirable to have service lights available when power is removed from all movable bridge controls and equipment.

12.2.16 Hydraulic Inspection Considerations

A hydraulic power specialist should be available for the inspection of the hydraulic equipment (see Figure 12.2.67). The inspector should observe the functional operation of the bridge and look for abnormal performance of the equipment. Check the safety features provided and evaluate the maintenance procedures being followed, checking the frequency of services performed. Due to the inter-related function of components, the requirements for fluid cleanliness, and the need for personnel safety, the reservoir and hydraulic lines should not be opened. In addition, no components or parts of the power circuit should be shut off or adjusted without complete understanding of their function and knowledge of the effect such action will have upon the system. Items which should be checked during a hydraulic inspection include the following:

- Leakage anywhere in the system should be noted. Significant leakage should immediately be brought to the attention of the bridge authority.
- Check for corrosion of reservoir, piping, and connections.
- Sight gauges should be inspected for proper fluid level in reservoir. Note gauges with low fluid levels or gauges which cannot be read.
- Unusual noises from any part of the system should be noted.
- Check filter indicators to make sure filters are clean.
- A sample of the hydraulic fluid should be taken for analysis by a testing laboratory during periodic inspections.



Figure 12.2.67 Hydraulic Power Specialists

12.2.17 Recordkeeping and Documentation

General

The owner of a movable bridge must keep a complete file available for the engineer who is responsible for the operation and maintenance of the bridge. The file should include (if applicable), but not be limited to, the following:

- Copy of the latest approved permit drawing
- Complete set of design plans and special provisions
- "As-built" shop plans for the structural steel, architectural, mechanical, electrical, and hydraulic
- Machinery Maintenance Manual
- Electrical Maintenance Manual
- Hydraulic Maintenance Manual
- Copy of maintenance procedures being followed
- Copy of the latest Operator's Instruction being followed
- Copies of all inspection reports
- Copy of all maintenance reports
- Copy of all repair plans
- Up-to-date running log on all spare parts that are available, on order, or out of stock

Inspection and maintenance reports should be reviewed with preventative maintenance measures in mind. An example would be the "megger" readings on wiring insulation, especially those taken on damp rainy days when moisture could influence (reduce) the values. An acceptable minimum reading is usually 1 megohm. If the value on a wire is decreasing on progressive reports, preventative maintenance may save a "short" that could burn out equipment and put the bridge out of operation.

Inspection and Maintenance Data

Examples of inspection and maintenance records that should be kept are shown in Figures 12.2.68 through 12.2.74.

South Tower — Differential Assembly GEARS - General — 1.

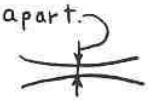
Gear	General Condition	Lubrication	Keys	Alignment		
				Center Distance	Axial	Parallel
Pinion P5	Very Good. Tooth profiles show normal wear	Very Good	Good	Good. Pitch Lines Tangent	Good	Good
Gear I5	Very Good. Tooth profiles normal.	Very Good	Good	No Pitch Line on G5. Looks good. Measured backlash.		
Gear G5	Very Good. Tooth profiles normal	Very Good	Good			
Pinion P4	Very Good. Tooth profiles normal.	Very Good	Integral with shaft	No pitch line on P4. Center distance looks. Looks good. Measured backlash.	Good	Good
Gear G4	Very Good. Tooth profiles normal.	Very Good	Not keyed to shaft. Clutch locks G4 to shaft.			
Bevel Gears BG3 (2)	Very Good. Tooth profiles normal.	Very Good	Integral with sleeves.	Good. Pitch Lines 1/16" to 1/8" apart. 	Good	Good
Bevel Pinions BP3 (2)	Very Good. Tooth profiles normal.	Very Good	Integral with shafts.			

Figure 12.2.68 Example of Notes on Operating Machinery (Gears-General)

South Tower Differential Assembly GEARS - Teeth 2.

Gear	Chordal Thickness		Backlash		Condition of Teeth					
					Normal	Pitting	Rolling-Peening	Abnormal		
	Original	Measured	Original	Measured				Scoring	Interference	Rust & Corr.
Pinion P5	.625"	Did not measure	.011" min to .020" max.	Did not measure. Pitch lines indicate good backlash.	✓					
Gear I5	.625"		.011" min		✓					
Gear G5	.625"		to .020" max.	.0135" Good.	✓					
Pinion P4	.625"		.011" min to	.020" Good	✓					
Gear G4	.625"		.020" max		✓					
Bevel Gears BG3 (2)	.875" at large end of teeth		.015" min to	Did not	✓					
Bevel Pinions BP3 (2)	.875" at large end of teeth		.029" max	measure. Pitch lines indicate good backlash.	✓					

Figure 12.2.69 Example of Notes on Operating Machinery (Gears-Teeth)

South Tower Differential Assembly BEARINGS 3

Bearing	General Condition	Clearance		Bolts	Lubrication
		Original	Measured		
West end Emer. Motor Shaft	Good. Fairly clean, paint good. Bearing has 45° angle lube fitting w/dust cap.	.0025" min. to .0073" max.	.006" Good	Good. Nuts tight. Clean, paint good.	Good.
East end Emer. Motor Shaft		.0025" min, to .0073" max.	.006" Good		
West end Intermediate Shaft		.0025" min. to .0073" max.	.007" Good		
East end Intermediate Shaft		.0025" min. to .0073" max.	.005" Good		
West end Normal Motor Shaft		.0025" min. to .0073" max.	.007" Good		
East end Normal Motor Shaft		.0025" min to .0073" max	.009" Fair		

Figure 12.2.70 Example of Notes on Operating Machinery (Bearings)

South Tower Differential Assembly MECHANICAL COMPONENTS 4.

Item	General Condition
Housing Cover	Very good condition. Cover has four hinged maintenance panels, secured with studs and wingnuts. Cover bolted to lower supports with 20 bolts.
Normal (Main) Drive Clutch Cone	Very good condition. No slippage during span operation, starting or stopping. Clutch cone is inside differential assembly and impossible to inspect without disassembly of differential.
Emergency Drive Clutch Cone Assembly	Very good condition. Design plans show cone type clutch. Actually have jaw type clutch.
Differential Clutch Operating Linkage	Very good condition. Well lubricated. Linkage operates smooth and quiet.
Emergency Drive Clutch Operating Linkage	Very good condition. Well lubricated. Linkage operates smooth and quiet.
Gear Motor for operation of Differential Clutch	Good condition. Operates smoothly. Operated with hand crank, turned fairly easy. GE AC Gearmotor, Model KY3AC2345, Motor 1800 rpm, 1/6 HP, 250:1 ratio
Support for above Gear Motor	Good. Some debris and oil on support.
Gear Motor for operation of Emer. Drive Clutch	Good. Operates smoothly. Turned easily with hand crank. Same gearmotor as at differential clutch
Support for above Gear Motor	Good. Some debris and oil on support.
Housing Support	Good condition. Some debris and oil on support and floor. Paint good. 2 lights attached to supports inside

Figure 12.2.71 Example of Notes on Operating Machinery (Mechanical Elements)

Electrical Equipment 125HP, 600RPM, 3φ, 60H

Motor A (Normal - Traction) Tower South - Side W

General Items	General Condition
Stiffness of Supports	Good
Connection to "	Bolts tight
Condition of Frame	Dirty & Dusty Inside & Out
Inspection Covers	Wire Mesh, 2 on Top (2 on Bottom missing)
Gaskets on "	None
Bolts on "	Tight
Ventilation	Open Ends
Operation-Noise	Normal
" -Vibration	Minimal
" --Bearings	Normal wear
Lubrication	Needs normal application
Oil-Dirt Build-Up	None (Except at couplings)
Insulation	See Megger test
Cable Connections	Good

Wound Rotor Motors	Wire No.	Raising Span Amps.	Lowering Span Amps.	
Motor Current - φ A	T1A	122	91	
B	T3A	124	93	
C	T2A	124	92	
Motor Voltage - A-B				460V
A-C				
B-C				
Rings - Surface	Normal wear			
" - Arcing	None Visible			
Brushes - Contact	Good			
" - Spring Pressure	Good, Springs Rusty			
" - Condition	Good, 2¼" length			
Wiring - Connection	Tight, Bolts Rusty			
" - Insulation	Good			
Rotor Current 3 φ A	M1A	50	31	
B	M3A	48	32	
C	M2A	50	32	

Figure 12.2.72 Example of Notes on Electrical Equipment (Motors)

Megger Insulation Test Temp 60's Weather Dry.

Rotating Cam - Normal Height Limit Switch.
Contacts shown for Bridge Closed. Tower South Side W

Bottom Connection			Gear Drive End North	Top Connection ..		
Remarks	500V MΩ to Ground	Wire No. Taged U.N.		Wire No.	500V MΩ to Ground	Remarks
	0.2	1084	1 Contacts	1081	10.	
	0.2	1085	2			
	16.	No Tag 1083	3	1003	8.	
	18.	1105	4	1010.	0.2	
	20.	No Tag 1110	5			
	18.	1117	6			
	18.	1125	7			
	0.2	2051	8	2022	0.2	
	0.2	2052	9			
Spare		No Wires	10			

Remarks: Cover has probably been left OFF for a period of time. No gaskets, clips on some switches not hooked. Connection screws inside all rusty on the bottom. Springs rusty but still springy. Contacts are clean with fair contact alignment.

Figure 12.2.73 Example of Notes on Electrical Equipment (Limit Switch)

Megger Insulation Test of the Submarine Cables

Temp 50°s Weather Dry

Equipment Being Controlled	Wire No. on Plans	Emergency Cables			Normal Cables		
		No. in Cable	500V M-Ω	Remarks	No. in Cable	500V M-Ω	Remarks
North Tower Elev.	261	1	6		2	500	
	261	3	6		4	500	
	263	5	1.5		6	<.2	>20k-Ω
	263	7	1.5		8	.1	
	262	9	.9		10	.1	
	262	11	.9		12	.1	
Service Brake C	447	13	2.0		14	1000	
	446	15	40.		16	1000	
	448	17	15.		18	1000	
Service Brake D	467	19	2.		20	1000	
	466	21	25.		22	1000	
	468	23	5.		24	1000	
Drag Brake L	519	25	20.		26	1000	
	516	27	35.		28	1000	
	520	29	5.		30	1000	
Drag Brake M	529	31	4.		32	1000	
516	526	33	5.		34	1000	
	535	35	1.		36	1000	
North Locks Motor	617	37	0.8		38	1000	
	616	39	10.		40	1000	
	618	41	0.2		42	1000	
North Barrier Gate Motor	647	43	12.		44	1000	
	646	45	.7		46	1000	
	648	47	90		48	∞	
N.W. Traffic Gate Motor	687	49	.2		50	1000	
	686	51	35.		52	∞	
	688	53	100.		54	∞	
N.E. Traffic Gate Motor	697	55	9.		56	1000	
	696	57	6.		58	1000	
	698	59	3.		60	1000	

Figure 12.2.74 Example of Notes on Electrical Equipment (Megger Insulation Test of the Submarine Cables)

12.2.18 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of movable bridges. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the PONTIS Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

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

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for appropriate movable bridge member. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. Element Level Smart Flags are also used to describe the condition of the steel superstructure both unpainted and painted.

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

Element No.	Description
101	Unpainted Steel Closed Web/Box Girder
102	Painted Steel Closed Web/Box Girder
106	Unpainted Steel Open Girder/Beam
107	Painted Steel Open Girder/Beam
112	Unpainted Steel Stringer(Stringer-Floorbeam System)
113	Painted Steel Stringer(Stringer-Floorbeam System)
120	Unpainted Steel Thru truss (Bottom Chord)
121	Painted Steel Thru truss (Bottom Chord)
125	Unpainted Steel Thru Truss (Excluding Bottom Chord)
126	Painted Steel Thru Truss (Excluding Bottom Chord)
130	Unpainted Steel Deck Truss
131	Painted Steel Deck Truss
140	Unpainted Steel Arch
141	Painted Steel Arch
151	Unpainted Steel Floorbeam
152	Painted Steel Floorbeam

The unit quantity for the superstructure is meters or feet and the total quantity must be placed in one of the available condition states. In both cases, Condition State 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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

Element No. 362, can be used and one of the three condition states assigned. For movable bridges with section loss, the "Section Loss" Smart Flag, Element No. 363, can be used and one of the four condition states assigned.

For mechanical, electrical, and hydraulic movable bridge members, individual states may choose to create their own non-CoRe elements.

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Topic 12.3 Concrete Pipe Culverts

12.3.1 Introduction

A common type of culvert used today is the concrete pipe culvert (see Figure 12.3.1). The concrete pipe culvert is typically circular or elliptical in shape. The size of the opening of the pipe is determined by the peak flow of the channel. The circular shape is the most common shape manufactured for pipe culverts. It is hydraulically and structurally efficient under most conditions. Possible hydraulic drawbacks are that circular pipe generally causes some reduction in stream width during low flows. It may also be more prone to clogging than some other shapes. Elliptical shapes are used in situations where horizontal or vertical clearance is limited. The oblong shape allows the pipe to fit where a circular pipe may not, but still allows for the necessary size opening. Elliptical shaped pipe culverts may also be used when a wider section is desirable for low flow levels. As with circular shaped pipe culverts, these shapes also are prone to clogging as the depth of flow increases. In situations where the required size of the opening is very large, two or more concrete pipe culverts may be used (see Figure 12.3.2).



Figure 12.3.1 Concrete Pipe Culvert

Culverts are somewhat protected by the soil backfill from rapid fluctuations in surface temperature and direct application chloride (salts) used for deicing. As a result they are generally more resistant to surface deterioration than concrete bridge elements. Concrete culverts are classified as rigid structures because they do not bend or deflect appreciably.



Figure 12.3.2 Twin Concrete Pipe Culvert

12.3.2 Design Characteristics

Structural Behavior

The load carrying capability of rigid culverts is essentially provided by the structural strength of the pipe itself and little benefit from the surrounding soil is required. When vertical loads are applied to rigid culverts, tension and compression zones are created (see Figure 12.3.3). Reinforcing steel is added to the tension zones to increase the tensile strength of the pipe. Shear stress in the haunch or "bell" area where the pipe sections are joined, can be critical for heavily loaded rigid pipe on hard foundations, especially if the pipe bed preparation is inadequate. Because rigid pipe is stiffer than the surrounding soil, it carries a substantial portion of the load.

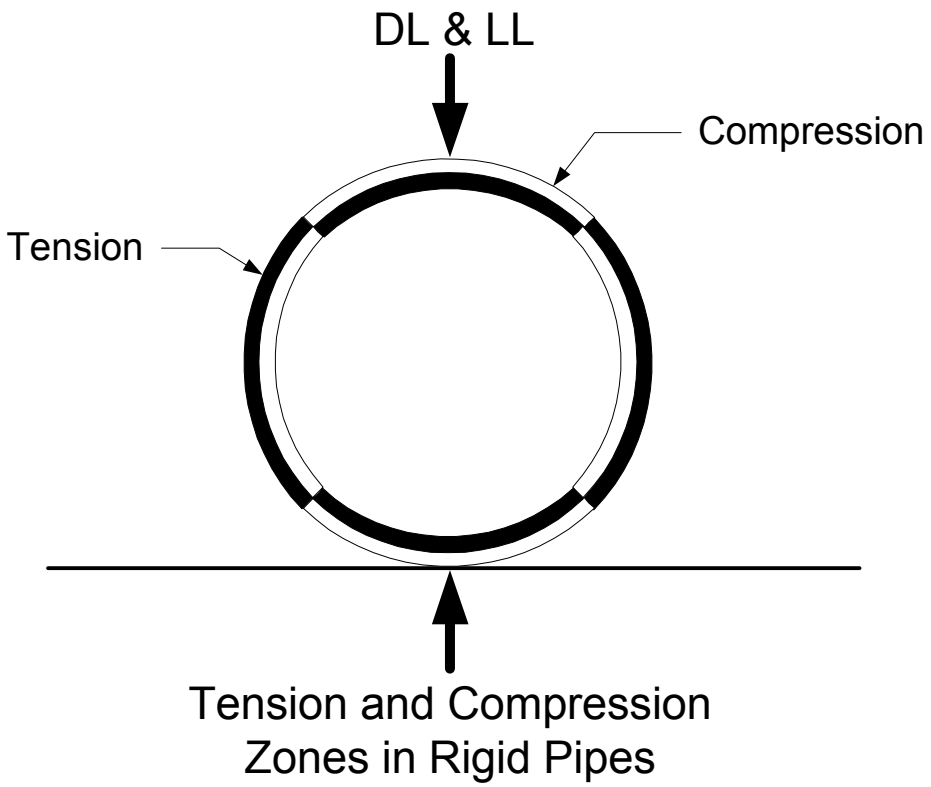


Figure 12.3.3 Rigid Culvert Stresses

The weight of earth that must be carried varies with soil characteristics and installation conditions. The installation conditions can have a significant influence on the loads that must be carried by a rigid culvert. There are two major classes of installation conditions: 1) trench, where culverts are placed in natural ground or compacted fill with a controlled trench width and 2) embankment, where culverts are placed in or covered by an embankment.

In narrow trench installations, the pipe is placed in a relatively narrow trench and covered with backfill material. The backfill tends to settle more than the undisturbed soil beside the trench. Friction between the backfill material and the sides of the trench tends to help support the backfill material reducing the load on the pipe. In effect the width of the soil column over the pipe is decreased.

As the trench width increases, the effect of the friction at the sides of the trench is reduced and dead load on the pipe is increased. The amount that the loading is increased depends on trench width and the amount of backfill settlement, which is related to compaction. Poorly compacted soil will settle more than well compacted soil. In a trench that is too wide, poor compaction can result in an increase in the dead load on the pipe. Pipes placed in a shallow bedding on top of the original ground surface and then covered by the embankment material will have loads similar to the very wide trench. Pipes placed in trenches in the original ground prior to being covered by embankment have reduced earth loads similar to those described for the narrow trench installations.

12.3.3 Types and Shapes of Concrete Pipe Culverts

Concrete culvert pipe is manufactured in up to five standard strength classifications. Higher classification numbers indicate higher strength. All of these standard shapes are manufactured in a wide range of sizes. Circular and elliptical pipes are available with standard sizes as large as 3.7 meters (12 feet) in diameter, with larger sizes available for special designs. Several factors such as span length, vertical and horizontal clearance, peak stream flow and terrain determine which shape of pipe culvert is used.

Precast concrete pipe culverts are manufactured in three standard shapes:

- Circular
- Horizontal elliptical
- Vertical elliptical

See pages 12.3.18 – 12.3.20 for the different standard sizes for concrete pipe culvert shapes.

12.3.4 Hazards of Culvert Inspection

The bridge inspector should be alerted to the following hazards when inspecting a culvert.

- Inadequate ventilation
- Drowning
- Toxic chemicals
- Animals
- Quick conditions at the outlet
- Insufficient number of inspectors

Refer to Topic 3.2.5 for a detailed discussion of each hazard.

12.3.5 Overview of Common Defects

Some of the more common defects that are found in concrete pipe culverts are:

- Cracking
- Spalling
- Delaminations
- Efflorescence
- Section loss of exposed reinforcing bars
- Embankment erosion at culvert entrance and exit
- Roadway settlement
- Foundation Failure
- Scour / Undermining
- Misalignment
- Settlement of pipe sections

For a detailed discussion of concrete defects, see Topic 2.2.

12.3.6 Inspection Procedures and Locations

Safety is the most important reason that culverts should be inspected. For a more detailed discussion on reasons for inspecting culverts, see Topic P.3.1.

Previous inspection reports and as-built plans, when available, should be reviewed prior to, and possibly during, the field inspection. A review of previous reports will familiarize the inspector with the structure and make detection of changed conditions easier. A review will also indicate critical areas that need special attention and the possible need for special equipment.

A logical sequence for inspecting culverts helps ensure that a thorough and complete inspection will be conducted. In addition to the culvert components, the inspector should also look for highwater marks, changes in the drainage area, settlement of the roadway, and other indications of potential problems. In this regard, the inspection of culverts is similar to the inspection of bridges.

For typical installations, it is usually convenient to begin the field inspection with general observations of the overall condition of the structure and inspection of the approach roadway. The inspector should select one end of the culvert and inspect the embankment, waterway, headwalls, wingwalls, and culvert barrel. The inspector should then move to the other end of the culvert. The following sequence is applicable to all culvert inspections:

- Review available information
- Observe overall condition
- Inspect approach roadway and embankment
- Inspect waterway (see Topic 11.2)
- Inspect end treatments
- Inspect culvert barrel

Procedures

Visual

The inspection of concrete for cracks, spalls, and other defects is primarily a visual activity. However, hammers can be used to detect areas of delamination. A delaminated area will have a distinctive hollow “clacking” sound when tapped with a hammer or revealed with a chain drag. A hammer hitting sound concrete will result in a solid "pinging" type sound.

Physical

If the inspector deems it necessary, core samples can be taken from the culvert and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete culverts are caused by corrosion of steel reinforcement. When the deterioration of a concrete member progresses to the point of needing rehabilitation, an in-depth inspection of the member is required to determine the extent, cause, and possible solution to the problem. Several techniques and methods are available, as described in Topic 2.2

Advanced Inspection Techniques

In addition, several advanced techniques are available for concrete inspection.

Nondestructive methods, described in Topic 13.2.1, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Delamination detection machinery
- Electrical methods
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
- Ground-penetrating radar
- 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

Other methods, described in Topic 13.2.7, include:

- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Rigid culverts such as precast concrete pipe do not deflect appreciably before cracking or fracturing. As a result, shape inspections, while very important in flexible structures, are of little value in inspecting precast concrete culverts.

Although the need for soil stability and side support is obviously important with flexible pipe, it is less important with rigid pipe. However, adequate stability of the surrounding soil is necessary to prevent settlement around the culvert and to achieve load carrying capability. The inspector should therefore look for any indications of a lack of soil stability such as settlement or misalignment as well as signs of structural distress such as cracking. Descriptions of the types of distress to look for during inspection are provided in the following paragraphs. Guidelines for condition ratings of concrete pipe are included at the end of this Topic.

The following is a list of areas that should be inspected in concrete pipe culverts.

- Misalignment
- Joint Defects
- Cracks
- Spalls
- Slabbing
- Durability
- End Section Drop-off

Misalignment

Misalignment may indicate the presence of serious problems in the supporting soil. The vertical and horizontal alignment of the culvert barrel should be checked by sighting along the crown and sides of the culvert and by checking for differential movement or settlement at joints between pipe sections. Vertical alignment should be checked for sags, faulting, and heaving. The inspector should be aware that pipes are occasionally laid with a camber or a grade change (broken back grade) to allow for fill settlement.

Sags which trap water may aggravate settlement problems by saturating the supporting soil. Horizontal alignment should be checked for straightness or smooth curvature for those culverts constructed with a curved alignment. Alignment problems may be caused by improper installation, undermining, or uneven settlement of fill. The inspector should attempt to determine which of those problems is causing the misalignment. If undermining is determined to be the probable cause, maintenance forces should be notified since damage will continue until the problem is corrected. The inspector should also try to determine whether the undermining is due to piping, water exfiltration, or infiltration of backfill material. When the misalignment is due to improper installation or uneven settlement, repeat inspections may be needed to determine if the settlement is still progressing or has stabilized.

Joint defects

Joint defects are fairly common and can range from minor problems to problems that are serious in nature. Typical joint defects include leakage (exfiltration and infiltration), cracks, and joint separation. Past and current criteria should be reviewed as some agencies design culverts with open joints to perform as subdrains.

(1) Exfiltration - Exfiltration occurs when leaking joints allow water flowing through the pipe to leak into the supporting material. Many culverts are built with joints that are not watertight or with mortar joints that crack with minor deflection, movement, or settlement of the pipe sections. Minor leakage may not always be a significant problem unless soils are quite erosive. However, if leaking joints contribute to or cause piping, then serious misalignment of the culvert or even failure may result. Leaking joints may be detected during low flows by visual observation of the joints and by checking around the ends of the culvert for evidence of piping.

(2) Infiltration – Infiltration is the opposite of exfiltration. Many culverts are essentially empty except during peak flows. When the water table is higher than the culvert invert, water may seep into the culvert between storms. This infiltration of water can cause settlement and misalignment problems if it carries fine grained soil particles from the surrounding backfill. Infiltration may be difficult to detect visually in its early stages although it may be indicated by open joints, staining at the joints on the sides and top of the culvert, deposits of soil in the invert, or by

depressions over the culvert, as shown in exhibit 105.

(3) Cracks – Cracks in the joint area may be caused by improper handling during installation, improper gasket placement, and movement or settlement of the pipe sections. Cracked joints are more than likely not watertight even if gaskets were used. However, if no other problems are evident, such as differential movement between pipe sections, and the cracks are not open or spalling, they may be considered a minor problem to only be noted in the inspection report. Severe joint cracks are similar in significance to separated joints.

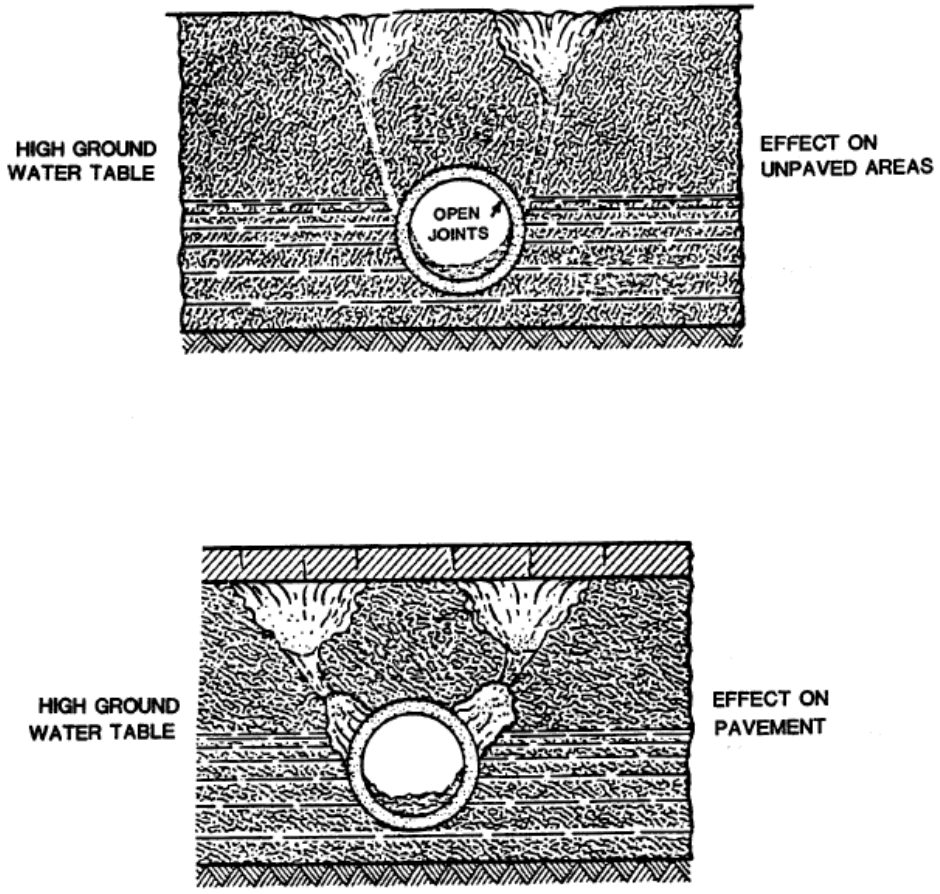


Figure 12.3.4 (Exhibit 105 – Culvert Inspection Manual, Report No. FHWA-IP-86-2) Surface Indications of Infiltration

(4) Separated Joints - Joint separations may be caused by the same forces described under misalignment (settlement, undermining, or improper installation). Joint separations are significant because they accelerate damage caused by exfiltration and infiltration resulting in the erosion of the backfill material. Examples of severe infiltration through separate joints are shown in exhibits 106 and 107. Separated joints are often found when severe misalignment is found. In fact either problem may cause or aggravate the other. Movement of the soil in the general direction of the culvert's centerline may cause sections

to gradually pull apart. Embankment slippage may also cause separations to occur.

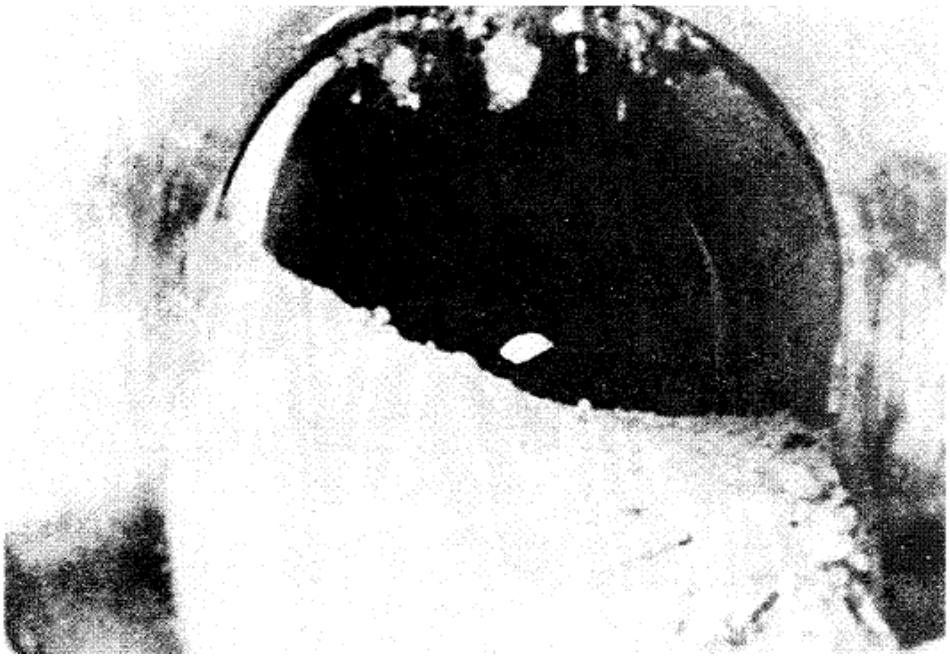


Figure 12.3.5 (Exhibit 106) Example of Severe Infiltration of Backfill Material through Separated Joints

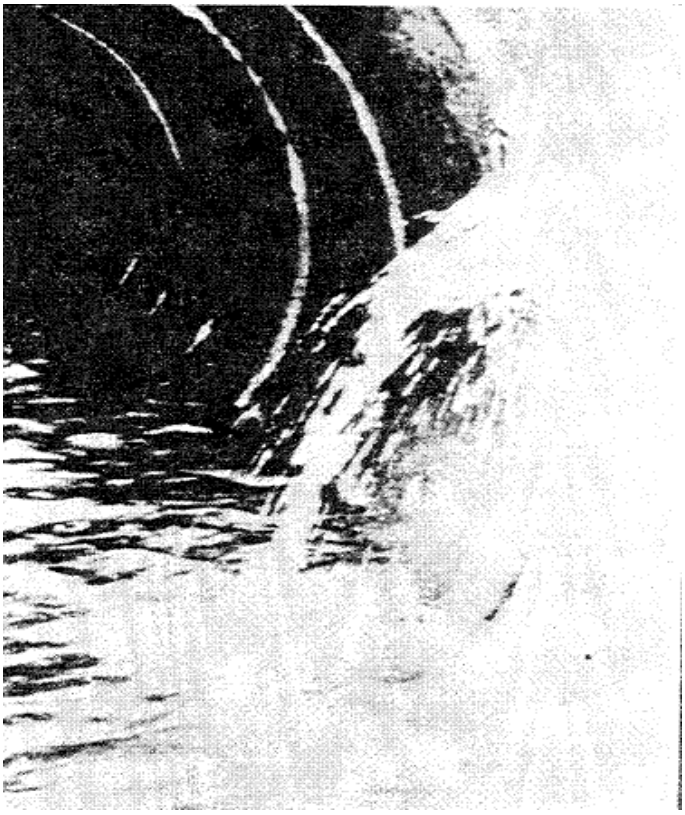


Figure12.3.6 (Exhibit 107) Severe Infiltration of Ground Water Through Separated Joint

Cracks

Longitudinal Cracks – Concrete is strong in compression but weak in tension. Reinforcing steel is provided to handle the tensile stresses. Hairline longitudinal cracks in the crown or invert indicate that the steel has accepted part of the load. Cracks less than 0.01 inches in width are minor and only need to be noted in the inspection report. Cracks greater than hairline cracks, or those more than 0.01 inch in width but less than 0.1 inches, should be described in the inspection report and noted as possible candidates for maintenance. Longitudinal cracking in excess of 0.1 inch in width may indicate overloading or poor bedding. If the pipe is placed on hard material and backfill is not adequately compacted around the pipe or under the haunches of the pipe, loads will be concentrated along the bottom of the pipe and may result in flexure or shear cracking, as illustrated in Figure 12.3.7.

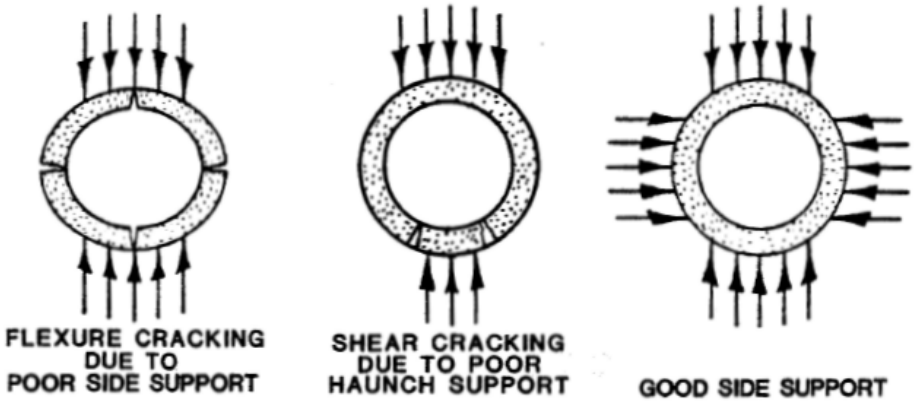


Figure 12.3.7 (Exhibit 108) Results of Poor and Good Side Support, Rigid Pipe

Other signs of distress such as differential movement, efflorescence, spalling, or rust stains should also be noted. Examples of longitudinal cracking are shown in Figures 12.3.8 and 12.3.9. When cracks are wider than 0.1 inch measurements should be taken of fill height and the diameter of the pipe both horizontally and vertically to permit analysis of the original design. Crack measurements and photographs may be useful for monitoring conditions during subsequent inspections.

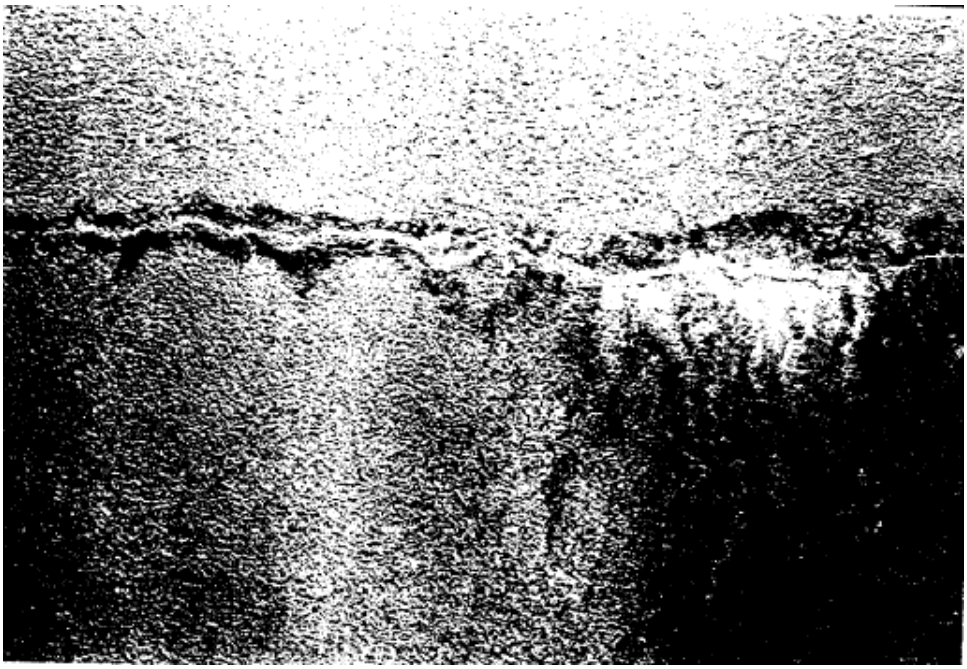


Figure 12.3.8 (Exhibit 108) Minor Longitudinal Crack with Efflorescence

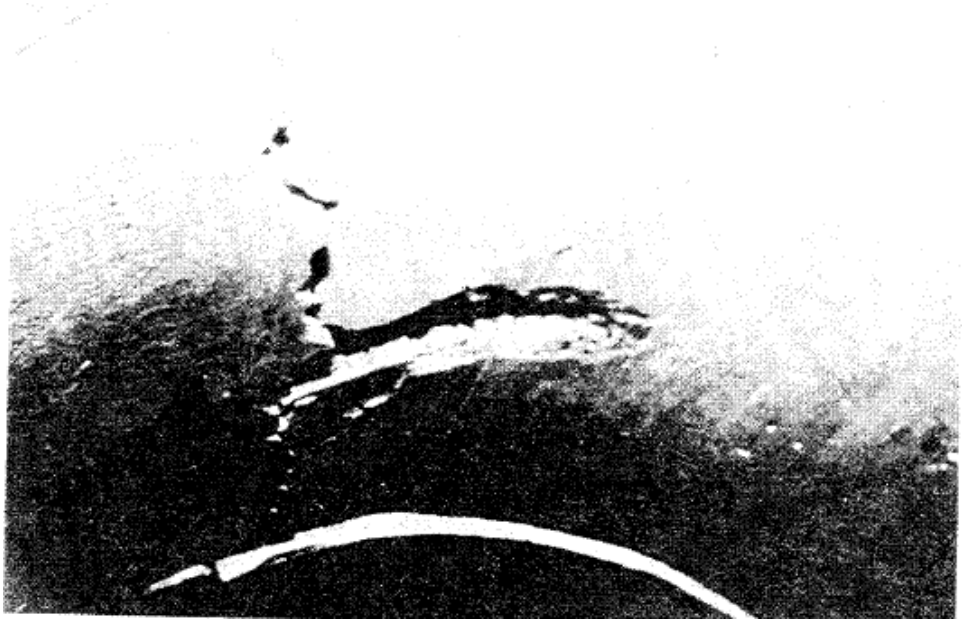


Figure 12.3.9 (Exhibit 110) Severe Longitudinal Cracks with Differential Movement and Spalling

Transverse Cracks – Transverse or circumferential cracks may also be caused by poor bedding. Cracks can occur across the bottom of the pipe (broken bell) when the pipe is only supported at the ends of each section. This is generally the result of poor installation practices such as not providing indentions (bell holes) in hard foundation material for the ends of bell and spigot-type pipe or not providing a sufficient depth of suitable bedding material. Cracks may occur across the top of pipe (broken back) when settlement occurs and rocks or other areas of hard

foundation material near the midpoint of a pipe section are not adequately covered with suitable bedding material. Transverse cracking is illustrated in Figure 12.3.10.

Spalls

Spalling is a fracture of the concrete parallel or inclined to the surface of the concrete. In precast concrete pipe, spalls often occur along the edges of either longitudinal or transverse cracks when the crack is due to overloading or poor support rather than simple tension cracking. Spalling may also be caused by the corrosion of the steel reinforcing when water is able to reach the steel through cracks or shallow cover. As the steel corrodes, the oxidized steel expands, causing the concrete covering the steel to spall. Spalling may be detected by visual examination of the concrete along the edges of cracks. Tapping with a hammer should be performed along cracks to check for areas that have fractured but are not visibly separated. Such areas will produce a hollow sound when tapped. These areas may be referred to as delaminations or incipient spalls. Figure 12.3.11 shows spalling with reinforcing steel exposed.

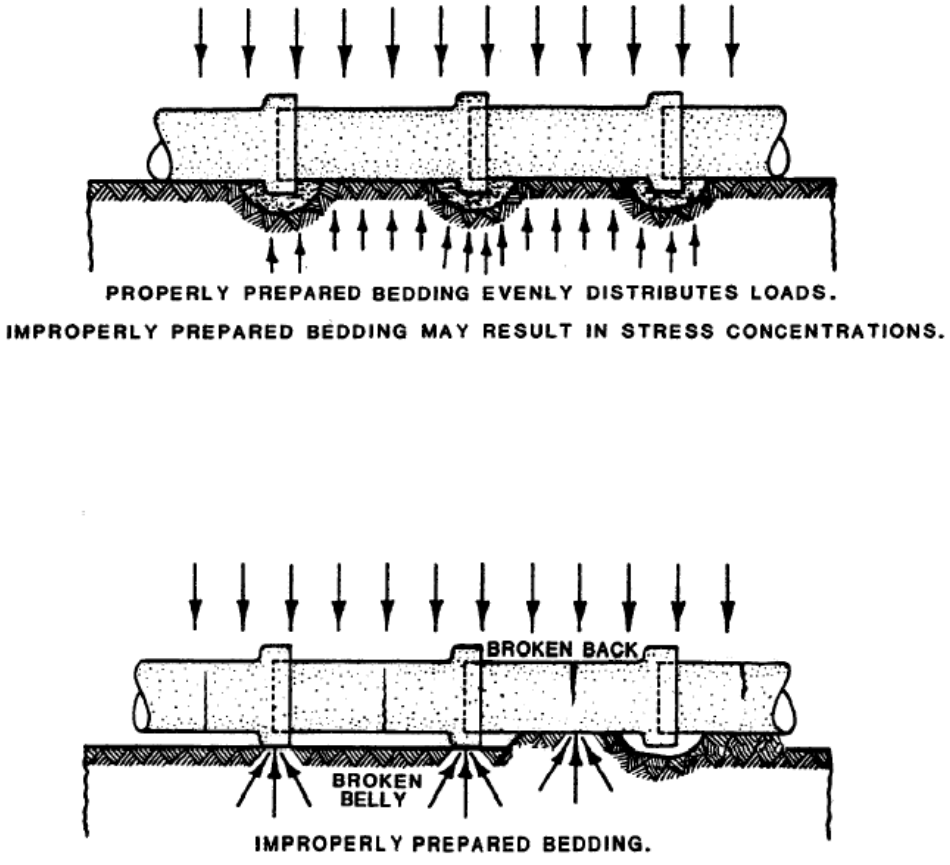


Figure 12.3.10 (Exhibit 111) Transverse or Circumferential Cracks

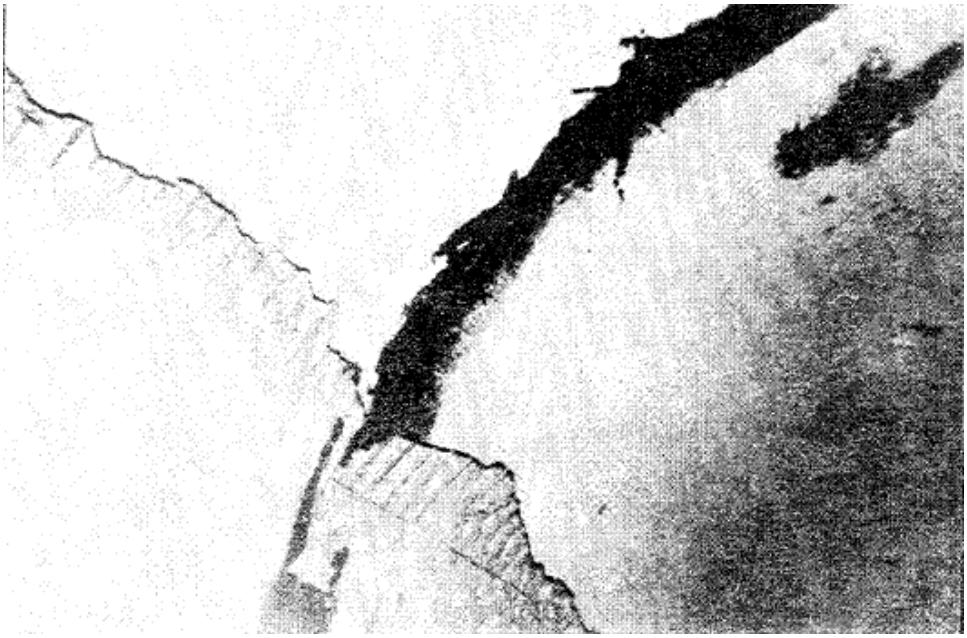


Figure 12.3.11 (Exhibit 112) Spalling Exposing Reinforcing Steel

Slabbing

The terms slabbing, shear slabbing, or slab shear refer to a radial failure of the concrete which occurs from straightening of the reinforcement cage due to excessive deflection. It is characterized by large slabs of concrete "peeling" away from the sides of the pipe and a straightening of the reinforcing steel as shown in Figure 12.3.12. Slabbing is a serious problem that may occur under high fills.

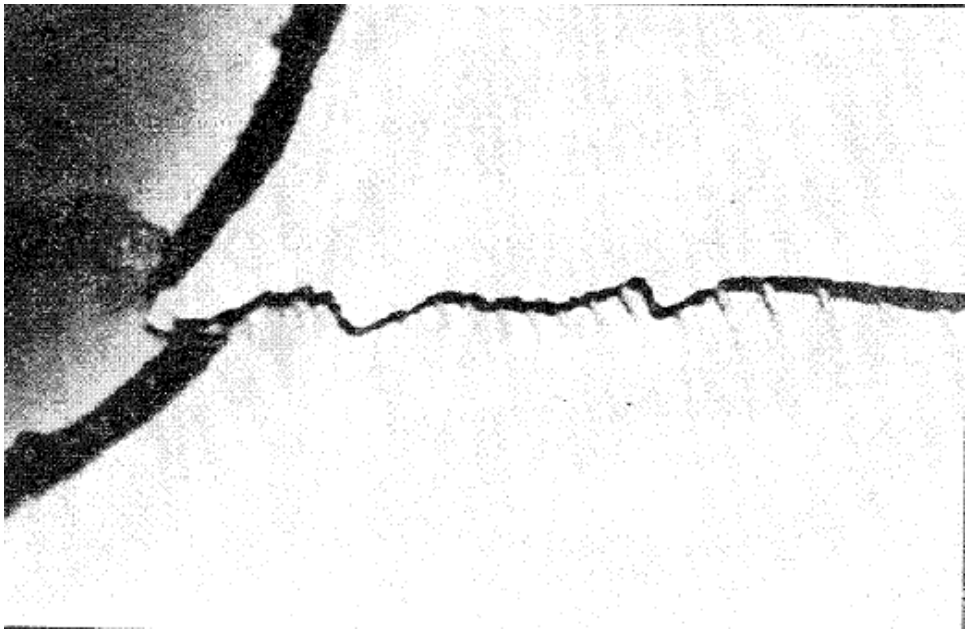


Figure 12.3.12 (Exhibit 113) Shear Slabbing

Durability

Durability is a measure of a culvert's ability to withstand chemical attack and abrasion. Concrete pipes are subject to chemical attack in strongly acidic environments such as drainage from mines and may also be damaged by abrasion. Abrasion damage is a wearing away of the concrete surface by sediment and debris being transported by the stream. Mild deterioration or abrasion less than 1/4 inch deep should be noted in the report. More severe surface deterioration should be reported as a potential candidate for maintenance. In severe cases where the invert is completely deteriorated, maintenance forces should be given immediate notification. When linings are used to protect against chemical attack or abrasion the condition of the lining should be noted in the report.

End Section Drop-off

This type of distress is usually due to outlet erosion as discussed earlier in the sections on end treatments and waterways. It is caused by the erosion of the material supporting the pipe sections on the outlet end of the culvert barrel.

RATING GUIDELINES FOR PRECAST CONCRETE PIPE CULVERT BARRELS

RATING	CONDITION
9	• New condition
8	• Alignment: good, no settlement or misalignment • Joints: tight with no defects apparent • Concrete: no cracking, spalling, or scaling present; surface in good condition
7	• Alignment: generally good; minor misalignment at joints; no settlement • Joints: minor openings, possible infiltration/exfiltration • Concrete: minor hairline cracking at isolated locations; slight spalling or scaling present on invert
6	• Alignment: fair, minor misalignment and settlement at isolated locations • Joints: minor backfill infiltration due to slight opening at joints; minor cracking or spalling at joints allowing exfiltration • Concrete: extensive hairline cracks, some with minor delaminations or spalling; invert scaling less than 0.25 in. deep or small spalls present
5	• Alignment: generally fair; minor misalignment or settlement throughout pipe; possible piping • Joints: open and allowing backfill to infiltrate; significant cracking or joint spalling • Concrete: cracking open greater than 0.12 in. with moderate delamination and moderate spalling exposing reinforcing steel at isolated locations; large areas of invert with surface scaling or spalls greater than 0.25 in. deep

RATING	CONDITION
4	• Alignment: marginal; significant settlement and misalignment of pipe; evidence of piping; end sections dislocated about to drop off • Joints: differential movement and separation of joints, significant infiltration or exfiltration at joints • Concrete: cracks open more than 0.12 in. with efflorescence and spalling at numerous locations; spalls have exposed rebars which are heavily corroded; extensive surface scaling on invert greater than 0.5 in.
3	• Alignment: poor with significant ponding of water due to sagging or misalignment pipes; end section drop off has occurred • Joints: significant openings, dislocated joints in several locations exposing fill material; infiltration or exfiltration causing misalignment of pipe and settlement or depressions in roadway • Concrete: extensive cracking, spalling, and minor slabbing; invert scaling has exposed reinforcing steel
2	• Alignment: critical; culvert not functioning due to alignment problems throughout • Concrete: severe slabbing has occurred in culvert wall, invert concrete completely deteriorated in isolated locations
1	• Culvert: partially collapsed • Road: closed to traffic
0	• Culvert: total failure of culvert and fill • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.3.13 (Exhibit 114) Condition Rating Guidelines

Wingwalls

Wingwalls are provided to support the embankment around the openings of the culvert. Wingwalls should be inspected to ensure they are in proper vertical alignment. Wingwalls may be tilted due to settlement, slides or scour. See Topic 10.1 for a detailed description of defects and inspection procedures of wingwalls.

12.3.7 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of concrete pipe culverts. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element Level Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

Using NBIS rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the culvert (Item 62). This item evaluates the alignment, settlement, joints, structural condition, scour, and other items associated with culverts. Rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2. The rating code is intended to be an overall evaluation of the culvert. Integral wingwalls to the first construction or expansion joint shall be included in the evaluation. It is also important to note that Items 58-Deck, 59-Superstructure, and 60-Substructure shall be coded "N" for all culverts.

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the culvert. The information from the narrative and condition state summaries is then used to complete the element level condition report showing quantities at the correct rating value. There are no Element Level Smart Flags specific to culverts.

In an element level condition state assessment of a concrete pipe culvert, the AASHTO CoRe element is:

Element No.	Description
241	Reinforced Concrete Culvert

The quantity unit for culverts is meters or feet of culvert length along the barrel. The total quantity equals the culvert length times the number of barrels. The inspector must visually evaluate each 1 m (1 ft) slice of the culvert barrel(s) and assign the appropriate condition state description. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements condition state assessment method for condition state descriptions.

The condition state descriptions for each slice are then compiled such that the total quantity of culvert is described by various quantities of culvert length distributed over a range of four condition state descriptions. The sum of the individual condition state quantities must equal the total element quantity.

Dimensions and Approximate Weights of Concrete Pipe

*ASTM C 76 – Reinforced concrete Culvert, Storm Drain and Sewer Pipe, Tongue and Groove Joints						
WALL A			**WALL B**		**WALL C**	
Internal Diameter inches	**Minimum Wall Thickness, inches**	**Approximate Weight, pounds per foot**	**Minimum Wall Thickness, inches**	**Approximate Weight, pounds per foot**	**Minimum Wall Thickness, inches**	**Approximate Weight, pounds per foot**
96	8	2710	9	3090	9 ¾	3355
102	8 ½	3078	9 ½	3480	10 ¼	3760
108	9	3446	10	3865	10 ¾	4160

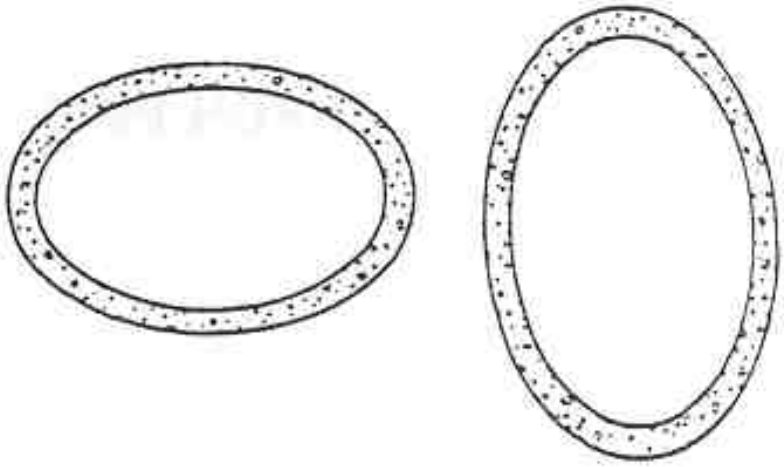
Large Sizes of Pipe Tongue and Groove Joint			
Internal Diameter Inches	**Internal Diameter Feet**	**Wall Thickness Inches**	**Approximate Weight, pounds per foot**
114	9 ½	9 ½	3840
120	10	10	4263
126	10 ½	10 ½	4690
132	11	11	5148
138	11 ½	11 ½	5627
144	12	12	6126
150	12 ½	12 ½	6647
156	13	13	7190
162	13 ½	13 ½	7754
168	14	14	8339
174	14 ½	14 ½	8942
180	15	15	9572

* For description of ASTM C 76 see page 12.3.20

Figure 12.3.14 Standard Sized for Concrete Pipe (Source: American Concrete Pipe Association)

Typical Cross Section of Arch Pipe

Horizontal and Vertical Ellipse Pipe



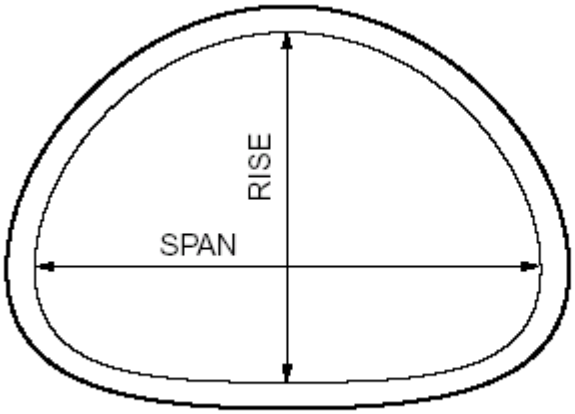
Dimensions and Approximate Weights of Elliptical Concrete Pipe

*ASTM C 507 – Reinforced Concrete Elliptical Culvert, Storm Drain and Sewer Pipe					
Equivalent Round Size, inches	**Minor Axis, inches**	**Major Axis, inches**	**Minimum Wall Thickness, inches**	**Water-Way Area, square feet**	**Approximate Weight, pounds per foot**
96	77	121	9 ½	52.4	3420
102	82	128	9 ¾	59.2	3725
108	87	136	10	66.4	4050
114	92	143	10 ½	74.0	4470
120	97	151	11	82.0	4930
132	106	166	12	99.2	5900
144	116	180	13	118.6	7000

* For description of ASTM C 507 see page 12.3.20

Figure 12.3.14 Standard Sized for Concrete Pipe (Source: American Concrete Pipe Association), continued

Typical Cross Section of Arch Pipe



Dimensions and Approximate Weights of Concrete Arch Pipe

*ASTM C 506 – Reinforced Concrete Arch Culvert, Storm Drain and Sewer Pipe					
Equivalent Round Size, inches	**Minimum Rise, inches**	**Minimum Span, inches**	**Minimum Wall Thickness, inches**	**Water-Way Area, square feet**	**Approximate Weight, pounds per foot**
96	77 1/4	122	9	51.7	3110
108	87 1/8	138	10	66.0	3850
120	96 7/8	154	11	81.8	5040
132	106 1/2	168 3/4	10	99.1	5220

* For description of ASTM C 506 see page 12.3.20

Figure 12.3.14 Standard Sized for Concrete Pipe (Source: American Concrete Pipe Association), continued

American Society for Testing and Materials (ASTM) descriptions for select rigid pipe culverts

ASTM C 76 Reinforced concrete Culvert, Storm Drain, and Sewer Pipe: Covers reinforced concrete pipe intended to be used for the conveyance of sewage, industrial wastes, and storm waters, and for the construction of culverts. Class I – 60 inches through 144 inches in diameter; Class II, III, IV and V – 12 inches through 144 inches in diameter. Larger sizes and higher classes are available as special designs.

ASTM C 506 Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe: Covers pipe to be used for the conveyance of sewage, industrial waste, and storm water and for the construction of culverts in sizes from 15 inch through 132 inch equivalent circular diameter. Larger sizes are available as special designs.

ASTM C 507 Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe: Covers reinforced elliptically shaped concrete pipe to be used for the conveyance of sewage, industrial waste and storm water, and for the construction of culverts. Five standard classes of horizontal elliptical, 18 inches through 144 inches in equivalent circular diameter and five standard classes of vertical elliptical, 36 inches through 144 inches in equivalent circular diameter are included. Larger sizes are available as special designs.

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Topic 12.4 Flexible Culverts

12.4.1 Introduction

Like all culverts, flexible culverts are designed for full flow. Unlike bridges, culverts have no distinction between substructure and superstructure and because earth backfill separates the culvert structure from the riding surface, culverts have no "deck." Most flexible culverts have a circular or elliptical configuration but some flexible box and arch culverts exist. From their design nature, flexible culverts have little structural bending strength on their own. The material from which they are made, such as corrugated steel or aluminum can be flexed or bent and can be distorted significantly without cracking. Consequently, flexible culverts depend on the backfill support to resist bending. In flexible culvert designs, proper interaction between the soil and structure is critical.

This Topic discusses the characteristics, inspection, and evaluation of flexible culverts (see Figure 12.4.1).



Figure 12.4.1 Pipe Arch Flexible Culvert

12.4.2 Design Characteristics

Structural Behavior

A flexible culvert is a composite structure made up of the culvert barrel and the surrounding soil. The barrel and the soil are both vital elements to the structural performance of the culvert.

Flexible pipe has relatively little bending stiffness or bending strength on its own. Flexible culvert materials include steel, aluminum, and plastic. As loads are applied to the culvert, it attempts to deflect. In the case of a round pipe, the vertical diameter decreases and the horizontal diameter increases (see Figure 12.4.2). When good embankment material is well compacted around the culvert, the increase in horizontal diameter of the culvert is resisted by the lateral soil pressure. With round pipe the result is a relatively uniform radial pressure around the pipe which creates a compressive thrust in the pipe walls. As illustrated in Figure P.3.23, the compressive thrust is approximately equal to vertical pressure times one-half the span length.

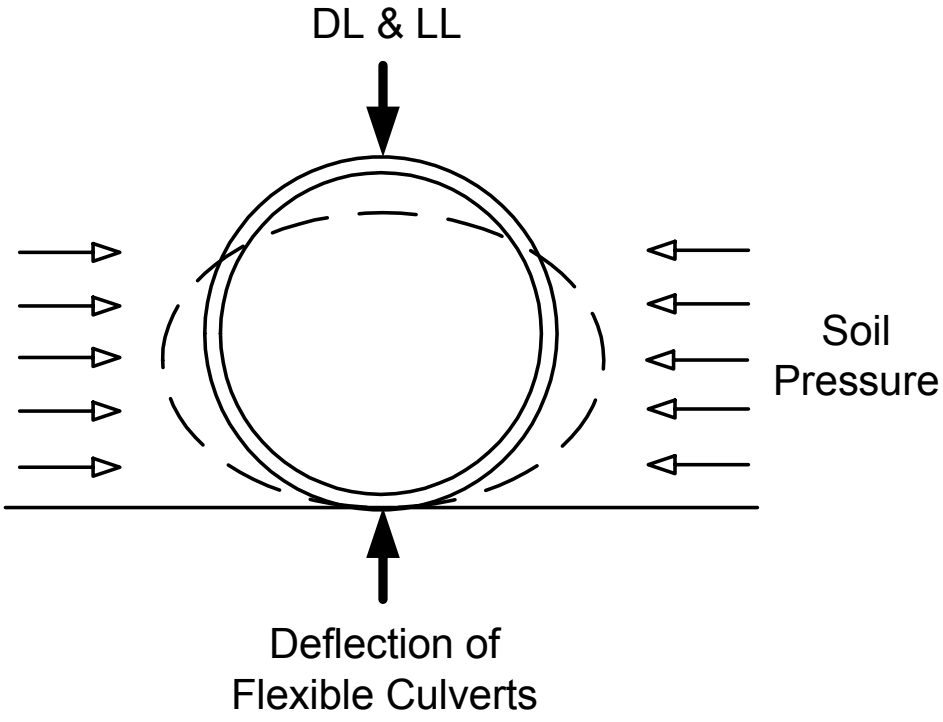
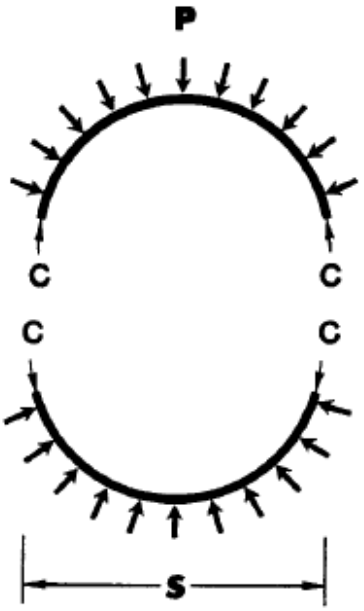


Figure 12.4.2 Flexible Culvert Deflection



SUMMING THE VERTICAL FORCES ON HALF OF THE PIPE AT A TIME SHOWS THAT

$$C = P \times \frac{S}{2}$$

WHERE

C= COMPRESSIVE THRUST IN THE CULVERT WALL

P= SUM OF SOIL PRESSURE ACTING ON THE CULVERT

S= THE SPAN OR DIAMETER

$\frac{S}{2}$= THE RADIUS (R)

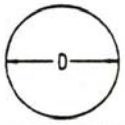
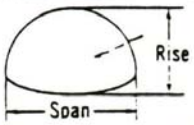
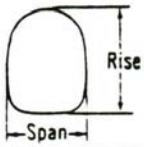
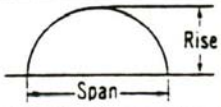
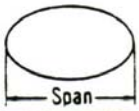
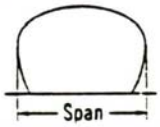
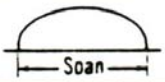
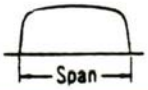
Figure 12.4.3 Formula for Ring Compression

An arc of a flexible round pipe, or other shape will be stable as long as adequate soil pressures are achieved, and as long as the soil pressure is resisted by the compressive force C on each end of the arc. Good quality backfill material and proper installation are critical in obtaining a stable soil envelope around a flexible culvert.

In long span culverts the radius (R) is usually large. To prevent excessive deflection due to dead and/or live loads, longitudinal or circumferential stiffeners are sometimes added. The circumferential stiffeners are usually metal ribs bolted to the outside of the culvert. Longitudinal stiffeners may be metal or reinforced concrete. Concrete thrust beams provide some circumferential stiffening as well as longitudinal stiffening. The thrust beams are added to the structure prior to backfill. They also provide a solid vertical surface for soil pressures to act on and a surface which is easier to backfill against. The use of concrete stress relieving slabs is another method used to achieve longer spans or reduce minimum cover. A stress-relieving slab is cast over the top of the backfill above the structure to distribute live loads to the adjacent soil.

12.4.3 Types and Shapes of Flexible Culverts

Flexible culverts are constructed from corrugated steel or aluminum pipe or field assembled structural plate products. Structural plate steel products are available as structural plate pipes, box culverts, or long span structures. See Figure 12.4.4 for standard shapes for corrugated flexible culverts.

Shape	Range of Sizes	Common Uses
Round 	6 in–26 ft	Culverts, subdrains, sewers, service tunnels, etc. All plates same radius. For medium and high fills (or trenches).
Vertically-elongated (ellipse) 5% is common 	4–21 ft nominal: before elongating	Culverts, sewers, service tunnels, recovery tunnels. Plates of varying radii; shop fabrication. For appearance and where backfill compaction is only moderate.
Pipe-arch 	Span x Rise 18 in. x 11 in. to 20 ft 7 in. x 13 ft 2 in.	Where headroom is limited. Has hydraulic advantages at low flows. Corner plate radius, 18 inches or 31 inches for structural plate.
Underpass* 	Span x Rise 5 ft 8 in. x 5 ft 9 in. to 20 ft 4 in. x 17 ft 9 in.	For pedestrians, livestock or vehicles (structural plate).
Arch 	Span x Rise 6 ft x 1 ft 9½ in. to 25 ft x 12 ft 6 in.	For low clearance large waterway opening, and aesthetics (structural plate).
Horizontal Ellipse 	Span 20–40 ft	Culverts, grade separations, storm sewers, tunnels.
Pear 	Span 25–30 ft	Grade separations, culverts, storm sewers, tunnels.
High Profile Arch 	Span 20–45 ft	Culverts, grade separations, storm sewers, tunnels. Ammo ammunition magazines, earth covered storage.
Low Profile Arch 	Span 20–50 ft	Low-Wide waterway enclosures, culverts, storm sewers.
Box Culverts 	Span 10–21 ft	Low-wide waterway enclosures, culverts, storm sewers.
Specials	Various	For lining old structures or other special purposes. Special fabrication.

*For equal area or clearance, the round shape is generally more economical and simpler to assemble.

Figure 12.4.4 (Exhibit 11 Culvert Inspection Manual Report No. FHWA-IP-86-2) Standard Corrugated Steel Culvert Shapes (Source: Handbook of Steel Drainage and Highway Construction Products, American Iron and Steel Institute)

Corrugated Pipe

Factory-made pipe is produced in two basic shapes: round and pipe arch. Both shapes are produced in several wall thicknesses, several corrugation sizes, and with annular (circumferential) or helical (spiral) corrugations. Pipes with helical corrugations have continuously welded seams or lock seams. Both round and arch steel pipe shapes are available in a wide range of standard sizes. Round pipe is available in standard sizes up to 3.7 m (12 feet) in diameter. Standard sizes for pipe arch are available in sizes up to the equivalent of a 3 m (10 feet) diameter round pipe. Round aluminum pipe is available in standard sizes up to3 m (10 feet) in nominal diameter. Aluminum pipe arch is available in sizes up to the equivalent of an 2.4 m (8 feet) diameter round pipe.

Flexible aluminum culverts are constructed from factory assembled corrugated aluminum pipe or field assembled from structural plates. Structural plate aluminum culverts are available as conventional structural plate structures, box culverts, or long span structures.

Structural Plate

Structural plate steel pipes are field assembled from standard corrugated galvanized steel plates. Standard plates have corrugations with a 150 mm (6-inch) pitch and a depth of 50 mm (2 inches). Plates are manufactured in a variety of thicknesses and are pre-curved for the size and shape of structure to be erected.

Structural steel plate pipes are available in four basic shapes:

- Round
- Pipe arch
- Arch
- Underpass

The standard sizes available range in span from 1.5 to 7.9 m (5 feet to 26 feet).

Structural plate aluminum pipes are field assembled with a 230 mm (9-inch) pitch and a depth of 65 mm (2.5 inches).

Structural plate aluminum pipes are produced in five basic shapes:

- Round
- Pipe arch
- Arch
- Pedestrian/animal underpass
- Vehicle underpass

A wide range of standard sizes is available for each shape. Spans as large as 9.1 m (30 feet) can be obtained for the arch shape.

Box Culverts

Corrugated steel box sections use standard corrugated galvanized steel plates with special reinforcing elements applied to the areas of maximum moments. Steel box culverts are available with spans that range from to 2.9 to 6.3 m (9 feet 8 inches to 20 feet 9 inches).

The aluminum box culvert utilizes standard aluminum structural plates with aluminum rib reinforcing added in the areas of maximum bending stresses. Ribs

are bolted to the exterior of the aluminum shell during installation. Aluminum box culverts are suitable for shallow depths of fill and are available with spans ranging from to 2.7 to 7.7 m (8 feet 9 inches to 25 feet 5 inches).

Long Span Culverts

Long span steel culverts are assembled using conventional 150 by 50 mm (6 by 2 inch) corrugated galvanized steel plates and longitudinal and circumferential stiffening members. There are five standard shapes for long span steel structures:

- Horizontal elliptical
- Pipe arch
- Low profile arch
- High profile arch
- Pear shape

The span lengths of typical sections range from 5.9 to 12.2 m (19 feet 4 inches to 40 feet). Longer spans are available for some shapes as special designs. It should be noted that each long span installation represents, to a certain extent, a custom design. The inspector should therefore use design or as-built plans when checking dimensions of existing long span structures.

Long span aluminum structures are assembled using conventional 230 by 65 mm (9 by 2 1/2 inch) corrugated aluminum plates and aluminum rib stiffeners. Long span aluminum structures are essentially the same size and available in the same five basic shapes as steel long spans.

See the end of this Topic for the different standard sizes for each flexible culvert shape (pg 164-193 Culvert Inspection Manual Report No. FHWA-IP-86-2)

12.4.4 Hazards of Culvert Inspection

The bridge inspector should be alert to the following hazards when inspecting a culvert.

- Inadequate ventilation
- Drowning
- Toxic chemicals
- Animals
- Quick conditions at the outlet
- Insufficient number of inspectors

Refer to Topic 3.2.5 for a detailed discussion of each hazard.

12.4.5 Overview of Common Defects

Common defects that can occur to flexible culvert materials include the following:

- Pitting
- Surface Rust
- Section Loss
- Overload Damage

- Heat Damage
- Buckling
- Embankment erosion at culvert entrance and exit
- Roadway settlement

Refer to Topic 2.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

12.4.6 Inspection Procedures and Locations

Safety is an important reason that culverts should be inspected. For a more detailed discussion on reasons for inspecting culverts, see Topic P.3.1.

A logical sequence for inspecting culverts helps ensure that a thorough and complete inspection will be conducted. In addition to the culvert components, the inspector should also look for highwater marks, changes in the drainage area, settlement of the roadway, and other indications of potential problems. In this regard, the inspection of culverts is similar to the inspection of bridges.

For typical installations, it is usually convenient to begin the field inspection with general observations of the overall condition of the structure and inspection of the approach roadway. The inspector should select one end of the culvert and inspect the embankment, waterway, headwalls, wingwalls, and culvert barrel. The inspector should then move to the other end of the culvert. The following sequence is applicable to all culvert inspections:

- Review available information
- Observe overall condition
- Inspect approach roadway and embankment
- Inspect waterway (see in Topic 11.2)
- Inspect end treatments
- Inspect culvert barrel

Procedures

Visual

Most defects in flexible culverts 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. The types of defects to look for when inspecting the culvert barrel will depend upon the type of culvert being inspected. In general, corrugated metal culvert barrels should be inspected for cross-sectional shape and barrel defects such as joint defects, seam defects, plate buckling, lateral shifting, missing or loose bolts, corrosion, excessive abrasion, material defects, and localized construction damage. A critical area for the inspection of long span metal culverts is at the 2 o'clock and 10 o'clock locations. An inward bulge at these locations may indicate potential failure of the structure.

Physical

In a steel culvert, the bolts on the longitudinal seams should be checked by tapping the nuts with a hammer. For aluminum structural plate, the bolts should be checked with a torque wrench.

A geologist's pick hammer can be used to scrape off heavy deposits of rust and scale. The hammer can then be used to locate areas of corrosion by striking the culvert walls. The walls will deform or the hammer will break through if severe corrosion exists.

Sometimes surveying the culvert is necessary to determine if there is any shape distortion, and if there is distortion how much exists.

Advanced Inspection Techniques

In steel culverts, visual inspections can only point out surface defects. Therefore, advanced inspection techniques may be used to achieve a more rigorous and thorough inspection of the steel culvert, including:

Nondestructive Testing Methods:

- Acoustic Emissions Testing
- Computer Programs
- Computer Tomography
- Corrosion Sensors
- Dye Penetrant
- Magnetic Flux Leakage
- Radiographic Testing
- Robotic Inspection
- Ultrasonic Testing
- Eddy Current

Other Testing Methods

- Brinell Hardness Test
- Charpy Impact Test
- Chemical Analysis
- Tensile Strength Test

See Topic 13.3 for Advanced Inspection Techniques

Locations

Inspect End Treatments

End treatments should be inspected like any other structural component. Their effectiveness can directly affect the performance of the culvert.

The most common types of end treatments for flexible culverts are:

- Projections
- Mitered
- Pipe end section

Projections - The inspector should indicate the location and extent of any undercutting around the ends of the barrel.

The depth of any scouring should be measured with a probing rod. In low flow conditions scour holes have a tendency to fill up with debris or sediment. If no

probing rod is used an inspector could mistakenly report less scour than has taken place.

Water flowing along the outside of a culvert can remove supporting material. This is referred to as piping and it can lead to the culvert end being unsupported. If not repaired in time, piping can cause cantilevered end portions of the culvert to bend down and restrict the stream flow.

Mitered Ends - Inspection items for mitered ends are the same as for projecting ends. Additional care should be taken to measure any deformation of the end. Mitering the end of corrugated pipe culvert reduces its structural capacity.

Pipe End Sections - These are typically used on relatively smaller culverts. For inspection purposes, treat the pipe end section as you would a projected end.

The following excerpts are from a reproduction of the out-of-print Culvert Inspection Manual Report No.-IP-86-2 – Chapter 5, Section 4.

Section 4 - CORRUGATED METAL CULVERTS

5-4.0 General

Corrugated aluminum and corrugated steel culverts are classified as flexible structures because they respond to and depend upon the soil backfill to provide structural stability and support to the culvert. The flexible corrugated metal acts essentially as a liner. The liner acts mainly in compression and can carry large ring compression thrust, but very little bending or moment force. (Rib reinforced box culverts are exceptions.) Inspection of the culvert determines whether the soil envelope provides adequate structural stability for the culvert and verifies that the "liner" is capable of carrying the compressive forces and protecting the soil backfill from water flowing through the culvert. Verification of the stability of the soil envelope is accomplished by checking culvert shape. Verification of the integrity of the "liner" is accomplished by checking for pipe and plate culvert barrel defects.

This section contains discussions on inspecting corrugated metal structures for shape and barrel defects. Because shape inspection requirements do vary somewhat for different shapes, separate sections with detailed guidelines are provided for corrugated metal pipe culvert shapes and long-span culvert shapes. Section 5 of this chapter addresses corrugated metal pipe culverts, and section 6 covers long-span corrugated metal culverts.

5-4.1 Shape Inspections

The single most important feature to observe and measure when inspecting corrugated metal culverts is the cross-sectional shape of the culvert barrel. The corrugated metal culvert barrel depends on the backfill or embankment to maintain its proper shape and stability. When the backfill does not provide the required support, the culvert will deflect, settle, or distort. Shape changes in the culvert therefore provide a direct indication of the adequacy and stability of the supporting soil envelope. By periodic observation and measurement of the culvert's shape, it is possible to verify the adequacy of the backfill. The design or theoretical cross-

section of the culvert should be the standard against which field measurements and visual observations are compared. If the design cross section is unknown, a comparison can be made between the unloaded culvert ends and the loaded sections beneath the roadway or deep fills. This can often provide an indication of structure deflection or settlement. Symmetrical shape and uniform curvature around the perimeter are generally the critical factors. If the curvature around the structure becomes too flat, and/or the soil continues to yield under load, the culvert wall may not be able to carry the ring thrust without either buckling inward or deflecting excessively to the point of reverse curvature. Either of these events leads to partial or total failure.

As explained in earlier in this Topic, an arc of a circular pipe or other shape structure will be stable and perform as long as the soil pressure on the outside of the pipe is resisted by the compression force in the pipe at each end of the arc.

Corrugated metal pipes can change shape safely within reasonable limits as long as there is adequate exterior soil pressure to balance the ring compression. Therefore, size and shape measurements taken at any one time do not provide conclusive data on backfill instability even when there is significant deviation from the design shape. Current backfill stability cannot be reliably determined unless changes in shape are measured over time. It is therefore necessary to identify current or recent shape changes to reliably check backfill stability. If there is instability of the backfill, the pipe will continue to change shape.

In general, the inspection process for checking shape will include visual observations for symmetrical shape and uniform curvature as well as measurements of important dimensions. The specific measurements to be obtained depend upon factors such as the size, shape, and condition of the structure. If shape changes are observed, more measurements may be necessary. For small structures in good condition, one or two simple measurements may be sufficient, for example, measuring the horizontal diameter on round pipe. For larger structures such as long span culverts, key measurements may be difficult to obtain. Horizontal diameters may be both high and large. The inspection process for long span culverts generally requires that elevations be established for key points on the structure. Although some direct measurements may also be required for long-span structures, elevations are needed to check for settlement and for calculating vertical distances such as the middle ordinate of the top arc. For structures with shallow cover, observations of the culvert with a few live loads passing over are recommended. Discernible movement in the structure may indicate possible instability and a need for more in-depth investigation.

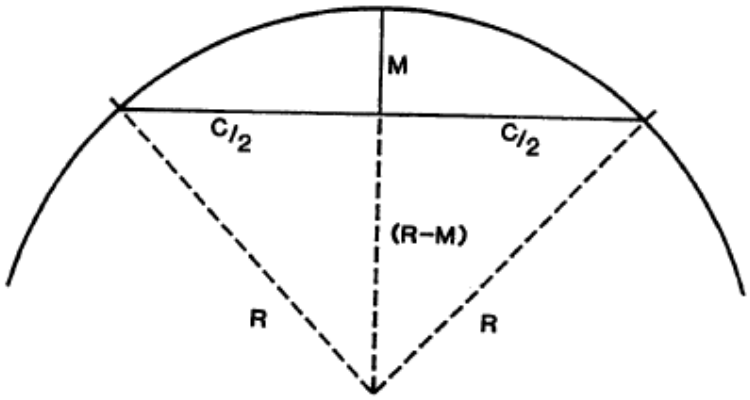
The number of measurement locations depends upon the size and condition of the structure. Long-span culverts should normally be measured at the end and at 7.6 m (25 foot) intervals. Measurements may be required at more frequent intervals if significant shape changes are observed. The smaller pipe culverts can usually be measured at longer intervals than long-span culverts.

Locations in sectional pipe can be referenced by using pipe joints as stations to establish the stationing of specific cross-sections. Stations should start with number 1 at the outlet and increase going upstream to the inlet. The location of points on a circular cross section can be referenced like hours on a clock. The clock should be oriented looking upstream. On structural plate corrugated metal

culverts, points can be referenced to bolted circumferential and longitudinal seams.

It is extremely important to tie down exact locations of measurement points. Unless the same point is checked on each inspection, changes cannot be accurately monitored. The inspection report must, therefore, include precise descriptions of reference point locations. It is safest to use the joints, seams, and plates as the reference grid for measurement points. Exact point locations can then be easily described in the report as well as physically marked on the structures. This guards against loss of paint or scribe marks and makes points easy to find or reestablish. All dimensions in structures should be measured to the inside crest of corrugation. When possible, measurement points on structural plate should be located at the center of a longitudinal seam. However, some measurement points are not on a seam.

When distortion or curve flattening is apparent, the extent of the flattened area, in terms of arc length, length of culvert affected, and the location of the flattened area should be described in the inspection report. The length of the chord across the flattened area and the middle ordinate of the chord should be measured and recorded. The chord and middle ordinate measurements can be used to calculate the curvature of the flattened area using the formula shown in exhibit 66.



C = MEASURED CHORD
M = MEASURED MIDDLE ORDINATE

SOLVE FOR R_A = ACTUAL RADIUS

$$R_A = \frac{4M^2 + C^2}{8M}$$

IF R_A IS > R_D (DESIGN RADIUS) THEN
ACTUAL CURVE IS FLATTER THAN DESIGN

Figure 12.4.5 (Exhibit 66) Checking Curvature by Curve and Middle Ordinate

5-4.2 Inspecting Barrel Defects

The structural integrity of corrugated metal culverts and long-span structures is dependent upon their ability to perform in ring compression and their interaction with the surrounding soil envelope. Defects in the culvert barrel itself, which can influence the culvert's structural and hydraulic performance, are discussed in the following paragraphs. Rating guidelines are provided in the sections dealing with specific shapes.

a. Misalignment - The inspector should check the vertical and horizontal alignment of the culvert. The vertical alignment should be checked visually for sags and deflection at joints. Poor vertical alignment may indicate problems with the subgrade beneath the pipe bedding. Sags trap debris and sediment and may impede flow. Since most highway culverts do not have watertight joints, sags which pocket water could saturate the soil beneath and around the culvert, reducing the soil's stability. The horizontal alignment should be checked by sighting along the sides for straightness. Vertical alignment can be checked by sighting along bolt lines. Minor horizontal and vertical misalignment is generally not a significant problem in corrugated metal structures unless it causes shape or joint problems. Occasionally culverts are intentionally installed with a change in gradient.

b. Joint Defects - Field joints are generally only found with factory manufactured pipe. There are ordinarily no joints in structural plate culverts, only seams. (In a few cases, preassembled lengths of structural plate pipe have been coupled or banded together like factory pipe.)

Field joints in factory pipe serve to maintain the water conveyance of the culvert from section to section, to keep the pipe sections in alignment, keep the backfill soil from infiltrating, and to help prevent sections from pulling apart. Joint separation may indicate a lack of slope stability as described in section 5-4.2 e., circumferential seams. Key factors to look for in the inspection of joints are indications of backfill infiltration and water exfiltration. Excessive seepage through an open joint can cause soil infiltration or erosion of the surrounding backfill material reducing lateral support. Open joints may be probed with a small rod or flat rule to check for voids. Indications of joint defects include open joints, deflection, seepage at the joints, and surface sinkholes over the culvert as illustrated in exhibits 67 and 68. Any evidence of joint defects should be recorded. Culverts in good condition should have no open joints, those in fair condition may have a few open joints but no evidence of soil infiltration, and those in marginal to poor condition will show evidence of soil infiltration.

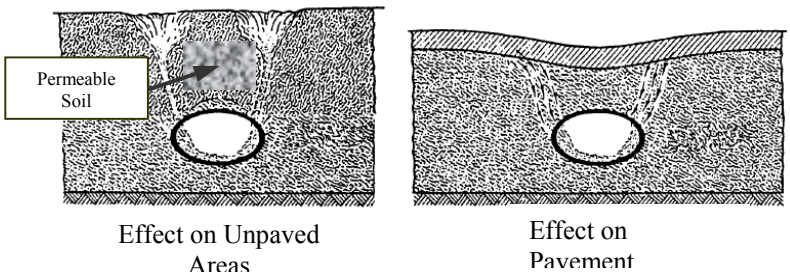


Figure 12.4.6 (Exhibit 67) Surface Indications of Infiltration



Figure 12.4.7 (Exhibit 68) Surface Hole Above Open Joint

c. Seam Defects in Fabricated Pipe - Pipe seams in helical pipe do not carry a significant amount of the ring compression thrust in the pipe. That is the reason that a lock seam is an acceptable seam. Helical seams should be inspected for cracking and separation. An open seam could result in a loss of backfill into the pipe, or exfiltration of water. Either condition could reduce the stability of the surrounding soil.

In riveted or spot welded pipes, the seams are longitudinal and carry the full ring compression in the pipe. These seams, then, must be sound and capable of handling high compression forces. They should be inspected for the same types of defects as those described in the text for structural plate culverts, Section 12.4.3, Structural Pipe. When inspecting the longitudinal seams of bituminous-coated corrugated metal culverts, cracking in the bituminous coating may indicate seam separation.

d. Longitudinal Seam Defects in Structural Plate Culverts - Longitudinal seams should be visually inspected for open seams, cracking at bolt holes, plate distortion around the bolts, bolt tipping, cocked seams, cusped seams, and for significant metal loss in the fasteners due to corrosion.

Culverts in good condition should have only minor joint defects. Those in fair condition may have minor cracking at a few bolt holes or minor opening at seams that could lead to infiltration or exfiltration. Marginal to poor culvert barrel conditions are indicated by significant cracking at bolt holes, or deflection of the structure due to infiltration of backfill through an open seam. Cracks 3 inches (76 mm) long on each side of the bolts indicate very poor to critical conditions.

(1) Loose Fasteners - Seams should be checked for loose or missing fasteners as shown in exhibit 69. For steel structures the longitudinal seams are bolted together with high-strength bolts in

two rows; one row in the crests and one row in the valleys of the corrugations. These are bearing type connections and are not dependent on a minimum clamping force of bolt tension to develop interface friction between the plates. Fasteners in steel structural plate may be checked for tightness by tapping lightly with a hammer and checking for movement.

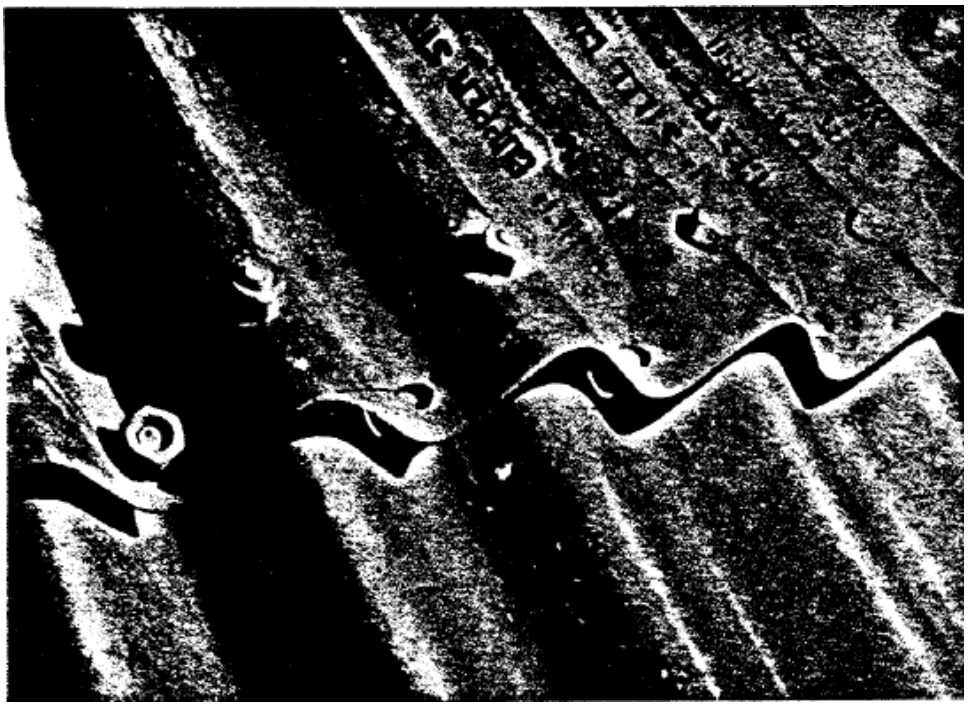


Figure 12.4.8 (Exhibit 69) Close-Up of Loose and Missing Bolts at a Cusped Seam; Loose Fasteners are Usually Detected by Tapping the Nuts with a Hammer

For aluminum structural plate, the longitudinal seams are bolted together with normal strength bolts in two rows with bolts in the crests and valleys of both rows. These seams function as bearing connections, utilizing bearing of the bolts on the edges of holes and friction between the plates. The seams in aluminum structural plate should be checked with a torque wrench (125 ft-lbs (169 Joules) minimum to 150 ft-lbs (203 Joules) maximum). If a torque wrench is not available fasteners can be checked for tightness with a hammer as described for steel plates.

(2) Cocked and Cusped Seams - The longitudinal seams of structural plate are the principal difference from factory pipe. The shape and curvature of the structure is affected by the lapped, bolted longitudinal seam. Improper erection or fabrication can result in cocked seams or cusped effects in the structure at the seam, as illustrated in exhibit 70. Slight cases of these conditions are fairly common and frequently not significant. However, severe cases can result in failure of the seam or structure. When a cusped seam is significant the structure's shape appearance and key dimensions will differ significantly from the design shape and dimensions. The cusp effect should cause the structure to receive very low

ratings on the shape inspection if it is a serious problem. A cocked seam can result in loss of backfill and may reduce the ultimate ring compression strength of the seam.

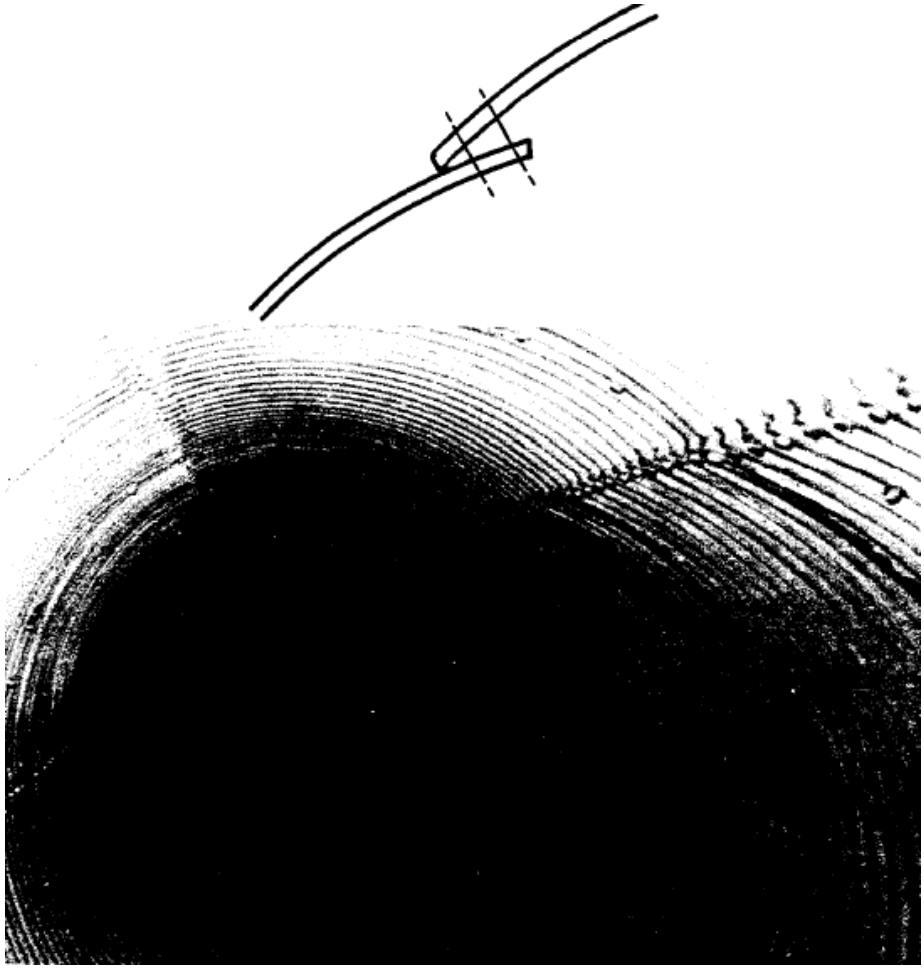


Figure 12.4.9 (Exhibit 70) Cocked Seam with Cusp Effect

(3) Seam Cracking - Cracking along the bolt holes of longitudinal seams can be serious if allowed to progress. As cracking progresses, the plate may be completely severed and the ring compression capability of the seam lost. This could result in deformation or possible failure of the structure. Longitudinal cracks are most serious when accompanied by significant deflection, distortion, and other conditions indicative of backfill or soil problems. Longitudinal cracks are caused by excessive bending strain, usually the result of deflection, exhibit 71. Cracking may occasionally be caused by improper erection practices such as using bolting force to "lay down" a badly cocked seam.

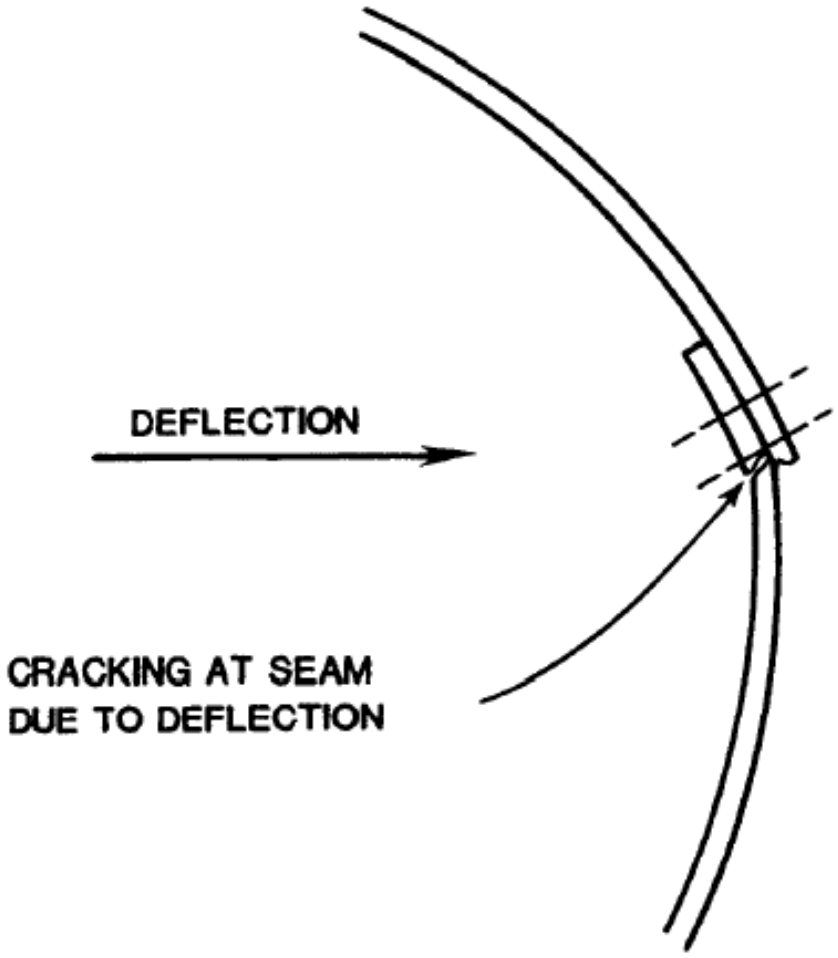


Figure 12.4.10 (Exhibit 71) Cracking Due to Deflection

(4) Bolt Tipping - The bolted seams in structural plate culverts only develop their ultimate strength under compression. Bolt tipping occurs when the plates slip. As the plates begin to slip, the bolts tip, and the bolt holes are plastically elongated by the bolt shank. High compressive stress is required to cause bolt tipping. Structures have rarely been designed with loads high enough to produce a ring compression that will cause bolt tip. However, seams should be examined for bolt tip particularly in structures under higher fills. Excessive compression on a seam could result in plate deformations around the tipped bolts and failure is reached when the bolts are eventually pulled through the plates.

e. Circumferential Seams - The circumferential seams, like joints in factory pipe, do not carry ring compression. They do make the conduit one continuous structure. Distress in these seams is rare and will ordinarily be a result of a severe differential deflection or distortion problem or some other manifestation of soil failure. For example, a steep sloping structure through an embankment may be pulled apart longitudinally if the embankment moves down as shown in exhibit 72. Plates should be installed with the upstream plate overlapping the downstream plate to provide a "shingle" effect in the

direction of flow.

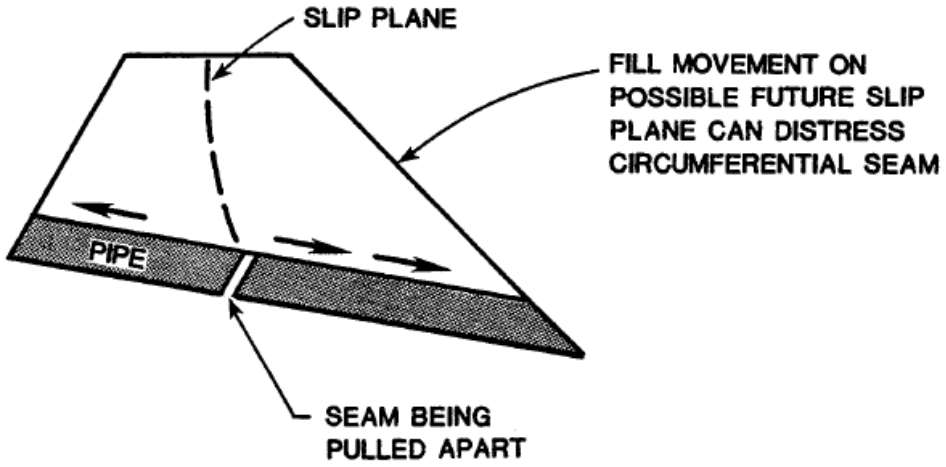


Figure 12.4.11 (Exhibit 72) Circumferential Seam Failure Due to Embankment Slippage

The circumferential seam at one or more locations would be distressed by the movement of the fill. Such distress is important to note during inspections since it would indicate a basic problem of stability in the fill. Circumferential seam distress can also be a result of foundation failure, but in such cases should be clearly evident by the vertical alignment.

f. Dents and Localized Damage - All corrugated metal culverts should be inspected for localized damage. Pipe wall damage such as dents, bulges, creases, cracks, and tears can be serious if the defects are extensive and can impair either the integrity of the barrel in ring compression or permit infiltration of backfill. Small, localized examples are not ordinarily critical. When the deformation type damages are critical, they will usually result in a poorly shaped cross section. The inspector should document the type, extent, and location of all significant wall damage defects. When examining dents in corrugated steel culverts, the opposite side of the plate should be checked, if possible, for cracking or disbonding of the protective coating.

g. Durability (Wall Deterioration) - Durability refers to the ability of a material to resist corrosion and abrasion. Corrosion is the deterioration of metal due to electrochemical or chemical reactions. Abrasion is the wearing away of culvert materials by the erosive action of bedload carried in the stream.

Abrasion is generally most serious in steep or mountainous areas where high flow rates carry sand and rocks that wear away the culvert invert. Abrasion can also accelerate corrosion by wearing away protective coatings.

As discussed in Section P.3.10 and P.3.11 of this manual, metal culverts are subject to corrosion in certain aggressive environments. For example, steel rapidly corrodes in salt water and in environments with highly acidic (low pH) conditions in the soil and water. Aluminum is fairly resistant to salt water but will corrode

rapidly in highly alkaline (high pH) environments, particularly if metals such as iron or copper and their salts are present. The electrical resistivity of soil and water also provide an indication of the likelihood of corrosion. Many agencies have established guidelines in terms of pH and resistivity that are based on local performance. The FHWA has also published guidelines for aluminum and steel culverts including various protective coatings.

Corrosion and abrasion of corrugated metal culverts can be a serious problem with adverse effects on structural performance. Damage due to corrosion and abrasion is the most common cause for culvert replacement. The inspection should include visual observations of metal corrosion and abrasion. As steel corrodes it expands considerably. Relatively shallow corrosion can produce thick deposits of scale. A geologist's pick-hammer can be used to scrape off heavy deposits of rust and scale permitting better observation of the metal. A hammer can also be used to locate unsound areas of exterior corrosion by striking the culvert wall with the pick end of the hammer. When severe corrosion is present, the pick will deform the wall or break through it. Protective coatings should be examined for abrasion damage, tearing, cracking, and removal. The inspector should document the extent and location of surface deterioration problems.

When heavy corrosion is found by observation or sounding, special inspection methods such as pH testing, electrical resistivity measurement, and obtaining cores from the pipe wall are recommended. A routine program for testing pH and electrical resistivity should be considered since it is relatively easy to perform and provides valuable information.

Durability problems are the most common cause for the replacement of pipe culverts. The condition of the metal in corrugated metal culverts and any coatings, if used, should be considered when assigning a rating to the culvert barrel. Suggested rating guidelines for metal culverts with metallic coatings are shown in exhibit 73. Modification of these guidelines may be required when inspecting culverts with non-metallic coatings. Aluminum culvert barrels may be rated as being in good condition if there is superficial corrosion. Steel culverts rated as in good condition may have superficial rust with no pitting. Perforation of the invert as shown in exhibit 74 would indicate poor condition. Complete deterioration of the invert in all or part of the culvert barrel would indicate a critical condition as shown in exhibit 75. Culverts with deteriorated inverts may function as an arch structurally, but are highly susceptible to failure due to erosion of the bedding.

Rating Value	General Description	Corrugated Steel	Corrugated Aluminum
9	New	Near original condition	Near original condition
8	Good	Superficial rust, no pitting	Superficial corrosion slight pitting
7	Generally Good	Moderate rust, slight pitting	Moderate corrosion no attack of core alloy
6	Fair	Fairly heavy rust, moderate pitting, slight thinning	Significant corrosion minor attack of core alloy
5	Generally Fair	Extensive heavy rust, deep pitting, moderate thinning	Significant corrosion moderate attack of core alloy
4	Marginal	Pronounced thinning (some deflection or penetration when struck with pick hammer)	Extensive corrosion significant attack of core alloy
3	Poor	Extensive heavy rust, deep pitting scattered perforations	Extensive corrosion attack of core alloy scattered perforations
2	Critical	Extensive perforations due to rust	Extensive perforations due to corrosion
1	Critical	Invert completely deteriorated	Invert completely deteriorated
0	Critical	Partial or complete collapse	Partial or complete collapse

Figure 12.4.12 (Exhibit 73) Suggested Rating Criteria for Condition of Corrugated Metal

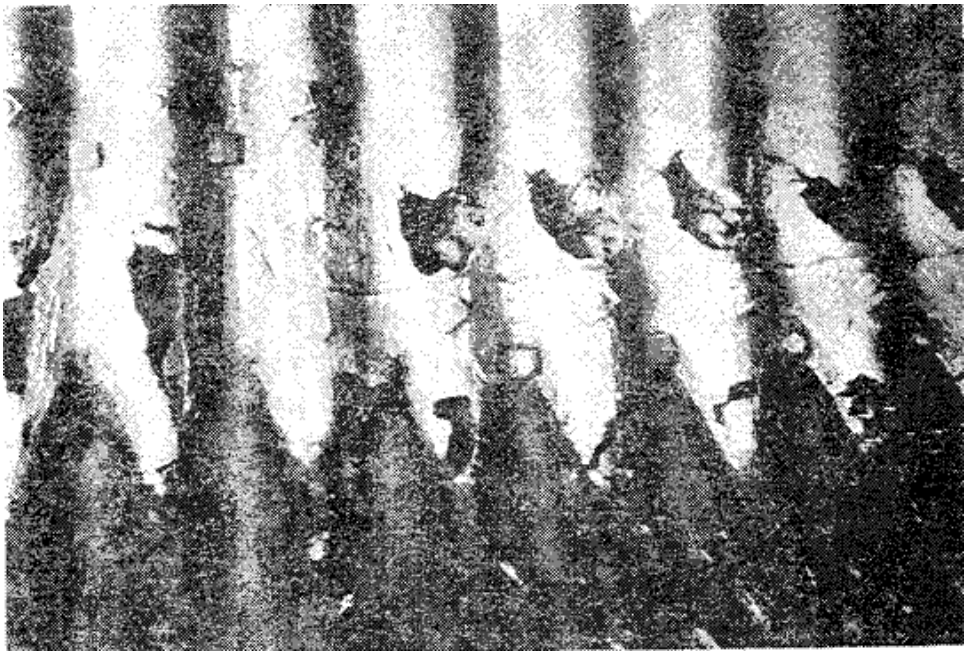


Figure 12.4.13 (Exhibit 74) Perforation of the Invert Due to Corrosion

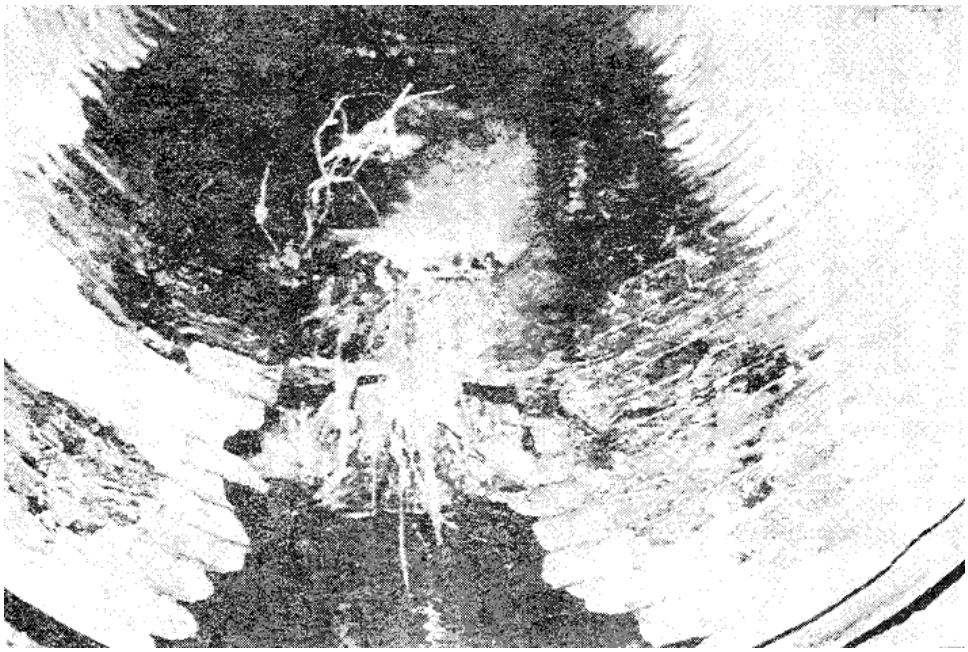


Figure 12.4.14 (Exhibit 75) Invert Deterioration

h. Concrete Footing Defects - Structural plate arches, long-span arches, and box culverts use concrete footings. Metal footings are occasionally used for the arch and box culvert shapes. The metal "superstructure" is dependent upon the footing to transmit the vertical load into the foundation. The structural plate arch is usually bolted in a base channel which is secured in the footing.

The most probable structural defect in the footing is differential settlement. One section of a footing settling more than the rest of the footing can cause wrinkling or other distortion in the arch. Flexible corrugated metal culverts can tolerate some differential settlement but will be damaged by excessive differential settlement. Uniform settlement will not ordinarily affect a metal arch but can affect the clearances in a grade separation structure if the footings settle and the road does not. The significance of differential footing settlement increases as the amount of the difference in settlement increases, the length it is spread over decreases, and the height of the arch decreases. This concept is illustrated in exhibit 76.

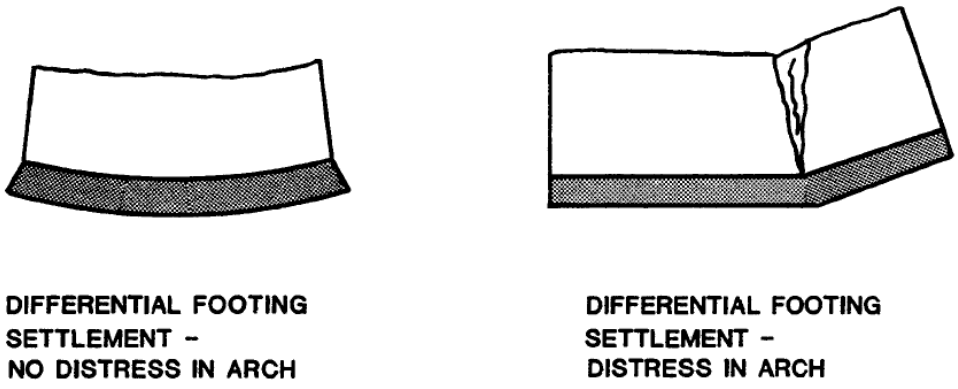


Figure 12.4.15 (Exhibit 76) Differential Footing Settlement

The inspection of footings in structural plate and long-span arches should include a check for differential settlement along the length of a footing. This might show up in severe cracking, spalling, or crushing across the footing at the critical spot. If severe enough, it might be evidenced by compression or stretching of the corrugations in the culvert barrel. Deterioration may occur in concrete and masonry footings which is not related to settlement but is caused by the concrete or mortar. In arches with no invert slab, the inspector should check for erosion and undermining of the footings and look for any indication of rotation of the footing as illustrated in exhibits 77 and 78.

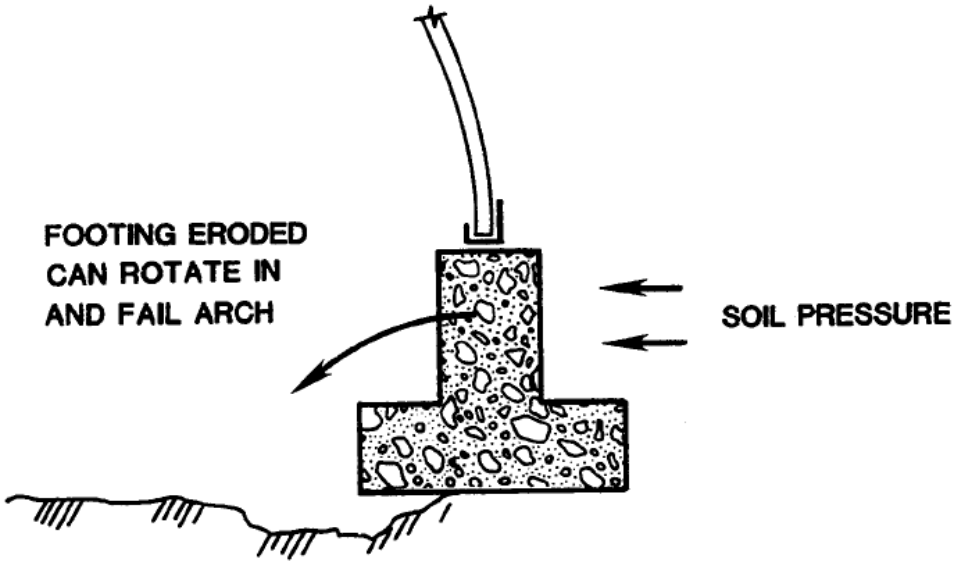


Figure 12.4.16 (Exhibit 77) Footing Rotation due to Undermining

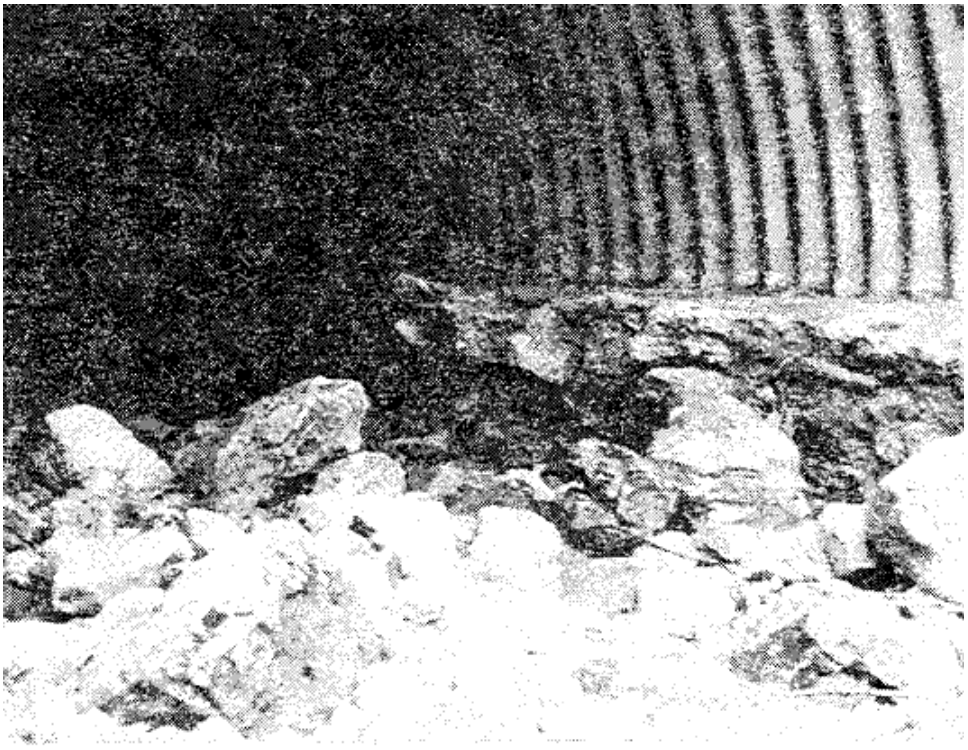


Figure 12.4.17 (Exhibit 78) Erosion of Invert Undermining footing of Arch

Culverts rated in good condition may have minor footing damage. Poor to critical condition would be indicated by severe footing undermining, damage, or rotation, or by differential settlement causing distortion and circumferential kinking in the corrugated metal as shown in exhibit 79.

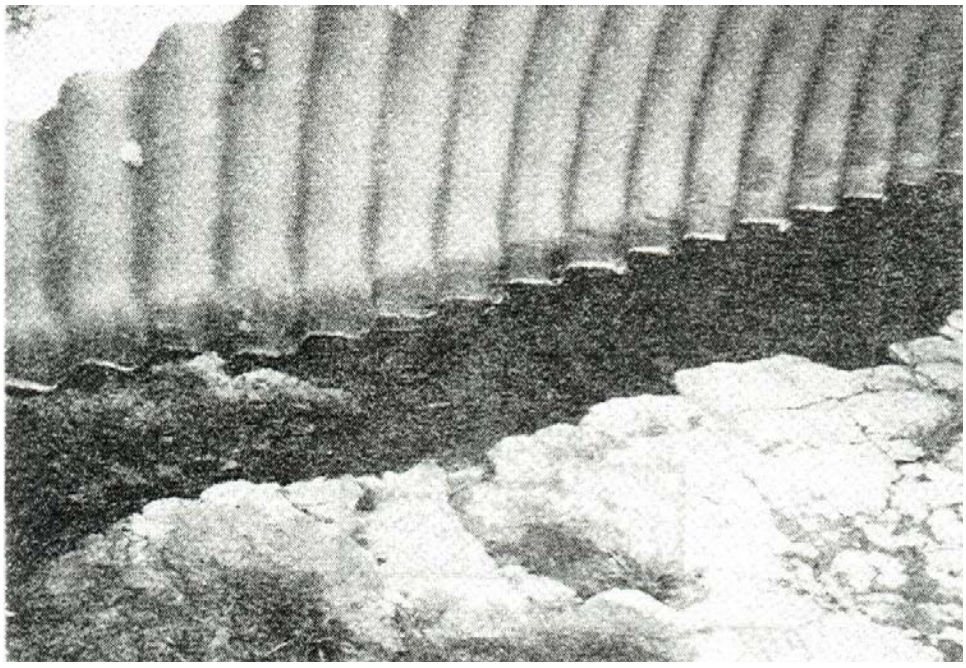


Figure 12.4.18 (Exhibit 79) Erosion Damage to Concrete Invert

i. Defects in Concrete Inverts - Concrete inverts in arches are usually floating slabs used to carry water or traffic. Invert slabs provide protection against erosion and undercutting, and are also used to improve hydraulic efficiency. Concrete inverts are sometimes used in circular, as well as other culvert shapes, to protect the metal from severe abrasive or severe corrosive action. Concrete invert slabs in arches should be checked for undermining and damage such as spalls, open cracks, and missing portions. The significance of damage will depend upon its effect on the footings and corrugated metal.

The following excerpts are from a reproduction of the out-of-print Culvert Inspection Manual (Supplement to Manual 70), July 1986 – Chapter 5, Section 5.

Section 5 - SHAPE INSPECTION OF CORRUGATED METAL CULVERT BARRELS

5-5.0 General

This section deals with shape inspections of common culvert shapes including round and vertical elongated, pipe arches, arches, and box culvert shapes. Specific guidelines for recommended measurements to be taken for each location are provided for each typical culvert shape. Additional measurements are also recommended when field measurements differ from the design dimensions or when significant shape changes are observed. Rating guidelines are also provided for each shape. The guidelines include condition descriptions with shape and barrel defects defined for each rating.

5-5.1 Using the Rating Guidelines

When using the rating guidelines, the inspector should keep the following factors in mind:

a. The inspector should select the lowest rating which best describes either the shape condition or the barrel condition. Structure shape is the most critical factor in flexible culverts, and this should be kept in mind when selecting the rating.

b. The shape criteria described for each numerical rating should be considered as a group rather than as separate criteria for each measurement check listed. Good curvature and the rate of change are critical. Significant changes in shape since the last inspection should be carefully evaluated even if the structure is still in fairly good condition.

c. The guidelines merely offer a starting point for the inspector. The inspector must still use judgment in assigning the appropriate numerical rating. The numerical rating should be related to the actions required. The inspector may wish to refer to Section 4.2 of this manual.

5-5.2 Round and Vertical Elongated Pipe

Round and vertically elongated pipes are expected to deflect vertically during construction resulting in a slightly increased horizontal span. Round pipes are sometimes vertically elongated five percent to compensate for settlement during construction. It is frequently difficult to determine in the field if a pipe was round or elongated when installed. Large round pipes may appear to be elongated if they were subjected to minor flattening of the sides during backfill.

Vehicular underpasses sometimes use 10 percent vertically elongated very large pipe which is susceptible to side flattening during installation. In shallow cover situations, adequate curvature in the sides is the important factor. The soil pressures on the sides may be greater than the weight of the shallow fill over the pipe. The result is a tendency to push the sides inward rather than outward as in deeper buried or round pipes. Side flattening, such as that shown in exhibit 80, can be caused by unstable backfill. A deteriorated invert may have contributed to the problem by reducing the pipe's ability to transmit compressive forces.

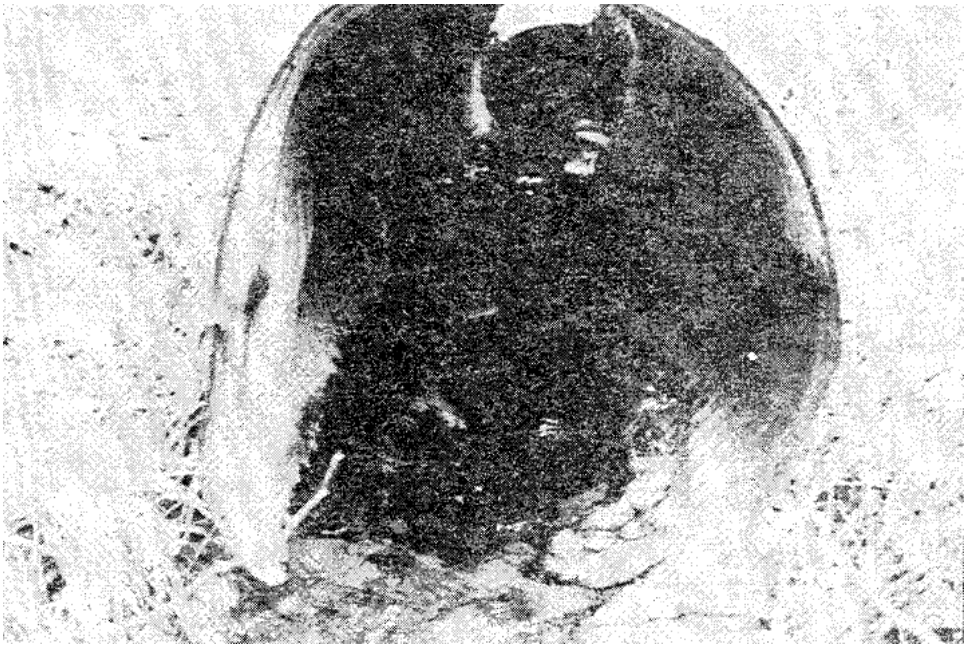


Figure 12.4.19 (Exhibit 80) Excessive Side Deflection

Flattening of the top arc is an indication of possible distress. Flattening of the invert is not as serious. Pipes not installed on shaped bedding will often exhibit minor flattening of the invert arc. However, severe flattening of the bottom arc would indicate possible distress.

The inspector should note the visual appearance of the culvert's shape and measure the horizontal span as shown in exhibit 81. Almost all round or vertical elongated pipe can be directly measured and will not require elevations. Exceptions are large vertical elongated grade separation structures. On such structures, elevations should be obtained similar to those recommended for the long-span pear shape.

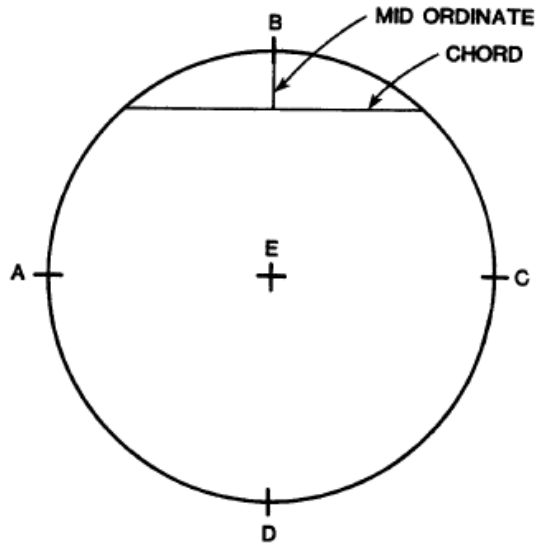


Figure 12.4.20 (Exhibit 81) Shape Inspection Circular and Vertical Elongated Pipe

If the visual appearance or measured horizontal diameter differ significantly from the design specifications, additional measurement, such as vertical diameter, should be taken. Flattened areas should be checked by measuring a chord and the mid ordinate of the chord. The chord length and ordinate measurement should be noted in the report with a description of the location and extent of the flattened area.

Round and vertically elongated pipe with good to fair shape will have a generally good shape appearance. Good shape appearance means that the culvert's shape appears to match the design shape, with smooth, symmetrical curvature and no visible deformations. The horizontal span should be within 10 percent of the design span. Pipe with marginal shape will be indicated by characteristics such as a fair or marginal general shape appearance, distortion in the upper half of the pipe, severe flattening in the lower half of the pipe, or horizontal spans 10 to 15 percent greater than design.

Pipe with poor to critical shape will have a poor shape appearance that does not match the design shape, does not have smooth or symmetrical curvature, and may have obvious deformations. Severe distortion in the upper half of the pipe, a

horizontal diameter more than 15 percent to 20 percent greater than the design diameter, or flattening of the crown to an arc with a radius of 20 to 30 feet or more would indicate poor to critical condition. It should be noted that pipes with deflection of less than 15 to 20 percent may be rated as critical based on poor shape appearance. Guidelines for rating round corrugated metal culvert are presented in exhibit 82.

RATING GUIDELINES FOR ROUND OR VERTICAL ELONGATED CORRUGATED METAL PIPE BARRELS

RATING	CONDITION
9	• New condition
8	• Shape: good, smooth curvature in barrel - Horizontal: within 10 percent of design • Seams and Joints: tight, no openings • Metal: - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting
7	• Shape: generally good, top half of pipe smooth but minor flattening of bottom - Horizontal Diameter: within 10 percent of design • Seams or Joints: minor cracking at a few bolt holes, minor joint or seam openings, potential for backfill infiltration • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting
6	• Shape: fair, top half has smooth curvature but bottom half has flattened significantly - Horizontal Diameter: within 10 percent of design • Seams or Joints: minor cracking at bolts is prevalent in one seam in lower half of pipe. Evidence of backfill infiltration through seams or joints • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting
5	• Shape: generally fair, significant distortion at isolated locations in top half and extreme flattening of invert - Horizontal Diameter: 10 percent to 15 percent greater than design • Seams or Joints: moderate cracking at bolt holes along one seam near bottom of pipe, deflection of pipe caused by backfill infiltration through seams or joints • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting
4	• Shape: marginal significant distortion throughout length of pipe, lower third may be kinked - Horizontal Diameter: 10 percent to 15 percent greater than design • Seams or Joints: Moderate cracking at bolt holes on one seam near top of pipe, deflection caused by loss of backfill through open joints • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting
3	• Shape: poor with extreme deflection at isolated locations, flattening of crown, crown radius 20 to 30 feet - Horizontal Diameter: in excess if 15 percent greater than design • Seams: 3 in. long cracks at bolt holes on one seam • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations
2	• Shape: critical, extreme distortion and deflection throughout pipe, flattening of crown, crown radius over 30 feet - Horizontal Diameter: More than 20 percent greater than design • Seams: plate cracked from bolt to bolt on one seam • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust
1	• Shape: partially collapsed with crown in reverse curve • Seams: failed • Road: closed to traffic
0	• Pipe: totally failed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.21 (Exhibit 82) Condition Rating Guidelines

5-5.3 Pipe Arch

The pipe arch is a completely closed structure but is essentially an arch. The load is transmitted to the foundation principally at the corners. The corners are much like footings of an arch. There is relatively little force or pressure on the large radius bottom plate. The principal type of distress in a pipe arch is a result of inadequate soil support at the corners where the pressure is relatively high. The corner may push down or out into the sail while the bottom stays in place. The effect will appear as if the bottom pushed up. This problem is illustrated in exhibits 83 and 84.

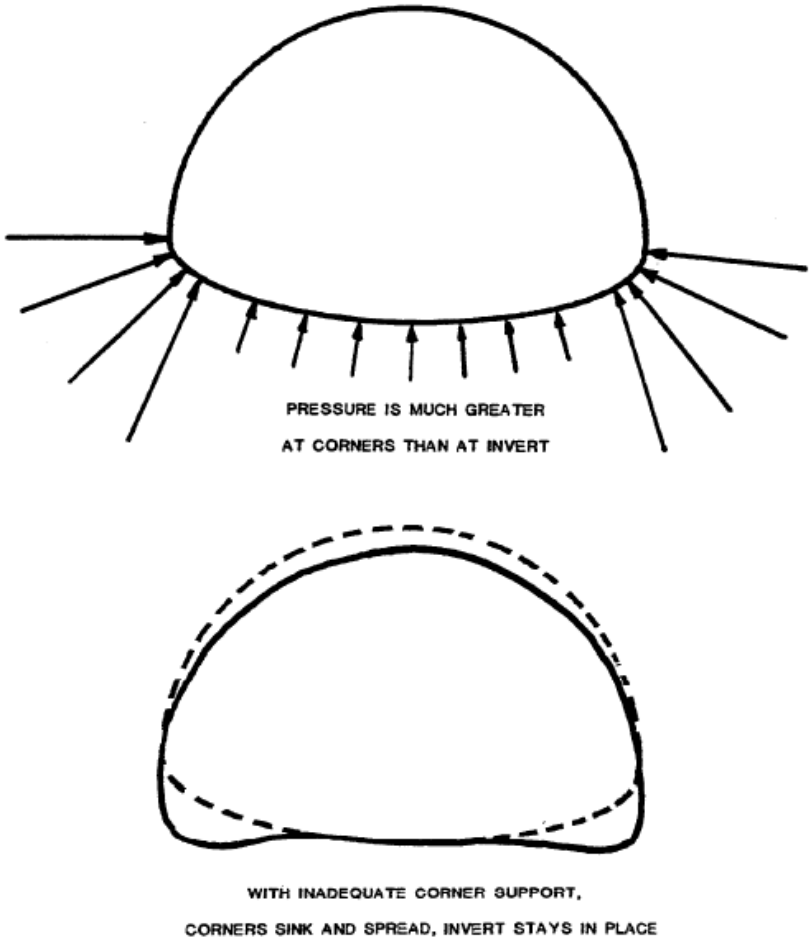


Figure 12.4.22 (Exhibit 83) Bottom Distortion in Pipe Arches

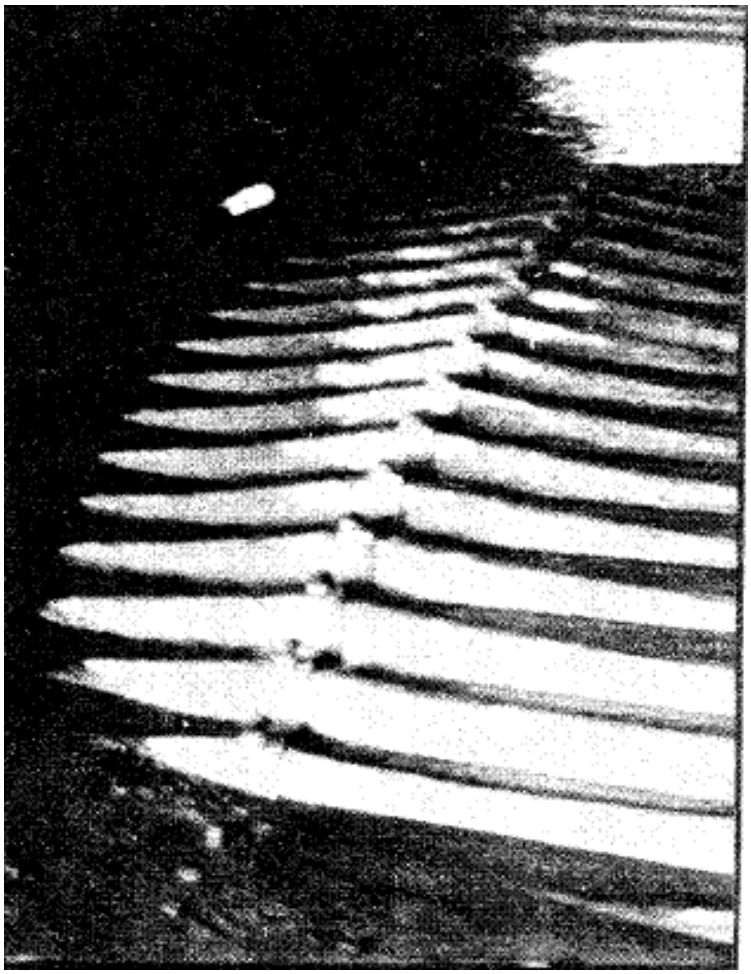


Figure 12.4.23 (Exhibit 84) Bottom and Corners of this Pipe Arch have Settled

The bottom arc should be inspected for signs of flattening and the bottom corners for signs of spreading. The extent and location of bottom flattening and corner spreading should be noted in the inspection report.

Complete reversal of the bottom arc can occur without failure if corner movement into the foundation has stabilized. The top arc of the structure is supporting the load above and its curvature is an important factor. However, if the "footing" corner should fail, the top arc would also fail. The spreading of the corners is therefore very important as it affects the curvature of the top arc.

The inspector should record the visual appearance of the shape and measure both the span and the rise. If the span exceeds the design span by more than 3 percent, the span of the top arc, the mid ordinate of the top arc, and the mid ordinate of the bottom arc should also be measured. Recommended measurements are shown in exhibit 85.

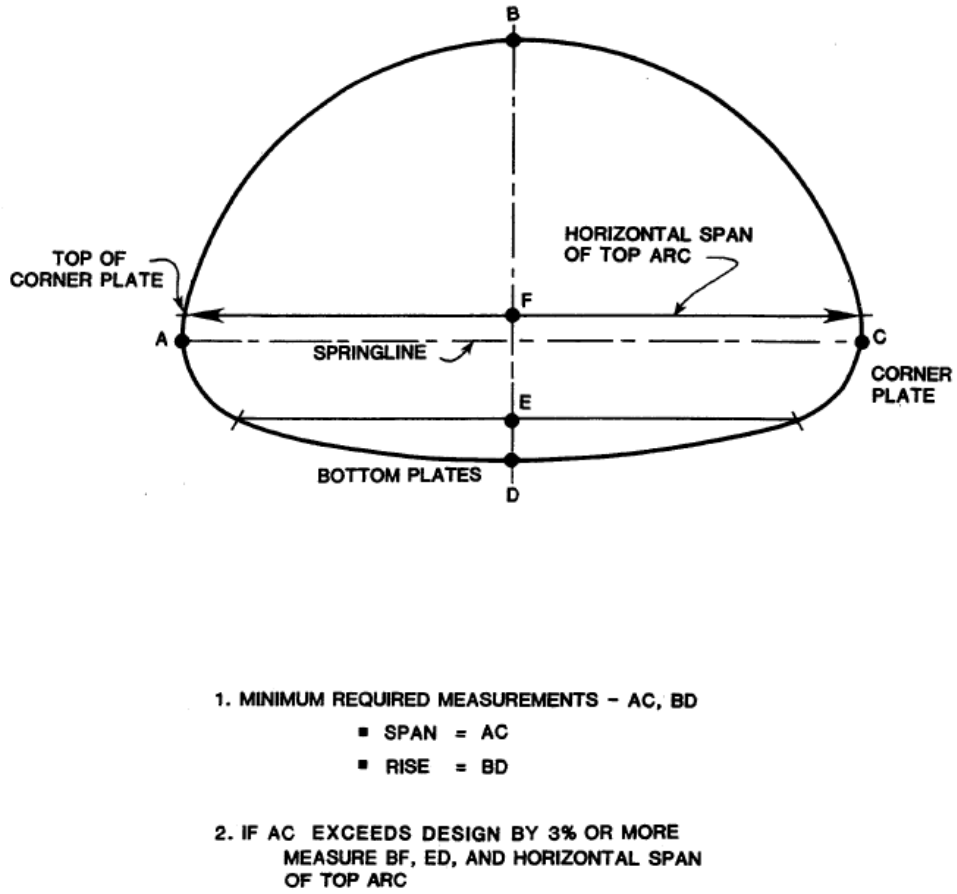


Figure 12.4.24 (Exhibit 85) Shape Inspection Structural Plate Pipe Arch

Pipe arches in fair to good condition will have a symmetrical appearance, smooth curvature in the top of the pipe, and a span less than five percent greater than theoretical. The bottom may be flattened but should still have curvature. Pipe arches in marginal condition will have fair to marginal shape appearance, with distortion in the top half of the pipe, slight reverse curvature in the bottom of the pipe, and a horizontal span five to seven percent greater than theoretical. Pipe in poor to critical condition will have characteristics such as a poor shape appearance, severe deflection or distortion in the top half of the pipe, severe reverse curvature in the bottom of the pipe, flattening of one side, flattening of the crown to an arc with a radius of 6.1 to 9.1 m (20 to 30 feet), or a horizontal span more than seven percent greater than theoretical. Guidelines for rating pipe arches are shown in exhibit 86.

RATING GUIDELINES FOR CORRUGATED METAL PIPE-ARCH BARRELS

RATING	CONDITION	RATING	CONDITION
9	• New condition	4	• Shape: marginal, significant distortion all along top of arch, bottom has reverse curve - Horizontal Span: more than 7 percent greater than design • Joints and Seams: moderate cracking all along one seam; backfill infiltration causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting
8	• Shape: good with smooth curvature - Horizontal Span: less than 3 percent greater than design • Joints or Seams: good condition • Metal: minor construction defects, protective coatings intact - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting		
7	• Shape: generally good, smooth curvature in top half, bottom flattened but still curved - Horizontal Span: within 3 to 5 percent greater than design • Joints or Seams: minor cracking at a few bolt holes; minor joint or seam openings, infiltration of backfill possible • Metal: protective coating ineffective - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting	3	• Shape: poor, extreme deflection in top arch in one section; bottom has reverse curvature throughout - Horizontal Span: more than 7 percent greater than design • Seams: seam cracked 3 in. on each side of bolt holes • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations
6	• Shape: fair, smooth curvature in top half, bottom flat - Horizontal Span: 5 percent greater than design • Joints or Seams: minor cracking all along one seam; minor joint openings with evidence of infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting	2	• Shape: critical, extreme deflection along top of pipe - Horizontal Span: more than 7 percent greater than design • Seams: seam cracked from bolt to bolt down one seam • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust
5	• Shape: generally fair, significant distortion in top in one location; bottom has slight reverse curvature in one location - Horizontal Span: within 5 to 7 percent greater than design • Joints and Seams: moderate cracking at bolt holes along a seam in one section, backfill being lost through seam or joint causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting	1	• Shape: structure partially collapsed • Seams: seam failed • Road: closed to traffic
		0	• Shape: structure collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.25 (Exhibit 86) Condition Rating Guidelines

5-5.4 Arches.

Arches are fixed on concrete footings, usually below or at the springline. The springline is a line connecting the outermost points on the sides of a culvert. This difference between pipes and arches means that an arch tends to deflect differently during backfill. Backfill forces tend to flatten the arch sides and peak its top because the springline cannot move inward like the wall of a round pipe as shown in exhibit 87. As a result, important shape factors to look for in an arch are flattened sides, peaked crown, and flattened top arc.

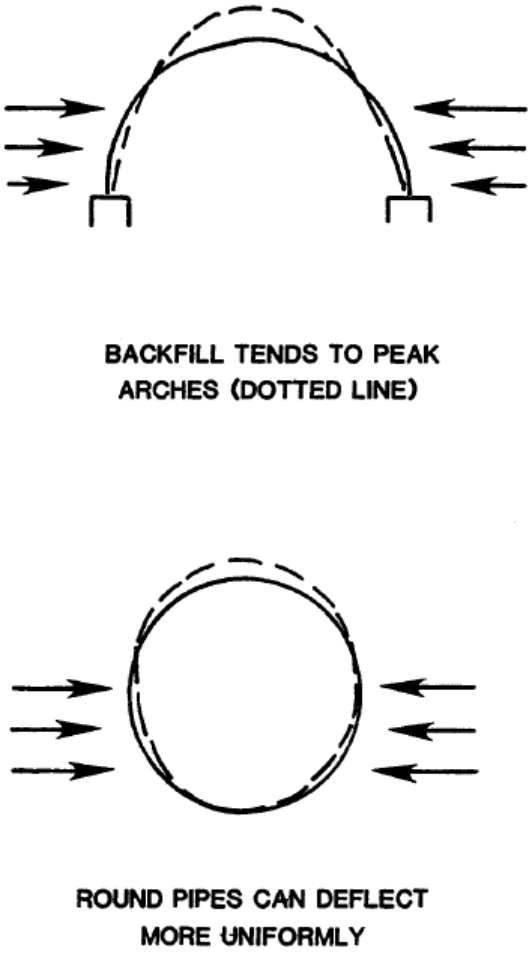


Figure 12.4.26 (Exhibit87) Arch Deflection During Installation

Another important shape factor in arches is symmetrical shape. If the arch was erected with the base channels not square to the centerline, it causes a racking of the cross section. A racked cross-section is one that is not symmetrical about the centerline of the culvert. One side tends to flatten, the other side tends to curve more while the crown moves laterally and possibly upward. If these distortions are not corrected before backfilling the arch, they usually get worse during backfill. Exhibit 88 illustrates racked or peaked arches.

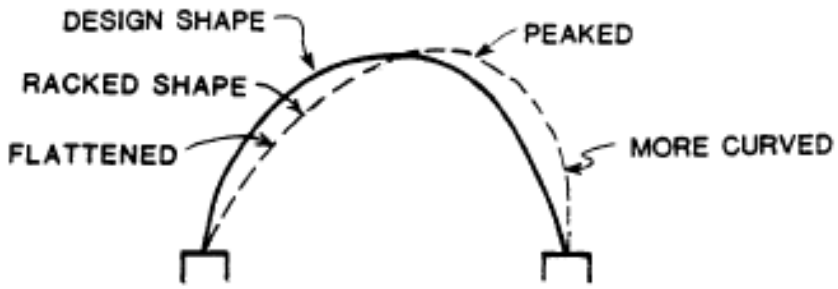
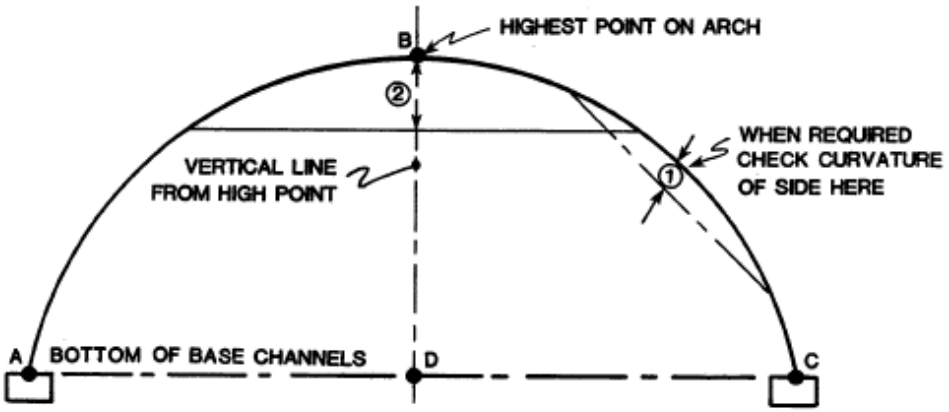




Figure 12.4.27 (Exhibit 88) Racked and Peaked Arch

Visual observation of the shape should involve looking for flattening of the sides, peaking or flattening of the crown, or racking to one side. The measurements to be recorded are illustrated in exhibit 89. Minimum measurements include the vertical distance from the crown to the bottom of the base channels and the horizontal distances from each of the base channels to a vertical line from the highest point on the crown. These horizontal distances should be equal. When they differ by more than 10 inches or 5 percent of the span, whichever is less, racking has occurred and the curvature on the flatter side of the arch should be checked by recording chord and midordinate measurements. Racking can occur when the rise checks with the design rise. When the rise is more than 5 percent less than the design rise, the curvature of the top arc should be checked.



1. MINIMUM REQUIRED MEASUREMENTS
   - SPAN = AD + DC
   - RISE = BD

2. MINIMUM REQUIRED ELEVATIONS – B

3. IF BD GREATER THAN DESIGN BY 5% OR MORE
   CHECK SIDE CURVATURE

4. IF AD AND DC NOT EQUAL CHECK SIDE CURVATURE ①

5. IF BD LESS THAN DESIGN BY 5% OR MORE
   CHECK TOP CURVATURE ②

Figure 12.4.28 (Exhibit 89) Shape Inspection Structural Plate Arch

Arches in fair to good condition will have the following characteristics: a good shape appearance with smooth and symmetrical curvature, and a rise within three to four percent of theoretical. Marginal condition would be indicated when the arch is significantly non-symmetrical, when arch height is five to seven percent less or greater than theoretical, or when side or top plate flattening has occurred such that the plate radius is 50 to 100 percent greater than theoretical. Arches in poor to critical condition will have a poor shape appearance including significant distortion and deflection, extremely non-symmetrical shape, severe flattening (radius more than 100 percent greater than theoretical) of sides or top plates, or a rise more than eight percent greater or less than the theoretical rise. Guidelines for rating structural plate arches are shown in exhibit 90.

RATING GUIDELINES FOR STRUCTURAL PLATE ARCH BARREL

RATING	CONDITION	RATING	CONDITION
9	• New condition	4	• Shape: marginal, significant distortion and deflection throughout; sides flattened with radius 100 percent greater than design • Rise: within 7 to 8 percent of design • Seams: major cracking of seam near crown; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting • Footings: rotated due to erosion and undercutting; settlement has caused damage to metal arch
8	• Shape: good, smooth symmetrical curvature • Rise: within ± 3 percent of design • Seams: properly made and tight • Metal: minor defects and damage due to contraction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion		
7	• Shape: generally good with smooth curvature, symmetrical; slight flattening of top or sides in one section • Rise: within 3 to 4 percent of design • Seams: minor cracking at a few bolt holes; minor seam opening, possibility of soil infiltration • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: moderate erosion causing differential settlement and minor cracking in footing	3	• Shape: poor, extreme distortion and deflection in one section; sides virtually flattened; extremely non-symetrical • Rise: within 8 to 10 percent of design • Seams: cracked 3" to either side of bolts • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut; major cracking and spalling
6	• Shape: fair, smooth curvature but non-symetrical; slight flattening of top and sides throughout • Rise: within 4 to 5 percent of design • Seams: minor cracking of bolt holes along one or more seams; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footings: moderate cracking and differential settlement of footing due to extensive erosion	2	• Shape: critical, extreme deflection, throughout; sides flattened; extremely non-symetrical • Rise: greater than 10 percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
5	• Shape: generally fair, significant distortion and deflection in one section; sides beginning to flattend; non-symetrical • Rise: within 5 to 7 percent of design • Seams: moderate cracking of one seam near footing; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking in footing	1	• Shape: severe due to partial collapse; local reverse curve of crown and sides • Seams: failed, backfill pushing in • Road: closed to traffic
		0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.29 (Exhibit 90) Condition Rating Guidelines

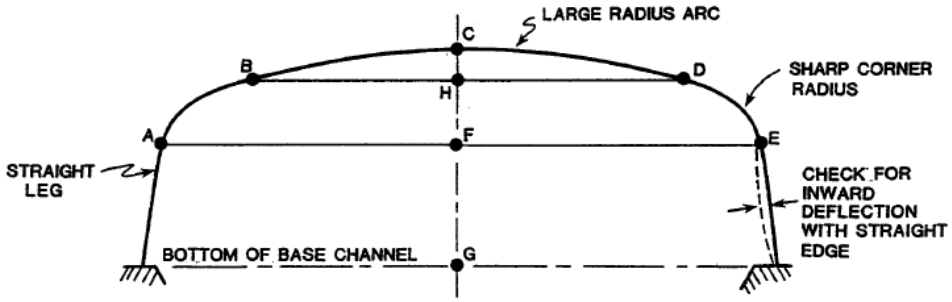
5-5.5 Corrugated Metal Box Culverts.

The box culvert is not like the other flexible buried metal structures. It behaves as a combination of ring compression action and conventional structure action. The sides are straight, not curved and the plates are heavily reinforced and have moment or bending strength that is quite significant in relation to the loads carried.

The key shape factor in a box culvert is the top arc. The design geometry is clearly very "flat" to begin with and therefore cannot be allowed to deflect much. The span at the top is also important and cannot be allowed to increase much.

The side plates often deflect slightly inward or outward. Generally an inward deflection would be the more critical as an outward movement would be restrained by soil.

Shape factors to be checked visually include flattening of top arc, outward movement of sides, or inward deflection of the sides. The inspector should note the visual appearance of the shape and should measure and record the rise and the horizontal span at the top of the straight legs as shown in exhibit 91. If the rise is more or less than 1 ½ percent of the design rise, the curvature of the large top radius should be checked.



1. MINIMUM REQUIRED MEASUREMENTS
   - RISE = CG
   - SPAN = AE

2. IF NOT POSSIBLE TO MEASURE CG, MEASURE BD AND CH

3. IF CG DIFFERS BY MORE THAN 1½% OF DESIGN OR AE DIFFERS BY MORE THAN ±3% OF DESIGN MEASURE
   - CHORD OF TOP ARC = BD
   - MIDDLE ORDINATE OF TOP ARC = CH

Figure 12.4.30 (Exhibit 91) Shape Inspection Structural Plate Box Culverts

The radius points are not necessarily located at the longitudinal seams. Many box culverts use double radius plates and the points where the radius changes must be estimated by the inspector or can be determined from the manufacturer's literature. These points can still be referenced to the bolt pattern to describe exactly where they are. Since these are all low structures, the spots should also be marked and

painted for convenient repeat inspection.

Box culverts in fair to good condition will appear to be symmetrical with smooth curves, slight or no deflection of the straight legs, a horizontal span length within five percent of the design span and the middle ordinate of the tops are within ten percent of the design. Culverts in marginal condition may appear to be non-symmetrical, have noticeable deflection in the straight legs, have spans that differ from design by five percent, or have a middle ordinate of the top arc that differ from design by 20 to 30 percent. Poor to critical conditions exist when the culvert shape appears poor, the culvert has severe deflections of the straight legs, a horizontal span that differs from design by more than five percent, or a middle ordinate of the top arc that differs from the theoretical by more than 40 to 50 percent. Guidelines for rating structural plate box culverts are shown in exhibit 92.

RATING GUIDELINES FOR CORRUGATED METAL BOX CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature – Top Arc Mid-Ordinate: within 11 percent of design – Horizontal Span: within 5 percent of design • Sides: straight leg very slightly deflected inward or outward and curvature smooth • Seams: properly made and tight • Metal: minor defects and damage due to construction – Aluminum: superficial corrosion, slight pitting – Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical – Top Arc Mid-Ordinate: within 11 percent to 15 percent of design – Horizontal Span: within 5 percent of design • Sides: straight leg slightly deflected inward or moderately deflected outward, curvature smooth • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: – Aluminum: moderate corrosion, no attack of core alloy – Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical – Top Arc Mid-Ordinate: within 15 percent of design – Horizontal Span: more than + or – 5 percent of design • Sides: straight leg moderately deflected inward or extremely deflected outward, curvature smooth • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: – Aluminum: significant corrosion, minor attack of core alloy – Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design – Top Arc Mid-Ordinate: within 15 to 20 percent of design – Horizontal Span: more than + or – 5 percent of design – Sides: straight leg bowed inward significantly or extremely bowed outward for distance of less than 1/4 span length • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: – Aluminum: significant corrosion, moderate attack of core alloy – Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design – Top Arc Mid-Ordinate: within 20 to 30 percent of design – Horizontal Span: more than + or – 5 percent of design – Sides: straight leg bowed inward significantly or extremely bowed outward for distance between 1/4 and 1/2 span length, curvature irregular • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: – Aluminum: extensive corrosion, significant attack of core alloy – Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design – Top Arc Mid-Ordinate: 30 to 40 percent less than design – Horizontal Span: more than + or – 6 percent of design – Sides: straight leg extremely bowed inward for distance less than 1/2 span length or leg bowed outward severely causing bulges in metal • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: – Aluminum: extensive corrosion, attack of core alloy, scattered perforations – Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design – Top Arc Mid-Ordinate: more than 40 percent less than design – Horizontal Span: more then + or – 8 percent of design – Sides: straight leg extremely bowed inward for a distance of 1/2 to 1 span length, or leg bowed outward severely causing bulges or kinking in metal • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: – Aluminum: extensive perforations due to corrosion – Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.31 (Exhibit 92) Condition Rating Guidelines

The following excerpts are from a reproduction of the out-of-print Culvert Inspection Manual (Supplement to Manual 70), July 1986 – Chapter 5, Section 6.

Section 6. CORRUGATED METAL LONG-SPAN CULVERTS

5-6.0 General.

This section describes procedures for conducting shape inspections of long-span structures. The long-span structures addressed include four typical shapes: low profile arch, horizontal ellipse, high profile arch, and pear. These shapes are illustrated in exhibit 93. The evaluation of shape characteristics of long-spans will vary somewhat depending upon the typical shape being inspected. However, the top or crown sections of all long-span structures have very similar geometry. The crown sections on all long-span structures can be inspected using the same criteria. This section therefore includes separate discussions on the crown section and on each of the typical long-span shapes. Guidelines are also provided for rating the condition of each shape in terms of shape characteristics and barrel defects. The procedures for using the rating guidelines are the same as those described in section 5-5.1.

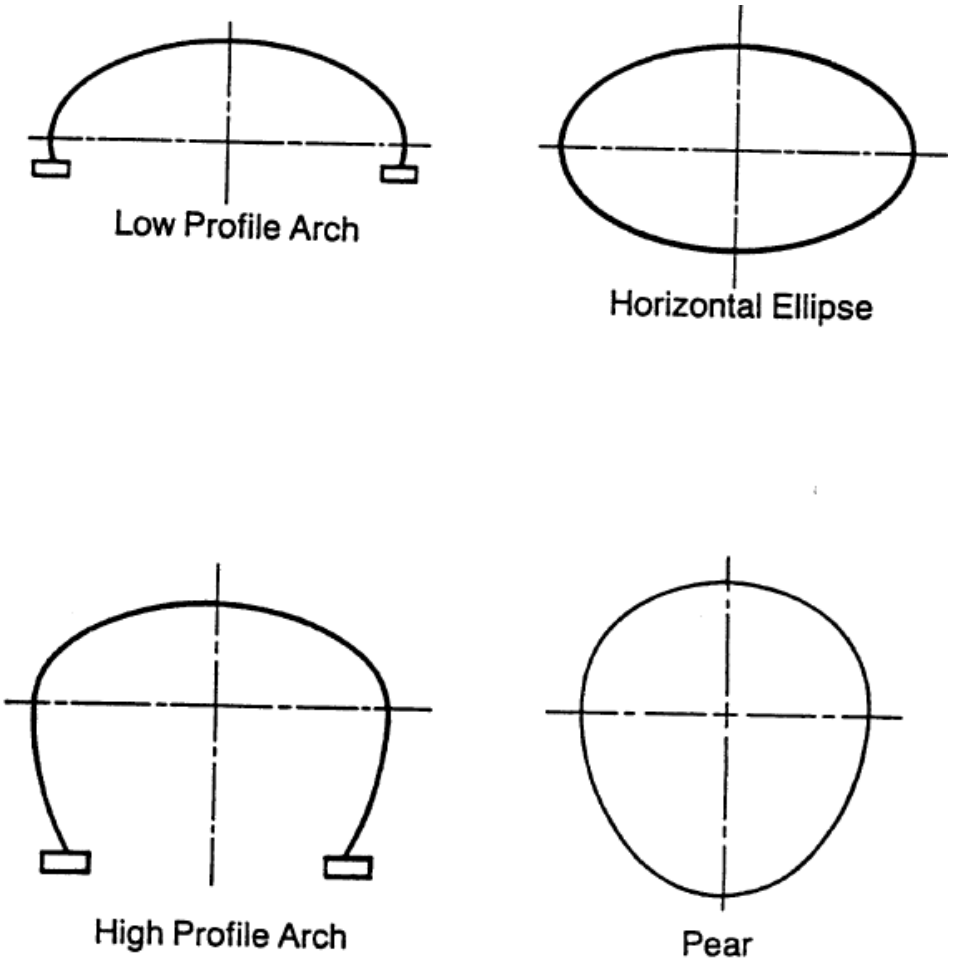
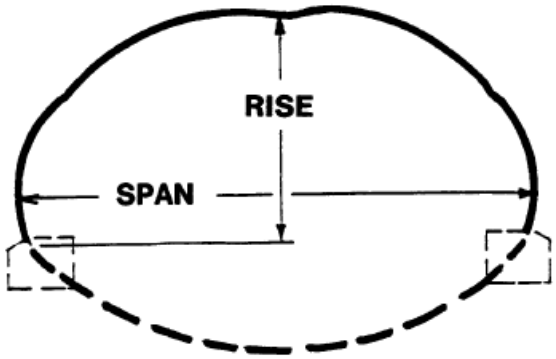


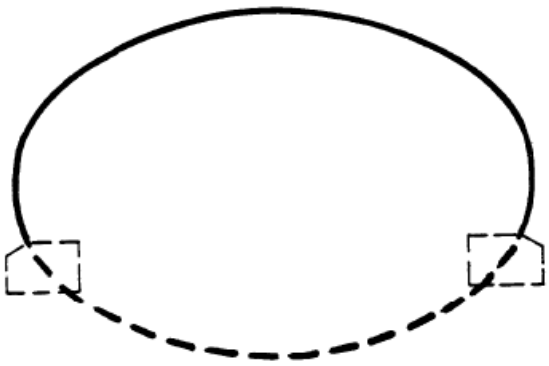
Figure 12.4.32 (Exhibit 93) Typical Long-Span Shapes

Shape inspections of long-span structures will generally consist of 1) visual observations of shape characteristics such as smooth or distorted curvature and symmetrical or non-symmetrical shape, 2) measurements of key dimensions, and 3) elevations of key points. Additional measurements may be necessary if measurements or observed shape differ significantly from design.

The visual observations are extremely important to evaluate the shape of the total cross section. Simple measurements such as rise and span do not describe curvature, yet adequate curvature is essential, as shown in exhibit 94. However, measurements and elevations are also needed to document the current shape so that the rate change, if any, can be monitored.



THE RISE AND SPAN ON THIS STRUCTURE ARE EQUAL TO THE DESIGN RISE AND SPAN, YET IT IS A VERY POOR SHAPE



THE RISE AND SPAN ON THIS STRUCTURE ARE CONSIDERABLY DIFFERENT FROM THE DESIGN RISE AND SPAN, YET IT IS IN EXCELLENT SHAPE

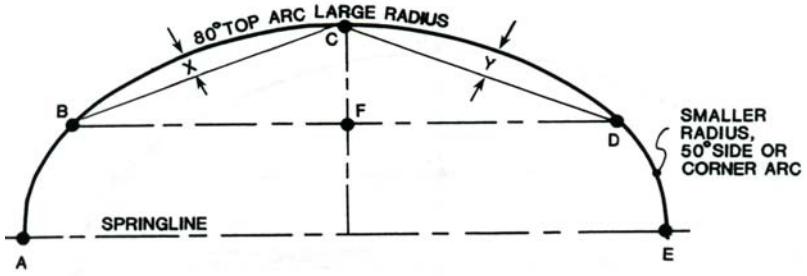
Figure 12.4.33 (Exhibit 94) Erosion Damage to Concrete Invert

Many long-spans will be too large to allow simple direct measuring. Vertical heights may be as large as 6.1 to 9.1 m (20 to 30 feet) and horizontal spans may be large and as high as 3.7 to 4.6 m (12 to 15 feet) above inverts. Culverts may have flowing water obscuring the invert and any reference points there. It is, therefore, in general desirable to have instrument survey points, which can be quickly checked for elevation. When direct measuring is practical a 7.6 m (25 foot) telescoping extension rod can be used for measuring. Such rods can also serve as level rods for taking elevations.

5-6.1 Long-Span Crown Section - Shape Inspection.

As previously mentioned, the section above the springline is essentially the same for most long-span shapes. With the exception of pear shapes, the standard top geometry uses a large radius top arc of approximately 80 degrees with a radius of 4.6 to 7.6 m (15 to 25 feet). The adjacent corner or side plates are from one-half to one-fifth the top arc radius. The most important part of a long-span shape is the standard top arch geometry. Adequate curvature of the large radius top arc is critical. Inspection of the crown section should consist of a visual inspection of the general shape for smooth curvature (no distortion, flattening, peaks, or cusps) and symmetrical shape (no racking).

An inspection should also include key measurements such as the middle ordinate of the top arc. Recommended measurements and elevations are shown in exhibit 95.



1. MINIMUM REQUIRED ELEVATIONS – B, C, D

   MINIMUM REQUIRED MEASUREMENTS –

   - TOP SPAN = AE

   CALCULATE CF = ELEV C – $\frac{\text{ELEV B + ELEV D}}{2}$

2. IF CF IS GREATER THAN OR LESS THAN DESIGN BY 10% MEASURE:

   - TOP ARC CHORD = BD

3. IF BD DIFFERS BY MORE THAN 3% FROM DESIGN MEASURE FOR EACH HALF OF TOP ARC

   - HALF TOP ARC MID ORDINATES = X & Y

Note: These measurements and elevations should be obtained on all long span inspections (see exhibits 96, 98, 100 and 103).

Figure 12.4.34 (Exhibit 95) Shape Inspection Crown Section of Long Span Structures

The initial inspection should establish elevations for the radius points and the top of the crown. From these elevations the middle ordinate for the top arc can be calculated. If the actual middle ordinate is 10 percent more or less than the theoretical design mid-ordinate the horizontal span for the top arc should also be measured. For standard 80 degree arcs the theoretical middle ordinate is equal to 0.234 times the theoretical radius of the top arc. This span is not easy to measure on many long-span structures and need not be measured if the top arc mid-ordinate is within 10 percent of theoretical. Even if it is convenient and practical to direct measure the vertical heights of the points on the top arc from the bottom of the structure, it is wise to also establish their elevations from a reliable benchmark. Bottom reference points can be wiped out by erosion, covered with debris, or covered by water. When direct vertical measuring is practical, the shape may be checked on subsequent inspections with direct measurement. However, it is still important to establish elevations in case bottom reference points are lost or inaccessible.

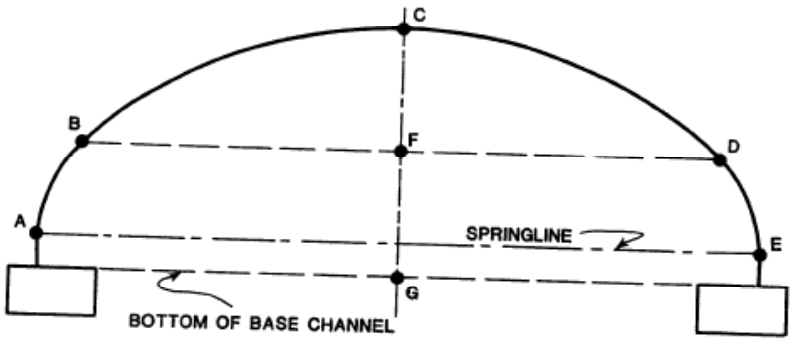
Crown sections in good condition will have a shape appearance that is good, with smooth and symmetrical curvature. The actual middle ordinate should be within 10 percent of the theoretical, and the horizontal span (if measured) should be within five percent of theoretical. Crown sections in fair condition will have a fair to good shape appearance, smooth curvature but possibly slightly non-symmetrical. Middle ordinates of the top arc may be within 11 to 15 percent of theoretical and the horizontal span may differ by more than 5 percent of theoretical.

Crown sections in marginal condition will have measurements similar to those described for fair shape. However, the shape appearance will be only fair to marginal with noticeable distortion, deflection, or non-symmetrical curvature. When the curvature is noticeably distorted or non-symmetrical, the sides should be checked for flattening by measuring the middle ordinates of the halves of the top arc. Crown sections with marginal shape may have middle ordinates for top half arcs that are 30 to 50 percent less than theoretical.

Crown sections in poor to critical condition will have a poor to critical shape appearance with severe distortion or deflection. The middle ordinate of the top arc may be as much as 20 percent less than theoretical, while middle ordinates of the top arc halves may be 50 to 70 percent less than theoretical.

5-6.2 Low Profile Long-Span Arch - Shape Inspection.

The low profile arch is essentially the same as the crown section except that the sides are carried about 10 degrees below the springline to the footing. These structures are low and can be measured more easily than other long-span shapes. Recommended measurements and elevations are shown in exhibit 96. Rating guidelines are listed in exhibit 97.



AE = SPAN, CG = RISE OR HEIGHT

1. MINIMUM REQUIRED MEASUREMENTS –
   - SPAN = AE
   - TOP ARC CHORD = BD
   - RISE = CG

2. MINIMUM REQUIRED ELEVATIONS B, C, D
3. CALCULATE CF FROM ELEVATIONS

$$CF = \text{ELEV. C} - \frac{\text{ELEV. B} + \text{ELEV. D}}{2}$$

Note: Use with exhibit 95, crown inspection.

Figure 12.4.35 (Exhibit 90) Shape Inspection Low Profile Long Span Arch

RATING GUIDELINES FOR LOW PROFILE ARCH LONG-SPAN CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosin, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core allow - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy, - Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 10 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.36 (Exhibit 97) Condition Rating Guidelines

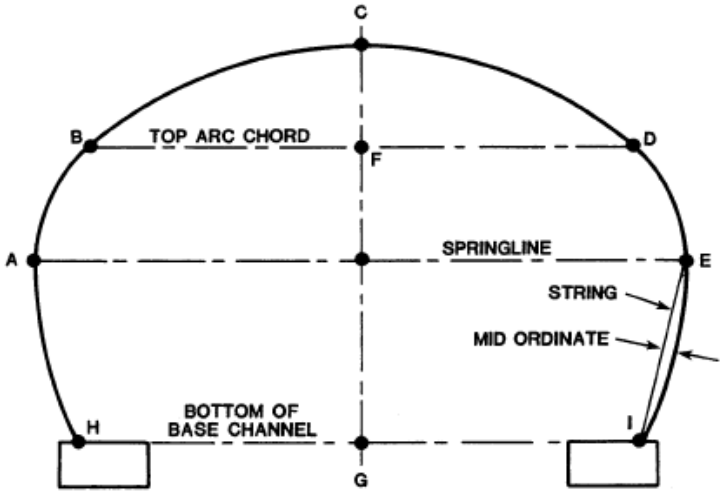
Because arches are fixed on concrete footings, backfill pressures will try to flatten the sides and peak the top. Another important shape factor is symmetry. If the base channels are not square to the centerline of the structure racking may occur during erection. In racked structures, the crown moves laterally and the curvature in one side becomes flatter while the curvature in the other side increases. Backfill pressures may cause this condition to worsen.

5-6.3 High Profile Long-Span Arch – Shape Inspection.

High profile arches have a standard crown section geometry but have high large radius side walls below the springline. Curvature in these side plates is important. In shallow fills or minimum covers, the lateral soil pressures may approach or exceed the loads over the culvert. Excessive lateral forces could cause the sidewall to flatten or buckle inward.

Inspectors should visually inspect high profile arches for flattening of the side plates. Additionally, high profile arches have the same tendencies as regular arches for peaking and racking, so inspectors must also look for peaked top arcs and non-symmetrical or racked arches.

Recommended measurements and elevations are shown in exhibit 98. The shape of the crown section is the most important shape factor. It can be measured and evaluated using the same criteria as that described for the standard crown section. If flattening is observed in the high sidewall the curvature of the sides should be checked by measuring the middle ordinate of the side walls. If the sidewall middle ordinate is no more than 50 to 70 percent less than the theoretical middle ordinate and no other shape problems are found the arch's shape may be considered fair. When the middle ordinate approaches 75 to 80 percent less than theoretical, the shape should be considered marginal. If the middle ordinate is more than 80 to 90 percent less than theoretical the shape should be considered poor to critical. Rating guidelines are provided in exhibit 99.



AE = SPAN, CG = RISE

1. MINIMUM REQUIRED MEASUREMENTS
   - SPAN = AE

2. MINIMUM REQUIRED ELEVATIONS – B, C, D, H, I

3. CALCULATE CF FROM ELEVATIONS

Note: Use with exhibit 95, crown inspection.

Figure 12.4.37 (Exhibit 98) Shape Inspection High Profile Long-Span Arch

RATING GUIDELINES FOR HIGH PROFILE ARCH LONG-SPAN CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design - Side Plates: smooth curvature • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footing: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design - Side Plates: side flattened, mid-ordinate less than 50 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footing: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 35 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 25 percent of design • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footing: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 20 percent of design • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting • Footing: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design - Side Plates: side flattened, mid-ordinate less than 12 percent of design • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design - Side Plates: side flattened, mid-ordinate less than 10percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved - Side Plates: side flat or reversed curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

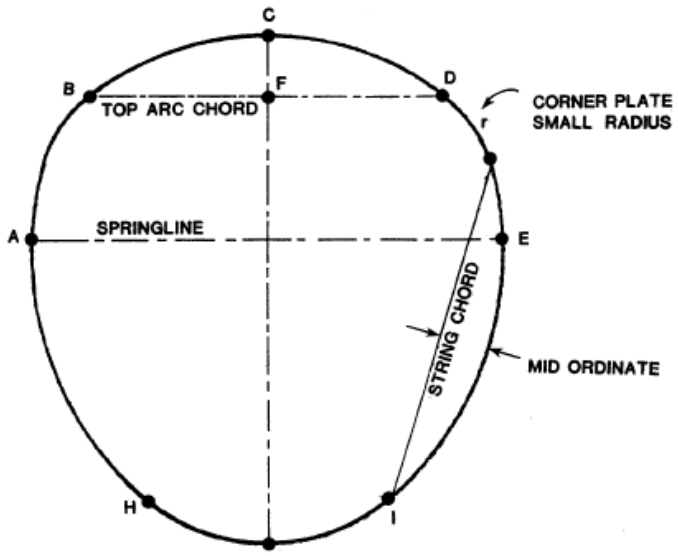
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.38 (Exhibit 99) Condition Rating Guidelines

5-6.4 Pear Shape Long-Span – Shape Inspection.

The crown section of the pear shape differs from the standard top arch in that smaller radius corner arcs stop short of the horizontal springline. The large radius sides extend above the plane of the horizontal span. In checking curvature of the sides, the entire arc should be checked. Side flattening, particularly in shallow fills, is the most critical shape factor.

The pear shape behaves similarly to the high profile arch. It is essentially a high profile with a metal bottom instead of concrete footings. Pears may be inspected using the criteria for a high profile arch. The recommended measurements and elevations are shown in exhibit 100. Rating guidelines are provided in exhibit 101.



AE = SPAN, CG = RISE

1. MINIMUM REQUIRED MEASUREMENT - AE
   ▪ SPAN = AE

2. MINIMUM REQUIRED ELEVATIONS B, C, D

3. WHEN FLATTENING OBSERVED IN SIDE, CHECK
   MID ORDINATE (RECORD CHORD LENGTH USED)

Note: Use with exhibit 95, crown inspection.

Figure 12.4.39 (Exhibit 100) Shape Inspection Long Span Pear-Shape

RATING GUIDELINES FOR PEAR SHAPED LONG-SPAN CULVERT BARREL

RATING	CONDITION	RATING	CONDITION
9	• New condition	4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 20 percent of design • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of alloy - Steel: extensive heavy rust, deep pitting
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design - Side Plates: smooth curvature • Seams: properly made and tight • Metal: minor defects and damage due to construction; superficial corrosion with no pitting - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting	3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design - Side Plates: side flattened, mid-ordinate less than 12 percent of design • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design - Side Plates: side flattened, mid-ordinate less than 50 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting	2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design - Side Plates: side flattened, mid-ordinate less than10percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 35 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting	1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved - Side Plates: side flat or reversed curved - Seams: failed, backfill pushing in • Road closed to traffic
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 25 percent of design • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: corroded locally - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting	0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.40 (Exhibit 101) Condition Rating Guidelines

5-6.5 Horizontal Ellipse – Shape Inspections.

For horizontal ellipses the most important shape factor is adequate curvature in the crown section. The crown section uses the standard long-span crown geometry. The sides and bottom behave similar to the corners and bottom of pipe arches. The invert has relatively minor pressure when compared with the sides, which may have several times the bearing pressure of the invert. As a result the corners and sides have the tendency to push down into the soil while the bottom does not move. The effect is as if the bottom pushed up. Inspectors should look for indications of bottom flattening and differential settlement between the side and bottom sections, as illustrated in exhibit 102.

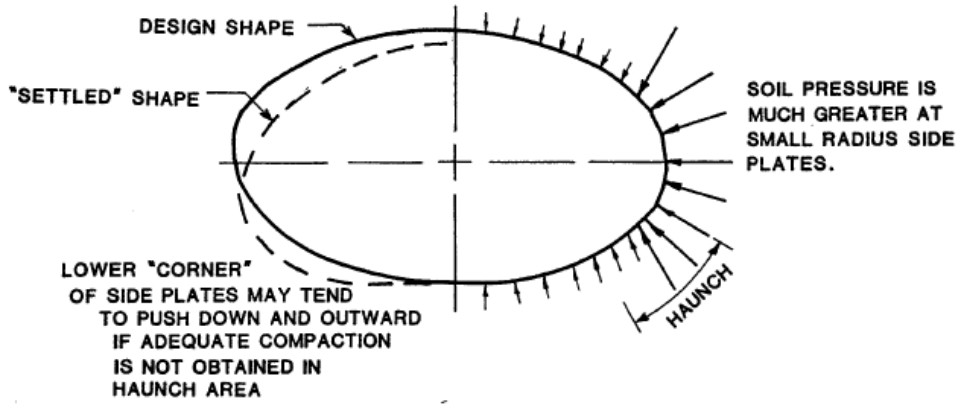
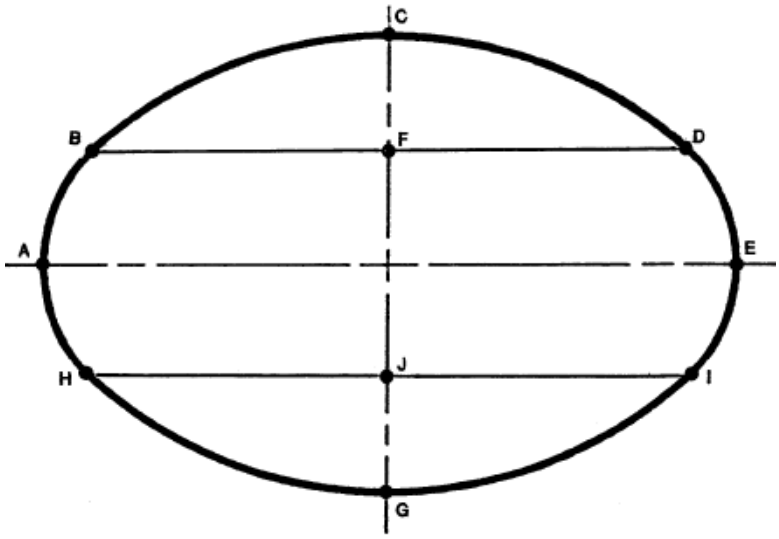


Figure 12.4.41 (Exhibit 102) Potential for Differential Settlement in Horizontal Ellipse

The recommended measurements and evaluations for a shape inspection of horizontal ellipse are shown in exhibit 103. The measurements are essentially the same as those recommended for a standard crown section. Shape evaluation of an ellipse is also essentially the same as the evaluation of a standard crown section except that the curvature of the bottom should also be evaluated. Marginal shape would be indicated when the bottom is flat in the center and corners are beginning to deflect downward or outward. Critical shape conditions would be indicated by reverse curvature in the bottom arc. Guidelines for rating horizontal ellipse shape culverts are provided in exhibit 104.



1. MINIMUM REQUIRED MEASUREMENTS
   - SPAN = AE

2. MINIMUM REQUIRED ELEVATIONS – B, C, D, G (IF POSSIBLE)

3. WHEN BOTTOM FLATTENING IS OBSERVED, CHECK CURVATURE, MEASURE
   - BOTTOM ARC CHORD = HI
   - BOTTOM ARC MIDDLE ORDINATE = JG

Note: Use with exhibit 95, crown inspection.

Figure 12.4.42 (Exhibit 103) Shape Inspection Long-Span Horizontal Ellipse

RATING GUIDELINES FOR HORIZONTAL ELLIPSE LONG SPAN CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design - Bottom Arc: smooth curvature, mid-ordinate within 50 percent of design • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design - Bottom Arc: bottom flattened, mid-ordinate less than 50 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design - Bottom Arc: bottom flattened and irregular, mid-ordinate less than 50 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Bottom Arc: bottom virtually flat over center half of arc • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing

RATING	CONDITION
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Bottom Arc: bottom virtually flat over center half of arc and deflected down at corners • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of alloy - Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design - Bottom Arc: bottom reverse curved in center • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design - Bottom Arc: bottom reversed curved in center and bulged out at sides • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footings: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 12.4.43 (Exhibit 104) Condition Rating Guidelines

12.4.7 Evaluation

State and federal rating guidelines systems have been developed in order to aid in the inspection of flexible culverts. The two major rating guidelines systems currently in use include the National Bridge Inspection Standards (NBIS) rating and the Element level inspection Bridge Management System (BMS).

Application of the NBIS Rating Guidelines

Using NBIS rating guidelines, a 1-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the culvert (Item 62). This item evaluates the alignment, settlement, joints, structural condition, scour, and other items associated with culverts. Rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 for general descriptive codes and Topics 12.4.6 through 12.4.8 for specific codes for the various flexible culverts. The rating code is intended to be an overall evaluation of the culvert. Integral wingwalls to the first construction or expansion joint shall be included in the evaluation. It is also important to note that Items 58-Deck, 59-Superstructure, and 60-Substructure shall be coded "N" for all culverts.

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

Application of Condition State Assessment (Element Level Inspection)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the culvert. The information from the narrative and condition state summaries are then used to complete the element level condition report showing quantities at the correct rating value. There are no Element Level Smart Flags specific to culverts.

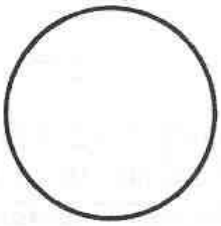
In an element level condition state assessment of a flexible culvert, the AASHTO CoRe element is:

Element No.	Description
240	Unpainted Steel Culvert
243	Culvert: Other

"Culvert: Other" includes masonry, aluminum and combination of other materials. The unit quantity for the culvert is meters or feet and the total length along the barrel must be placed in one of the four available condition states. Condition State 1 is the best possible rating. See Topic 4.6 for condition state descriptions.

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

ROUND



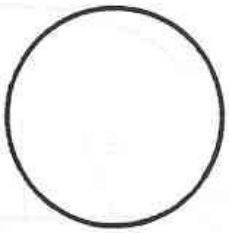
Handling Weight of Corrugated Steel Pipe (2⅔ × ½ in.)
Estimated Average Weights—Not for Specification Use*

Inside Diameter, in.	Specified Thickness, in.	Approximate Pounds per Lineal Ft**			
		Galvanized	Full-Coated	Full-Coated and Invert Paved	Full-Coated and Full Paved
12	0.052	8	10	13	
	0.064	10	12	15	
	0.079	12	14	17	
15	0.052	10	12	15	
	0.064	12	15	18	
	0.079	15	18	21	
18	0.052	12	14	17	
	0.064	15	19	22	
	0.079	18	22	25	
21	0.052	14	16	19	
	0.064	17	21	26	
	0.079	21	25	30	
24	0.052	15	17	20	
	0.064	19	24	30	45
	0.079	24	29	35	50
30	0.052	20	22	25	
	0.064	24	30	36	55
	0.079	30	36	42	60
36	0.052	24	26	29	
	0.064	29	36	44	65
	0.079	36	43	51	75
42	0.052	28	30	33	
	0.064	34	42	51	
	0.079	42	50	59	85
48	0.052	31	33	36	
	0.064	38	48	57	
	0.079	48	58	67	95
54	0.064	44	55	66	95
	0.079	54	65	76	105
60	0.079	60	71	85	
	0.109	81	92	106	140
66	0.109	89	101	117	160
	0.138	113	125	141	180
72	0.109	98	112	129	170
	0.138	123	137	154	210
78	0.109	105	121	138	200
	0.138	133	149	166	230
84	0.109	113	133	155	225
	0.138	144	161	179	240
90	0.109	121	145	167	
	0.138	154	172	192	
	0.168	186	204	224	
96	0.138	164	191	217	
	0.168	198	217	239	

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute)

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

ROUND

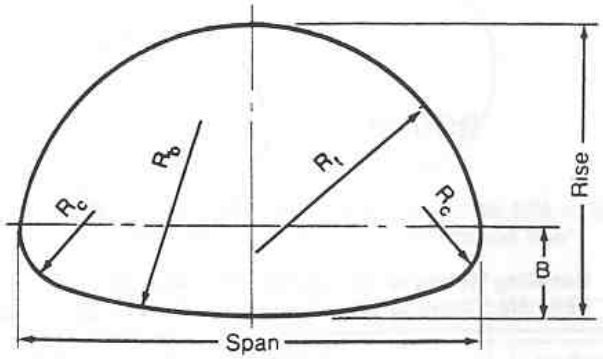


Handling Weight of Corrugated Steel Pipe (3 × 1 in. or 5 × 1 in.)***
Estimated Average Weights—Not for Specification Use

Inside Diameter, in.	Specified Thickness, in.	Approximate Pounds per Lineal Ft**			
		Galvanized	Full-Coated	Full-Coated and Invert Paved	Full-Coated and Full Paved
54	0.064	50	66	84	138
	0.079	61	77	95	149
60	0.064	55	73	93	153
	0.079	67	86	105	165
66	0.064	60	80	102	168
	0.079	74	94	116	181
72	0.064	66	88	111	183
	0.079	81	102	126	197
78	0.064	71	95	121	198
	0.079	87	111	137	214
84	0.064	77	102	130	213
	0.079	94	119	147	230
90	0.064	82	109	140	228
	0.079	100	127	158	246
96	0.064	87	116	149	242
	0.079	107	136	169	262
102	0.064	93	124	158	258
	0.079	114	145	179	279
108	0.064	98	131	166	273
	0.079	120	153	188	295
114	0.064	104	139	176	289
	0.079	127	162	199	312
120	0.064	109	146	183	296
	0.079	134	171	210	329
	0.109	183	220	259	378
126	0.079	141	179	220	346
	0.109	195	233	274	400
132	0.079	148	188	231	363
	0.109	204	244	287	419
138	0.079	154	196	241	379
	0.109	213	255	300	438
144	0.109	223	267	314	458
	0.138	282	326	373	517

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Sizes and Layout Details—CSP Pipe Arches
2⅔ × ½ in. Corrugation

Equiv. Diameter, in.	Span, in.	Rise, in.	Waterway Area, ft²	Layout Dimensions			
				B in.	R_c in.	R_t in.	R_b in.
15	17	13	1.1	4⅛	3½	8⅝	25⅝
18	21	15	1.6	4⅞	4⅛	10¾	33⅛
21	24	18	2.2	5⅝	4⅞	11⅞	34⅝
24	28	20	2.9	6½	5½	14	42¼
30	35	24	4.5	8⅛	6⅞	17⅞	55⅛
36	42	29	6.5	9¾	8¼	21½	66⅛
42	49	33	8.9	11⅜	9⅝	25⅛	77¼
48	57	38	11.6	13	11	28⅝	88¼
54	64	43	14.7	14⅝	12⅜	32¼	99¼
60	71	47	18.1	16¼	13¾	35¾	110¼
66	77	52	21.9	17⅞	15⅛	39⅜	121¼
72	83	57	26.0	19½	16½	43	132¼

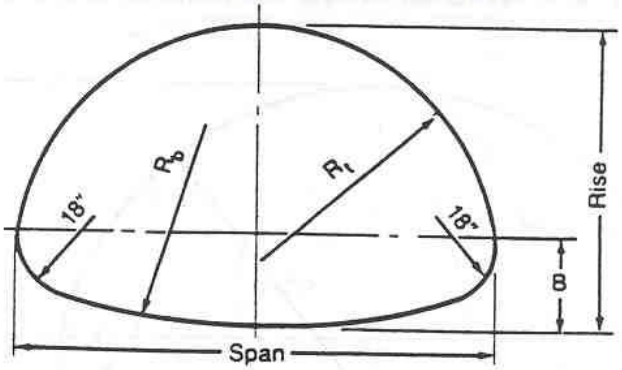
Dimensions shown not for specification purposes, subject to manufacturing tolerances.

Sizes and Layout Details—CSP Pipe-Arches
3 × 1 in. Corrugation

Equiv. Diameter, in.	Size, in.	Span, in.	Rise, in.	Waterway Area, ft²	Layout Dimensions			
					B in.	R_c in.	R_t in.	R_b in.
54	60 × 46	58½	48½	15.6	20½	18¾	29⅜	51⅛
60	66 × 51	65	54	19.3	22¾	20¾	32⅝	56¼
66	73 × 55	72½	58¼	23.2	25⅛	22⅞	36¾	63¾
72	81 × 59	79	62½	27.4	23¾	20⅞	39½	82⅝
78	87 × 63	86½	67¼	32.1	25¾	22⅝	43⅜	92¼
84	95 × 67	93½	71¾	37.0	27¾	24⅜	47	100¼
90	103 × 71	101½	76	42.4	29¾	26⅛	51¼	111⅝
96	112 × 75	108½	80½	48.0	31⅝	27¾	54⅞	120¼
102	117 × 79	116½	84¾	54.2	33⅝	29½	59⅜	131¾
108	128 × 83	123½	89¼	60.5	35⅝	31¼	63¼	139¾
114	137 × 87	131	93¾	67.4	37⅝	33	67⅜	149½
120	142 × 91	138½	98	74.5	39½	34¾	71⅝	162⅜

Figure 12.4.44 Standard Sizes for Corrugated Steel (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

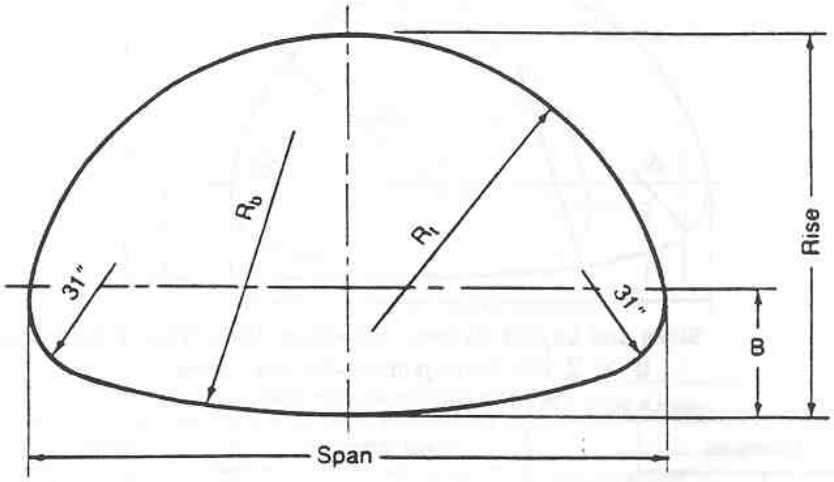


Sizes and Layout Details—Structural Plate Steel Pipe-Arches
6 × 2 in. Corrugations—Bolted Seams
18-inch Corner Radius *Rc*

Dimensions		Waterway Area, ft²	Layout Dimensions			Periphery		
Span, ft-in.	Rise, ft-in.		*B* in.	R_t ft	R_b ft	No. of Plates	Total *N*	Total *Pi*
6-1	4-7	22	21.0	3.07	6.36	5	22	66
6-4	4-9	24	20.5	3.18	8.22	5	23	69
6-9	4-11	26	22.0	3.42	6.96	5	24	72
7-0	5-1	28	21.4	3.53	8.68	5	25	75
7-3	5-3	31	20.8	3.63	11.35	6	26	78
7-8	5-5	33	22.4	3.88	9.15	6	27	81
7-11	5-7	35	21.7	3.98	11.49	6	28	84
8-2	5-9	38	20.9	4.08	15.24	6	29	87
8-7	5-11	40	22.7	4.33	11.75	7	30	90
8-10	6-1	43	21.8	4.42	14.89	7	31	93
9-4	6-3	46	23.8	4.68	12.05	7	32	96
9-6	6-5	49	22.9	4.78	14.79	7	33	99
9-9	6-7	52	21.9	4.86	18.98	7	34	102
10-3	6-9	55	23.9	5.13	14.86	7	35	105
10-8	6-11	58	26.1	5.41	12.77	7	36	108
10-11	7-1	61	25.1	5.49	15.03	7	37	111
11-5	7-3	64	27.4	5.78	13.16	7	38	114
11-7	7-5	67	26.3	5.85	15.27	8	39	117
11-10	7-7	71	25.2	5.93	18.03	8	40	120
12-4	7-9	74	27.5	6.23	15.54	8	41	123
12-6	7-11	78	26.4	6.29	18.07	8	42	126
12-8	8-1	81	25.2	6.37	21.45	8	43	129
12-10	8-4	85	24.0	6.44	26.23	8	44	132
13-5	8-5	89	26.3	6.73	21.23	9	45	135
13-11	8-7	93	28.9	7.03	18.39	9	46	138
14-1	8-9	97	27.6	7.09	21.18	9	47	141
14-3	8-11	101	26.3	7.16	24.80	9	48	144
14-10	9-1	105	28.9	7.47	21.19	9	49	147
15-4	9-3	109	31.6	7.78	18.90	9	50	150
15-6	9-5	113	30.2	7.83	21.31	10	51	153
15-8	9-7	118	28.8	7.89	24.29	10	52	156
15-10	9-10	122	27.4	7.96	28.18	10	53	159
16-5	9-11	126	30.1	8.27	24.24	10	54	162
16-7	10-1	131	28.7	8.33	27.73	10	55	165

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



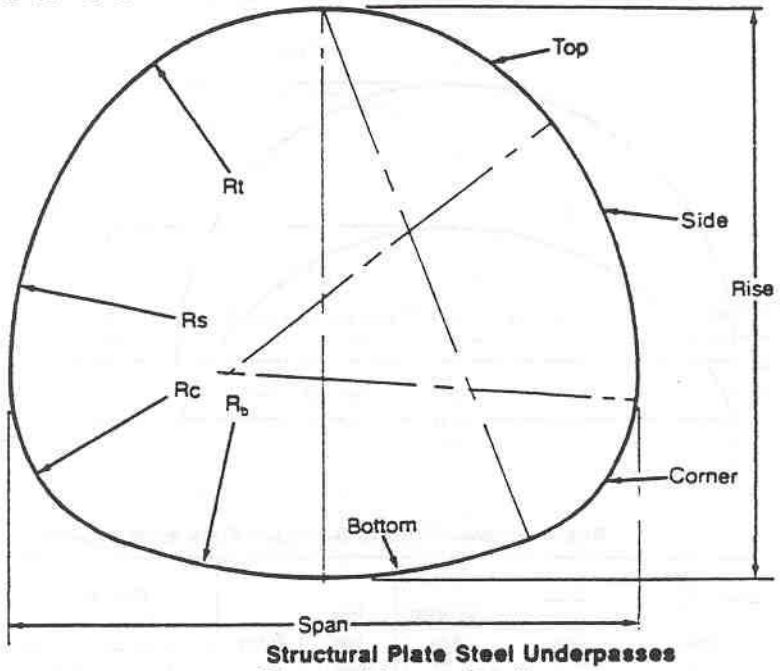
Sizes and Layout Details—Structural Plate Steel Pipe-Arches[11]
6 × 2 in. Corrugations—Bolted Seams
31-in Corner Radius, R_c

Dimensions		Waterway Area, ft²	Layout Dimensions			Periphery		
Span, ft-in.	Rise, ft-in.		B in.	R_t ft	R_b ft	No. of Plates	Total N	Total Pi
13-3	9-4	97	38.5	6.68	16.05	8	46	138
13-6	9-6	102	37.7	6.78	18.33	8	47	141
14-0	9-8	105	39.6	7.03	16.49	8	48	144
14-2	9-10	109	38.8	7.13	18.55	8	49	147
14-5	10-0	114	37.9	7.22	21.38	8	50	150
14-11	10-2	118	39.8	7.48	18.98	9	51	153
15-4	10-4	123	41.8	7.76	17.38	9	52	156
15-7	10-6	127	40.9	7.84	19.34	10	53	159
15-10	10-8	132	40.0	7.93	21.72	10	54	162
16-3	10-10	137	42.1	8.21	19.67	10	55	165
16-6	11-0	142	41.1	8.29	21.93	10	56	168
17-0	11-2	146	43.3	8.58	20.08	10	57	171
17-2	11-4	151	42.3	8.65	22.23	10	58	174
17-5	11-6	157	41.3	8.73	24.83	10	59	177
17-11	11-8	161	43.5	9.02	22.55	10	60	180
18-1	11-10	167	42.4	9.09	24.98	10	61	183
18-7	12-0	172	44.7	9.38	22.88	10	62	186
18-9	12-2	177	43.6	9.46	25.19	10	63	189
19-3	12-4	182	45.9	9.75	23.22	10	64	192
19-6	12-6	188	44.8	9.83	25.43	11	65	195
19-8	12-8	194	43.7	9.90	28.04	11	66	198
19-11	12-10	200	42.5	9.98	31.19	11	67	201
20-5	13-0	205	44.9	10.27	28.18	11	68	204
20-7	13-2	211	43.7	10.33	31.13	12	69	207

Dimensions are to inside crests and are subject to manufacturing tolerances.
$N = 3\ Pi = 9.6$ in.

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



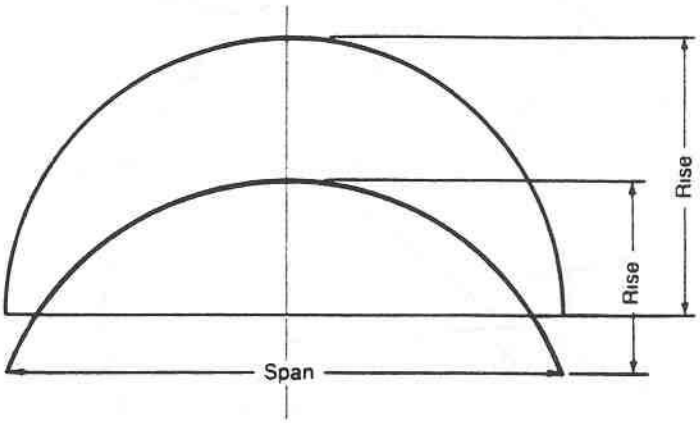
Structural Plate Steel Underpasses
Sizes and Layout Details

Span × Rise, ft and in.		Periphery			Layout Dimensions in In.			
		N	Pi	No. of Plates per Ring	R_t	R_s	R_c	R_b
5-8	5-9	24	72	6	27	53	18	Flat
5-8	6-6	26	78	6	29	75	18	Flat
5-9	7-4	28	84	6	28	95	18	Flat
5-10	7-8	29	87	7	30	112	18	Flat
5-10	8-2	30	90	6	28	116	18	Flat
12-2	11-0	47	141	8	68	93	38	136
12-11	11-2	49	147	9	74	92	38	148
13-2	11-10	51	153	11	73	102	38	161
13-10	12-2	53	159	11	77	106	38	168
14-1	12-10	55	165	11	77	115	38	183
14-6	13-5	57	171	11	78	131	38	174
14-10	14-0	59	177	11	79	136	38	193
15-6	14-4	61	183	12	83	139	38	201
15-8	15-0	63	189	12	82	151	38	212
16-4	15-5	65	195	12	86	156	38	217
16-5	16-0	67	201	12	88	159	38	271
16-9	16-3	68	204	12	89	168	38	246
17-3	17-0	70	210	12	90	174	47	214
18-4	16-11	72	216	12	99	157	47	248
19-1	17-2	74	222	13	105	156	47	262
19-6	17-7	76	228	13	107	158	47	295
20-4	17-9	78	234	13	114	155	47	316

All dimensions, to nearest whole number, are measured from inside crests.
Tolerances should be allowed for specification purposes. 6 × 2 in. Corrugations.

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Representative Sizes of Structural Plate Steel Arches

Dimensions(1)		Waterway Area, ft²	Rise over Span(2)	Radius, in.	Nominal Arc Length	
Span, ft	Rise, ft-in.				N(3)	Pi, in.
6.0	1-9½	7½	0.30	41	9	27
	2-3½	10	0.38	37½	10	30
	3-2	15	0.53	36	12	36
7.0	2-4	12	0.34	45	11	33
	2-10	15	0.40	43	12	36
	3-8	20	0.52	42	14	42
8.0	2-11	17	0.37	51	13	39
	3-4	20	0.42	48½	14	42
	4-2	26	0.52	48	16	48
9.0	2-11	18½	0.32	59	14	42
	3-10½	26½	0.43	55	16	48
	4-8½	33	0.52	54	18	54
10.0	3-5½	25	0.35	64	16	48
	4-5	34	0.44	60½	18	54
	5-3	41	0.52	60	20	60
11.0	3-6	27½	0.32	73	17	51
	4-5½	37	0.41	67½	19	57
	5-9	50	0.52	66	22	66
12.0	4-0½	35	0.34	77½	19	57
	5-0	45	0.42	73	21	63
	6-3	59	0.52	72	24	72
13.0	4-1	38	0.32	86½	20	60
	5-1	49	0.39	80½	22	66
	6-9	70	0.52	78	26	78
14.0	4-7½	47	0.33	91	22	66
	5-7	58	0.40	86	24	72
	7-3	80	0.52	84	28	84

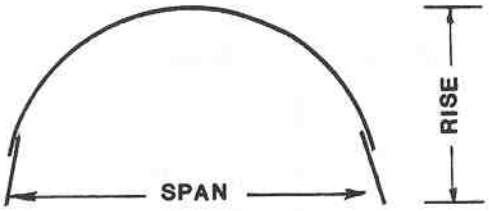
(Table continued on following page)

(1)Dimensions are to inside crests and are subject to manufacturing tolerances.
(2)R/S ratio varies from 0.30 to 0.52. Intermediate spans and rises are available.
(3)W = 3 Pi = 9.6 in. 6 × 2 in. Corrugations—Bolted Seams.

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

ARCH



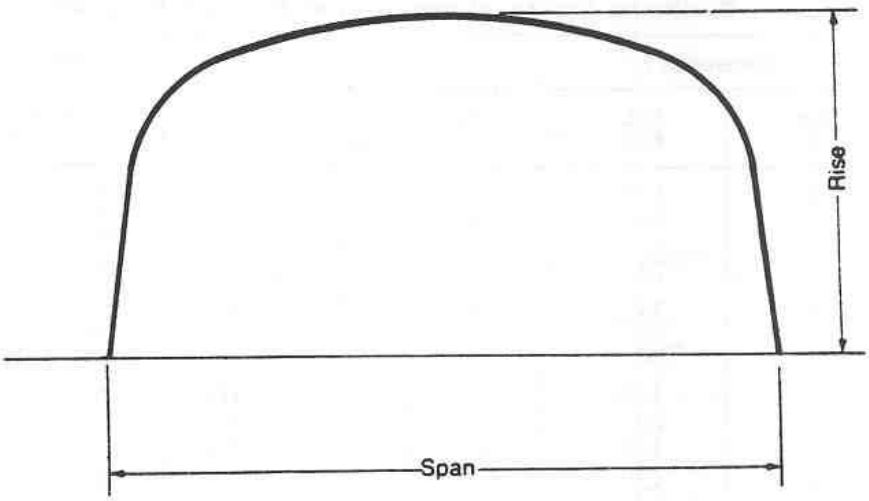
Continued. Representative Sizes of Structural Plate Steel Arches

Dimensions(1)		Waterway Area, ft²	Rise over Span(2)	Radius, in.	Nominal Arc Length	
Span, ft	Rise, ft-in.				N(3)	Pi, in.
15.0	4-7½	50	0.31	101	23	69
	5-8	62	0.38	93	25	75
	6-7	75	0.44	91	27	81
	7-9	92	0.52	90	30	90
16.0	5-2	60	0.32	105	25	75
	7-1	86	0.45	97	29	87
	8-3	105	0.52	96	32	96
17.0	5-2½	63	0.31	115	26	78
	7-2	92	0.42	103	30	90
	8-10	119	0.52	102	34	102
18.0	5-9	75	0.32	119	28	84
	7-8	104	0.43	109	32	96
	8-11	126	0.50	108	35	105
19.0	6-4	87	0.33	123	30	90
	8-2	118	0.43	115	34	102
	9-5½	140	0.50	114	37	111
20.0	6-4	91	0.32	133	31	93
	8-3½	124	0.42	122	35	105
	10-0	157	0.50	120	39	117
21.0	6-11	104	0.33	137	33	99
	8-10	140	0.42	128	37	111
	10-6	172	0.50	126	41	123
22.0	6-11	109	0.31	146	34	102
	8-11	146	0.40	135	38	114
	11-0	190	0.50	132	43	129
23.0	8-0	134	0.35	147	37	111
	9-10	171	0.43	140	41	123
	11-6	208	0.50	138	45	135
24.0	8-6	150	0.35	152	39	117
	10-4	188	0.43	146	43	129
	12-0	226	0.50	144	47	141
25.0	8-6½	155	0.34	160	40	120
	10-10½	207	0.43	152	45	135
	12-6	247	0.50	150	49	147

(1)Dimensions are to inside crests and are subject to manufacturing tolerances.
(2)R/S ratio varies from 0.30 to 0.52. Intermediate spans and rises are available.
(3)W = 3 Pi = 9.6 in. 6 × 2 in. Corrugations—Bolted Seams.

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Layout Details
Corrugated Steel Box Culverts

Rise, ft-in.	Span, ft-in.	Area ft²	Rise, ft-in.	Span, ft-in.	Area ft²
2-7	9-8	20.8	3-9	12-10	41.0
2-8	10-5	23.2	3-10	13-6	44.5
2-9	11-1	25.7	3-10	17-4	55.0
2-10	11-10	28.3	3-11	14-2	48.2
2-11	12-6	31.1	3-11	18-0	59.1
3-1	13-3	34.0	4-1	14-10	52.0
3-2	13-11	37.1	4-1	18-8	63.4
3-3	14-7	40.4	4-2	10-7	36.4
3-4	10-1	28.4	4-2	15-6	55.9
3-5	10-10	31.4	4-3	11-2	39.9
3-5	15-3	43.8	4-3	19-4	67.9
3-6	11-6	34.5	4-4	11-10	43.5
3-6	16-0	47.3	4-4	16-2	60.1
3-8	12-2	37.7	4-5	12-6	47.3
3-8	16-8	51.1	4-6	13-2	51.2

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

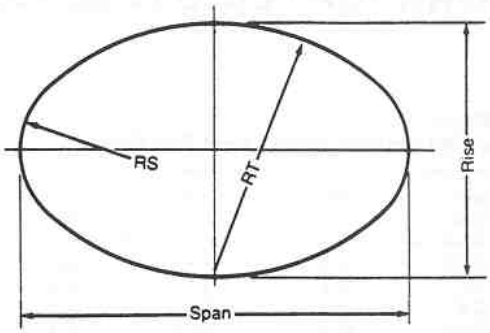
STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

Continued.
Layout Details Corrugated Steel Box Culverts

Rise, ft-in.	Span, ft-in.	Area ft²	Rise, ft-in.	Span, ft-in.	Area ft²
4-6	16-10	64.4	6-9	13-7	77.9
4-7	17-6	68.9	6-9	16-9	99.3
4-7	20-8	77.6	6-10	14-2	83.3
4-8	13-10	55.3	6-10	17-4	105.1
4-9	14-6	59.5	7-0	14-9	88.9
4-9	18-1	73.5	7-0	17-11	111.1
4-10	15-1	63.8	7-0	20-8	127.2
4-11	11-0	44.7	7-1	15-4	94.6
4-11	18-9	78.4	7-2	18-6	117.3
5-0	11-7	48.7	7-3	12-3	71.5
5-0	15-9	68.3	7-3	15-10	100.5
5-1	12-3	52.9	7-4	12-10	77.1
5-1	16-4	73.0	7-4	16-5	106.5
5-1	19-5	83.4	7-4	19-1	123.6
5-2	12-10	57.2	7-5	13-5	82.8
5-3	17-0	77.8	7-6	13-11	88.6
5-4	13-6	61.7	7-6	17-0	112.7
5-5	14-1	66.2	7-8	14-6	94.5
5-5	17-7	82.8	7-8	17-6	119.0
5-5	20-8	94.1	7-9	15-0	100.6
5-6	14-9	71.0	7-9	18-1	125.5
5-7	18-3	88.0	7-11	15-7	106.8
5-8	11-5	53.3	7-11	18-7	132.1
5-8	15-4	75.8	8-0	12-8	81.1
5-8	18-10	93.4	8-0	16-1	113.1
5-9	12-0	57.9	8-1	19-2	138.9
5-9	16-0	80.9	8-2	16-8	119.6
5-10	12-7	62.6	8-2	13-9	93.3
5-10	19-6	98.9	8-3	19-8	145.9
5-11	16-7	86.1	8-4	17-2	126.2
6-0	13-3	67.4	8-5	14-10	106.0
6-1	13-10	72.4	8-5	17-8	133.0
6-1	17-2	91.4	8-7	18-3	139.9
6-2	14-5	77.5	8-7	20-9	160.3
6-2	17-9	96.9	8-8	15-10	119.2
6-2	20-8	110.6	8-9	18-9	147.0
6-4	15-0	82.7	8-11	16-10	132.9
6-4	18-4	102.6	8-11	19-3	154.2
6-5	11-10	62.2	9-1	19-9	161.6
6-5	15-7	88.1	9-3	17-10	147.1
6-6	18-11	108.5	9-5	20-9	176.9
6-7	12-5	67.3	9-6	18-10	162.0
6-7	16-2	93.6	9-10	19-10	177.4
6-8	13-0	72.5	10-2	20-9	193.5
6-8	19-6	114.5			

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

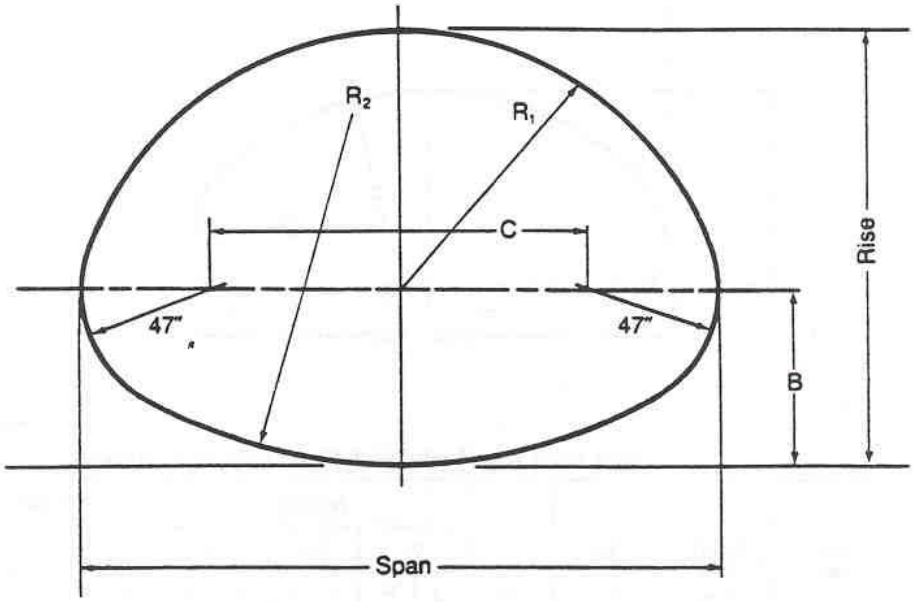


Long Span Horizontal Ellipse Sizes and Layout Details

Span, ft-in.	Rise, ft-in.	Area, ft²	Periphery						Inside Radius	
			Top or Bottom		Side		Total		Top	Side
			N	Pi	N	Pi	N	Pi	Rad. in.	Rad. in.
19- 4	12- 9	191	22	66	10	30	64	192	12- 6	4- 6
20- 1	13- 0	202	23	69	10	30	66	198	13- 1	4- 6
20- 2	11-11	183	24	72	8	24	64	192	13- 8	3- 7
20-10	12- 2	194	25	75	8	24	66	198	14- 3	3- 7
21- 0	15- 2	248	23	69	13	39	72	216	13- 1	5-11
21-11	13- 1	221	26	78	9	27	70	210	14-10	4- 1
22- 6	15- 8	274	25	75	13	39	76	228	14- 3	5-11
23- 0	14- 1	249	27	81	10	30	74	222	15- 5	4- 6
23- 3	15-11	288	26	78	13	39	78	234	14-10	5-11
24- 4	16-11	320	27	81	14	42	82	246	15- 5	6- 4
24- 6	14- 8	274	29	87	10	30	78	234	16- 6	4- 6
25- 2	14-11	287	30	90	10	30	80	240	17- 1	4- 6
25- 5	16- 9	330	29	87	13	39	84	252	16- 6	5-11
26- 1	18- 2	369	29	87	15	45	88	264	16- 6	6-10
26- 3	15-10	320	31	93	11	33	84	252	17- 8	4-11
27- 0	16- 2	334	32	96	11	33	86	258	18- 3	4-11
27- 2	19- 1	405	30	90	16	48	92	276	17- 1	7- 3
27-11	19- 5	421	31	92	16	48	94	282	17- 8	7- 3
28- 1	17- 1	369	33	99	12	36	90	270	18-10	5- 5
28-10	17- 5	384	34	102	12	36	92	276	19- 5	5- 5
29- 5	19-11	455	33	99	16	48	98	294	18-10	7- 3
30- 1	20- 2	472	34	102	16	48	100	300	19- 5	7- 3
30- 3	17-11	415	36	108	12	36	96	288	20- 7	5- 5
31- 2	21- 2	512	35	105	17	51	104	312	20- 0	7- 9
31- 4	18-11	454	37	111	13	39	100	300	21- 1	5-11
32- 1	19- 2	471	38	114	13	39	102	306	21- 8	5-11
32- 3	22- 2	555	36	108	18	54	108	324	20- 7	8- 2
33- 0	22- 5	574	37	111	18	54	110	330	21- 1	8- 2
33- 2	20- 1	512	39	117	14	42	106	318	22- 3	6- 4
34- 1	23- 4	619	38	114	19	57	114	342	21- 8	8- 8
34- 7	20- 8	548	41	123	14	42	110	330	23- 5	6- 4
34-11	21- 4	574	41	123	15	45	112	336	23- 5	6-10
35- 1	24- 4	665	39	117	20	60	118	354	22- 3	9- 1
35- 9	25- 9	718	39	117	22	66	122	366	22- 3	10- 0
36- 0	22- 4	619	42	126	16	48	116	348	24- 0	7- 3
36-11	25- 7	735	41	123	21	63	124	372	23- 5	9- 7
37- 2	22- 2	631	44	132	15	45	118	354	25- 2	6-10
38- 0	26- 7	785	44	132	22	66	128	384	24- 0	10- 0
38- 8	27-11	843	42	126	24	72	132	396	24- 0	10-11
40- 0	29- 7	927	43	129	26	78	138	414	27-11	11-10

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



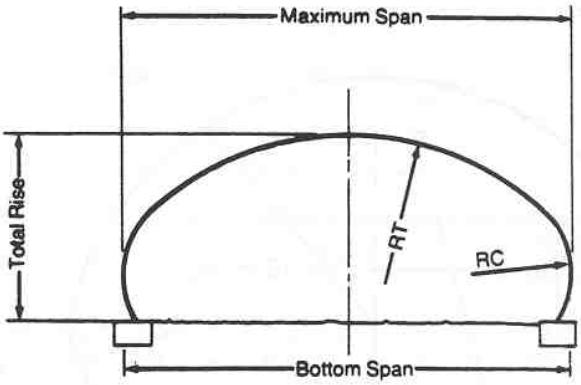
Long Span Pipe Arch Sizes and Layout Details

Span, ft-in.	Rise, ft-in.	Area, ft²	Total No. Plates	Periphery						B, in.	C, in.	Inside Radius	
				Top		Bottom		Total				R_1, in.	R_2, in.
				N	Pi	N	Pi	N	Pi				
20- 0	13-11	218	10	34	102	20	60	68	204	62.8	146.2	122.5	223.6
20- 6	14- 3	231	10	36	108	20	60	70	210	61.4	152.3	124.7	255.7
21- 5	14- 6	243	11	36	108	22	66	72	216	65.3	162.8	131.4	236.7
21-11	14-11	256	11	38	114	22	66	74	222	63.7	168.9	133.5	268.1
22- 5	15- 3	270	11	40	120	22	66	76	228	62.1	174.6	135.5	307.1
23- 4	15- 7	284	11	40	120	24	72	78	234	66.2	185.5	142.4	280.2
24- 2	15-11	297	12	40	120	26	78	80	240	70.7	196.2	149.7	262.1
24- 8	16- 2	312	12	42	126	26	78	82	246	68.8	202.2	151.4	292.2
25- 2	16- 7	326	12	44	132	26	78	84	252	66.9	207.9	153.2	328.6
25- 7	16-11	342	12	46	138	26	78	86	258	64.8	213.3	155.0	373.3
26- 7	17- 3	357	12	46	138	28	84	88	264	69.4	224.7	162.1	339.4
27- 6	17- 6	372	12	46	138	30	90	90	270	74.2	235.8	169.6	315.8
28- 0	17-10	388	12	48	144	30	90	92	276	72.1	241.5	171.1	350.2
28- 5	18- 3	405	13	50	150	30	90	94	282	69.9	246.8	172.7	392.3
29- 4	18- 6	421	13	50	150	32	96	96	288	74.8	258.2	180.2	361.1
30- 4	18-10	438	14	52	156	34	102	100	300	80.0	269.4	188.2	339.1

*Includes 14N for two N7 corner plates

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



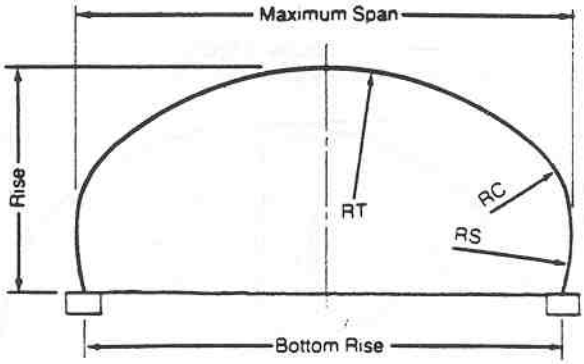
Long Span Low Profile Arch Sizes and Layout Details

Max. Span, ft-in.	Bottom Span, ft-in.	Total Rise, ft-in.	Area, ft²	Periphery						Inside Radius	
				Top		Side		Total		Top	Side
				N	Pi	N	Pi	N	Pi	rad. in.	rad. in.
20- 1	19-10	7- 6	121	23	69	6	18	35	105	13- 1	4- 6
19- 5	19- 1	6-10	105	23	69	5	15	33	99	13- 1	3- 7
21- 6	21- 4	7- 9	134	25	75	6	18	37	111	14- 3	4- 6
22- 3	22- 1	7-11	140	26	78	6	18	38	114	14-10	4- 6
23- 0	22- 9	8- 0	147	27	81	6	18	39	117	15- 5	4- 6
23- 9	23- 6	8- 2	154	28	84	6	18	40	120	16- 0	4- 6
24- 6	24- 3	8- 4	161	29	87	6	18	41	123	16- 6	4- 6
25- 2	25- 0	8- 5	169	30	90	6	18	42	126	17- 1	4- 6
25-11	25- 9	8- 7	176	31	93	6	18	43	129	17- 8	4- 6
27- 3	27- 1	10- 0	217	31	93	8	24	47	141	17- 8	6- 4
28- 1	27-11	9- 7	212	33	99	7	21	47	141	18-10	5- 5
28- 9	28- 7	10- 3	234	33	99	8	24	49	147	18-10	6- 4
28-10	28- 8	9- 8	221	34	102	7	21	48	144	19- 5	5- 5
30- 3	30- 1	9-11	238	36	108	7	21	50	150	20- 7	5- 5
30-11	30- 9	10- 8	261	36	108	8	24	52	156	20- 7	6- 4
31- 7	31- 2	12- 1	309	36	108	10	30	56	168	20- 7	7- 3
31- 0	30-10	10- 1	246	37	111	7	21	51	153	21- 1	5- 5
32- 4	31-11	12- 3	320	37	111	10	30	57	171	21- 1	7- 3
31- 9	31- 7	10- 3	255	38	114	7	21	52	156	21- 8	5- 5
33- 1	32- 7	12- 5	330	38	114	10	30	58	174	21- 8	7- 3
33- 2	33- 0	11- 1	289	39	117	8	24	55	165	22- 3	6- 4
34- 5	34- 1	13- 3	377	39	117	11	33	61	183	22- 3	8- 2
34- 7	34- 6	11- 4	308	41	123	8	24	57	183	23- 5	6- 4
37-11	37- 7	15- 8	477	41	123	14	42	69	207	23- 5	10-11
35- 4	35- 2	11- 5	318	42	126	8	24	58	174	24- 0	6- 4
38- 8	38- 4	15- 9	490	42	126	14	42	70	210	24- 0	10-11

NOTE: Larger sizes available for special designs.

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



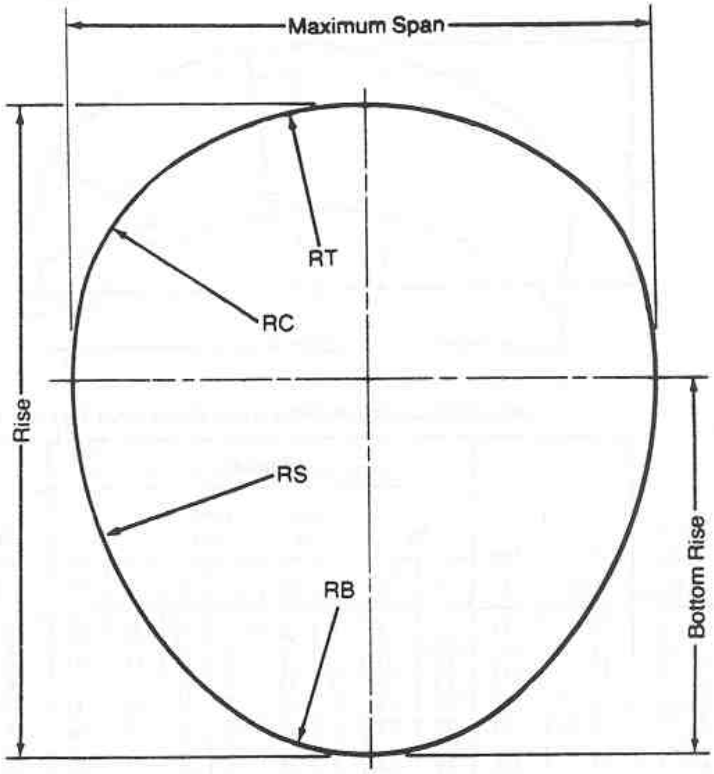
Long Span High Profile Arch Sizes and Layout Details

Max. Span, ft-in.	Bottom Span, ft-in.	Total Rise, ft-in.	Area, ft²	Periphery								Inside Radius		
				Top		Upper Side		Lower Side		Total		Top Radius, ft-in.	Upper Side, ft-in.	Lower Side, ft-in.
				N	Pi	N	Pi	N	Pi	N	Pi			
20- 1	19- 6	9- 1	152	23	69	5	15	3	9	39	117	13- 1	4- 6	13- 1
20- 8	18-10	12- 1	214	23	69	6	18	6	18	47	141	13- 1	5- 5	13- 1
21- 6	19-10	11- 8	215	25	75	5	15	6	18	47	141	14- 3	4- 6	14- 3
22-10	19-10	14- 7	285	25	75	7	21	8	24	55	165	14- 3	6- 4	14- 3
22- 3	20- 7	11-10	225	26	78	5	15	6	18	48	144	14-10	4- 6	14-10
22-11	20- 0	14- 0	276	26	78	6	18	8	24	54	162	14-10	5- 5	14-10
23- 0	21- 5	12- 0	235	27	81	5	15	6	18	49	147	15- 5	4- 6	15- 5
24- 4	21- 6	14-10	310	27	81	7	21	8	24	57	171	15- 5	6- 4	15- 5
23- 9	22- 2	12- 1	245	28	84	5	15	6	18	50	150	16- 0	4- 6	16- 0
24- 6	21-11	13- 9	289	29	87	5	15	8	24	55	165	16- 6	4- 6	16- 6
25- 9	23- 2	15- 2	335	29	87	7	21	8	24	59	177	16- 6	6- 4	16- 6
25- 2	23- 3	13- 2	283	30	90	5	15	7	21	54	162	17- 1	4- 6	17- 1
26- 6	24- 0	15- 3	348	30	90	7	21	8	24	60	180	17- 1	6- 4	17- 1
25-11	24- 1	13- 3	295	31	93	5	15	7	21	55	165	17- 8	4- 6	17- 8
27- 3	24-10	15- 5	360	31	93	7	21	8	24	61	183	17- 8	6- 4	17- 8
27- 5	25- 8	13- 7	317	33	99	5	15	7	21	57	171	18-10	4- 6	18-10
29- 5	27- 1	16- 5	412	33	99	8	28	8	24	65	195	18-10	7- 3	18-10
28- 2	25-11	14- 5	349	34	102	5	15	8	24	60	180	19- 5	4- 6	19- 5
30- 1	26- 9	18- 1	467	34	102	8	24	10	30	70	210	19- 5	7- 3	19- 5
30- 3	28- 2	15- 5	399	36	108	6	18	8	24	64	192	20- 7	5- 5	20- 7
31- 7	28- 4	18- 4	497	36	108	8	24	10	30	72	216	20- 7	7- 3	20- 7
31- 0	29- 0	15- 7	413	37	111	6	18	8	24	65	195	21- 1	5- 5	21- 1
31- 8	28- 6	17- 9	484	37	111	7	21	10	30	71	213	21- 1	6- 4	21- 1
32- 4	27-11	19-11	554	37	111	8	24	12	36	77	231	21- 1	7- 3	21- 1
31- 9	28- 8	17- 3	470	38	114	6	18	10	30	70	210	21- 8	5- 5	21- 8
33- 1	28- 9	20- 1	571	38	114	8	24	12	36	78	234	21- 8	7- 3	21- 8
32- 6	29- 6	17- 4	484	39	117	6	18	10	30	71	213	22- 3	5- 5	22- 3
33-10	29- 7	20- 3	588	39	117	8	24	12	36	79	237	22- 3	7- 3	22- 3
34- 0	31- 2	17- 8	514	41	123	6	18	10	30	73	219	23- 5	5- 5	23- 5
34- 7	30- 7	19-10	591	41	123	7	21	12	36	79	237	23- 5	6- 4	23- 5
35- 3	30- 7	21- 3	645	41	123	8	24	13	39	83	249	23- 5	7- 3	23- 5
37- 3	32- 6	23- 5	747	41	123	11	33	13	39	89	267	23- 5	10- 0	23- 5
34- 8	31-11	17-10	529	42	126	6	18	10	30	74	222	24- 0	5- 5	24- 0
35- 4	31- 5	20- 0	608	42	126	7	21	12	36	80	240	24- 0	6- 4	24- 0
36- 0	31- 5	21- 5	663	42	126	8	24	13	39	84	252	24- 0	7- 3	24- 0
38- 0	33- 5	23- 6	767	42	126	11	33	13	39	90	270	24- 0	10- 0	24- 0

NOTE. Larger sizes available for special designs

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Long Span Pear Shape Sizes and Layout Details

Max. Span, ft-in.	Rise, ft-in.	Rise Bottom, ft-in.	Area	Periphery										Inside Radius			
				Top		Corner		Side		Bottom		Total		Bottom Radius, ft-in.	Side Radius, ft-in.	Corner Radius, ft-in.	Top Radius, ft-in.
				N	Pi	N	Pi	N	Pi	N	Pi	N	Pi				
23- 8	25- 8	14-11	481	25	75	5	15	24	72	15	30	98	294	8-11	16- 7	6- 3	14- 8
24- 0	25-10	15- 1	496	22	66	7	21	22	66	20	60	100	300	9-11	17- 4	7- 0	16- 2
25- 6	25-11	15-10	521	27	81	7	21	20	60	21	63	102	306	10- 7	18- 1	6-11	15-10
24-10	27- 8	16- 9	544	27	81	5	15	25	75	18	54	105	315	9- 3	19- 8	5- 9	15-11
27- 5	27- 0	18- 1	578	30	90	6	18	26	78	16	48	110	330	9- 7	20- 4	4- 7	19-11
26- 8	28- 3	18- 0	593	28	84	5	15	30	90	12	36	110	330	8- 0	20- 1	4- 9	20-11
28- 1	27-10	16-10	624	27	81	8	24	22	66	25	75	112	336	12- 2	19- 0	7- 3	20- 5
28- 7	30- 7	19- 7	689	32	96	7	21	24	72	24	72	118	354	11- 2	24- 0	7- 0	18- 2
30- 0	29- 8	20- 0	699	32	96	8	24	23	69	25	75	119	357	11-11	24- 0	6- 7	21-10
30- 0	31- 2	19-11	736	34	102	7	21	24	72	26	78	122	366	12- 1	24- 0	7- 0	19- 3

Figure 12.4.44 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Helical Pipe Availability, Weights

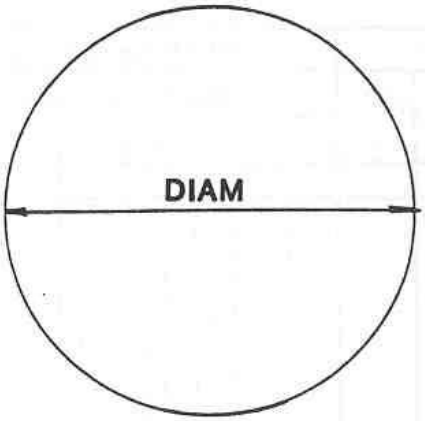
CORR. PATTERN				WEIGHT (Lbs./Lineal Ft.)					
1-1/2 x 1/4	2-2/3 x 1/2	3 x 1	6" x 1	Equiv. Standard Gauge					
				18	16	14	12	10	8
Diameter (In.)									
6				1.4	1.7				
8				1.8	2.2				
10				2.2	2.7				
	12				3.2	4.0	5.5		
	15				3.9	4.9	6.8		
	18				4.7	5.9	8.1		
	21				5.4	6.8	9.4		
	24				6.2	7.8	10.7	13.8	
	27				7.0	8.7	12.1	15.4	
	30				7.8	9.6	13.4	17.1	
		30			8.9	11.2	15.5	19.9	
	36					11.5	16.0	20.5	
		36			10.7	13.4	18.5	23.7	
	42						18.6	23.8	
		42			12.4	15.5	21.5	27.5	
	48						21.2	27.2	32.7
		48			14.1	17.7	24.5	31.4	37.8
			48		12.5	15.6	21.8	28.1	34.1
	54						23.8	30.5	36.7
		54			15.8	19.9	27.5	35.2	42.4
			54		14.0	17.5	24.5	31.5	38.3
	60							33.9	40.8
		60			17.6	22.0	30.5	39.0	47.0
			60		15.5	19.4	27.2	34.9	42.5
	66							37.2	44.8
		66			17.0	21.3	29.8	38.4	46.6
	72								48.8
		72				26.3	36.5	46.7	56.2
			72			23.2	32.5	41.8	50.8
	78								52.9
		78				28.5	39.5	50.5	60.8
			78			25.1	35.2	45.2	55.0
	84								56.9
		84				30.7	42.5	54.3	65.4
			84				37.8	48.7	59.1
		90					45.4	58.2	70.0
			90				40.5	52.1	63.3
		96					48.4	62.0	74.6
			96				43.2	55.5	67.5
		102					51.4	65.8	79.3
			102				45.8	58.9	71.6
		108					54.4	69.7	83.9
			108				48.5	62.4	75.8
		114					57.4	73.5	88.5
			114				51.2	65.8	80.0
		120					60.4	77.3	93.1
			120				53.8	69.2	84.1

NOTES: 1. Sizes 6" thru 10" are available in helical corrugation only.

2. Sizes 12" through 21" in helical configuration have corrugation depth of 7/16" rather than 1/2".

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association)

STANDARD SIZES
FOR ALUMINUM CULVERTS



Geometric Data — Structural Plate Pipe

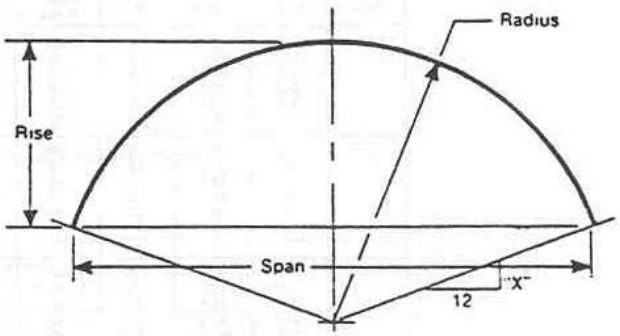
Nom. Diam. In.	Area Sq. Ft.	Total N	Nom. Diam. In.	Area Sq. Ft.	Total N
60	19	20	162	145	54
66	23	22	168	156	56
72	27	24	174	167	58
78	32	26	180	179	60
84	38	28	186	191	62
90	44	30	192	204	64
96	50	32	198	217	66
102	56	34	204	231	68
108	63	36	210	245	70
114	71	38	216	259	72
120	79	40	222	274	74
126	87	42	228	289	76
132	95	44	234	305	78
138	104	46	240	321	80
144	114	48	246	337	82
150	124	50	252	354	84
156	134	52	—	—	—

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

GEOMETRIC DATA – ARCH

"X" Values For Rise/Span Ratio			
R/S Ratio	"X"	R/S Ratio	"X"
.30	6.40	.42	2.10
.31	5.96	.43	1.82
.32	5.54	.44	1.54
.33	5.13	.45	1.27
.34	4.74	.46	1.00
.35	4.37	.47	.74
.36	4.01	.48	.48
.37	3.67	.49	.24
.38	3.33	.50	.00
.39	3.01	.51	.24
.40	2.70	.52	.47
.41	2.40		

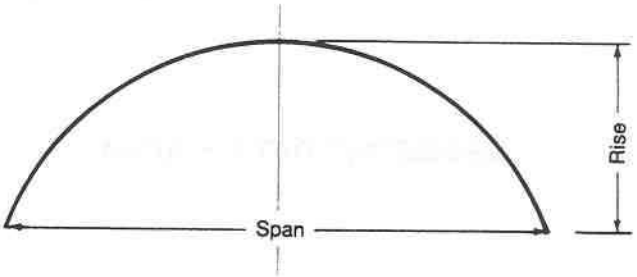


Typical Section

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
5-0	2-7	10.4	10	.52	30
	2-3	8.5	9	.44	30 1/4
	1-9	6.5	8	.36	31 1/4
6-0	3-2	14.9	12	.52	36
	2-9	12.6	11	.46	36 1/4
	2-4	10.2	10	.38	37 1/4
	1-10	7.8	9	.30	40 1/2
7-0	3-8	20.3	14	.52	42
	3-3	17.5	13	.46	42
	2-10	14.8	12	.40	43
	2-4	12.0	11	.34	45 1/4
8-0	4-2	26.4	16	.52	48
	3-9	23.3	15	.47	48
	3-4	20.2	14	.42	48 1/4
	2-11	17.0	13	.36	50 1/2
	2-5	13.6	12	.30	54 1/2
9-0	4-8	33.4	18	.50	54
	4-3	29.9	17	.48	54
	3-10	26.3	16	.43	54 1/2
	3-5	22.8	15	.38	56
	2-11	19.1	14	.33	59
10-0	5-2	41.2	20	.52	60
	4-10	37.3	19	.48	60
	4-5	33.3	18	.44	60 1/2
	3-11	29.4	17	.40	61 1/2
	3-6	25.3	16	.35	64
	3-0	21.1	15	.30	68 1/2
11-0	5-8	49.8	22	.52	66
	5-4	45.5	21	.48	66
	4-11	41.2	20	.45	66 1/2
	4-6	36.8	19	.41	67 1/2
	4-0	32.4	18	.36	69 1/4
	3-6	27.8	17	.32	72 3/4

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS



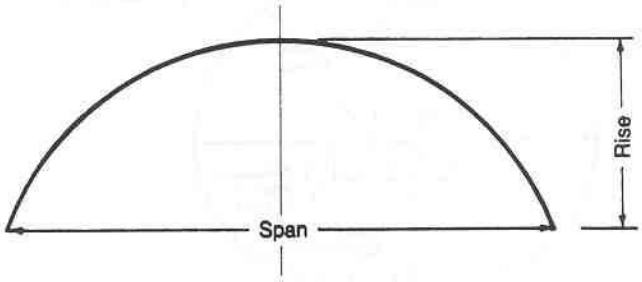
ARCH
(CONT'D)

Geometric Data—Arch (Continued)

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
12-0	6-3	59.3	24	.52	72
	5-10	54.5	23	.49	72
	5-5	49.8	22	.45	72½
	5-0	45.0	21	.42	73¼
	4-7	40.2	20	.38	75
	4-1	35.3	19	.34	77½
13-0	6-9	69.5	26	.52	78
	6-4	64.4	25	.49	78
	5-11	59.3	24	.46	78¼
	5-6	54.1	23	.42	79
	5-1	48.9	22	.39	80½
	4-7	43.6	21	.35	82¾
	4-1	38.1	20	.31	86½
14-0	7-3	80.6	28	.52	84
	6-10	75.1	27	.49	84
	6-5	69.5	26	.46	84¼
	6-0	64.0	25	.43	85
	5-7	58.4	24	.40	86
	5-2	52.7	23	.37	88
	4-8	46.9	22	.33	91½
15-0	7-9	92.5	30	.52	90
	7-5	86.5	29	.49	90
	7-0	80.6	28	.46	90¼
	6-7	74.7	27	.44	91
	6-1	68.7	26	.41	92
	5-8	62.6	25	.38	93½
	5-2	56.4	24	.34	96¼
	4-8	50.0	23	.31	100½
16-0	8-3	105.2	32	.52	96
	7-11	98.9	31	.49	96
	7-6	92.5	30	.47	96¼
	7-1	86.2	29	.44	96¾
	6-8	79.8	28	.41	97¾
	6-2	73.3	27	.39	99½
	5-9	66.8	26	.36	101½
	5-3	60.0	25	.32	105
17-0	8-10	118.7	34	.52	102
	8-5	112.0	33	.49	102
	8-0	105.2	32	.47	102¼
	7-7	98.5	31	.45	102¾
	7-2	91.7	30	.42	103½
	6-9	84.9	29	.39	105
	6-3	77.9	28	.37	107
	5-9	70.9	27	.34	110
	5-3	63.5	26	.31	114½
18-0	9-4	133.1	36	.52	108
	8-11	125.9	35	.50	108
	8-6	118.8	34	.47	108¼
	8-1	111.6	33	.45	108½
	7-8	104.5	32	.43	109¼
	7-3	97.2	31	.40	110½
	6-9	89.9	30	.38	112½
	6-4	82.5	29	.35	115
	5-9	74.8	28	.32	118¾
19-0	9-10	148.2	38	.52	114
	9-5	140.7	37	.50	114
	9-0	133.2	36	.48	114¼
	8-8	125.8	35	.45	114½
	8-2	118.0	34	.43	115¼
	7-9	110.4	33	.41	116¼
	7-4	102.7	32	.38	118
	6-10	94.9	31	.36	120¼
	6-4	86.9	30	.33	123½
	5-10	78.7	29	.31	128¼
20-0	10-4	164.2	40	.52	120
	10-0	156.3	39	.50	120
	9-7	148.3	38	.48	120
20-0 cont.	9-2	140.4	37	.46	120½
	8-9	132.4	36	.44	121
	8-3	124.4	35	.41	122¼
	7-10	116.3	34	.39	123½
	7-4	108.4	33	.37	125¾
	6-10	99.8	32	.34	128½
	6-4	91.2	31	.32	132½
21-0	10-10	181.0	42	.52	126
	10-6	172.7	41	.50	126
	10-1	164.3	40	.48	126
	9-8	156.0	39	.46	126½
	9-3	147.6	38	.44	127
	8-10	139.2	37	.42	128
	8-4	130.7	36	.40	129¼
	7-11	122.2	35	.38	131¼
	7-5	113.5	34	.35	133¾
	6-11	104.6	33	.33	137¼
	6-4	95.4	32	.30	142
22-0	11-5	198.6	44	.52	132
	11-0	189.9	43	.50	132
	10-7	181.1	42	.48	132
	10-2	172.4	41	.46	132½
	9-9	163.6	40	.44	133
	9-4	154.8	39	.42	133¾
	8-11	146.0	38	.40	135
	8-5	137.0	37	.38	135¾
	7-11	127.9	36	.36	139
	7-5	118.7	35	.34	142½
	6-11	109.2	34	.31	142¼
23-0	11-11	217.1	46	.52	138
	11-6	207.9	45	.50	138
	11-1	198.8	44	.48	138
	10-8	189.6	43	.47	138¼
	10-3	180.5	42	.45	139
	9-10	171.3	41	.43	139½
	9-5	162.0	40	.41	140¾
	8-11	152.7	39	.39	142¼
	8-6	143.2	38	.37	144½
	8-0	133.6	37	.35	147¼
	7-6	123.8	36	.33	151
	6-11	113.8	35	.30	156
24-0	12-5	236.3	48	.52	144
	12-0	226.8	47	.50	144
	11-7	217.2	46	.48	144
	11-3	207.7	45	.47	144¼
	10-10	198.1	44	.45	144¾
	10-4	188.5	43	.43	145½
	9-11	178.9	42	.41	146½
	9-5	169.2	41	.39	148
	9-0	159.3	40	.38	150
	8-6	149.4	39	.36	152½
	8-0	139.2	38	.33	155¾
	7-6	128.9	37	.31	160¼
25-0	12-11	256.4	50	.52	150
	12-6	246.4	49	.50	150
	12-2	236.5	48	.49	150
	11-9	226.6	47	.47	150½
	11-4	216.6	46	.45	150¾
	10-11	206.6	45	.44	151½
	10-5	196.6	44	.42	152½
	10-0	186.4	43	.40	153¾
	9-6	176.3	42	.38	155½
	9-1	165.9	41	.36	157¾
	8-7	155.4	40	.34	160¾
	8-1	144.7	39	.32	164¾
	7-6	133.7	38	.30	170
26-0	13-5	277.3	52	.52	156
	13-1	266.9	51	.50	156
	12-8	256.6	50	.49	156

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS



ARCH (CONT'D)

Geometric Data—Arch (Continued)

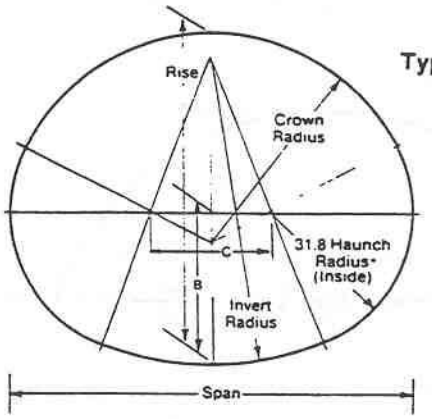
Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
26-0 cont.	12-3	246.2	49	.47	156¼
	11-10	235.9	48	.46	156¾
	11-5	225.5	47	.44	157¼
	11-0	215.1	46	.42	158¼
	10-6	204.6	45	.40	159½
	10-1	194.0	44	.39	161
	9-7	183.3	43	.37	163¼
	9-1	172.4	42	.35	166
	8-7	161.4	41	.33	169½
	8-1	150.1	40	.31	174
27-0	14-0	299.0	54	.52	162
	13-7	288.2	53	.50	162
	13-2	277.5	52	.49	162
	12-9	266.7	51	.47	162¼
	12-4	256.0	50	.46	162¾
	11-11	245.2	49	.44	163¼
	11-6	234.4	48	.43	164
	11-1	223.5	47	.41	165¼
	10-7	212.6	46	.39	166¾
	10-2	201.4	45	.38	168¾
	9-8	190.2	44	.36	171¼
	9-2	178.8	43	.34	174½
	8-7	167.2	42	.32	178½
	8-1	155.3	41	.30	183¾
28-0	14-6	321.5	56	.52	168
	14-1	310.4	55	.50	168
	13-8	299.2	54	.49	168
	13-3	288.1	53	.47	168¼
	12-10	276.9	52	.46	168½
	12-5	265.7	51	.44	169¼
	12-0	254.5	50	.43	170
	11-7	243.2	49	.41	171
	11-1	231.9	48	.40	172½
	10-8	220.4	47	.38	174¼

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
28-0 cont.	10-2	208.8	46	.36	176½
	9-8	197.1	45	.35	179½
	9-2	185.1	44	.33	183¼
	8-8	172.9	43	.31	188
29-0	15-0	344.8	58	.52	174
	14-7	333.3	57	.50	174
	14-2	321.7	56	.49	174
	13-10	310.2	55	.48	174¼
	13-5	298.6	54	.46	174½
	13-0	287.1	53	.45	175
	12-6	275.4	52	.43	175¾
	12-1	263.8	51	.42	176¾
	11-8	252.0	50	.40	178¼
	11-2	240.2	49	.39	180
	10-9	228.2	48	.37	182
	10-3	216.1	47	.35	184¾
	9-9	203.8	46	.34	188
	9-2	191.3	45	.32	192¼
	8-8	178.5	44	.30	197¾
30-0	15-6	369.0	60	.52	180
	15-1	357.1	59	.50	180
	14-9	345.1	58	.49	180
	14-4	333.2	57	.48	180¼
	13-11	321.2	56	.46	180½
	13-6	309.2	55	.45	181
	13-1	297.2	54	.44	181¾
	12-7	285.1	53	.42	182¾
	12-2	273.0	52	.41	184
	11-9	260.8	51	.39	185½
	11-3	248.5	50	.37	187½
	10-9	236.0	49	.36	190
	10-3	223.3	48	.34	193
	9-9	210.5	47	.32	197
	9-2	197.3	46	.31	201¾

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

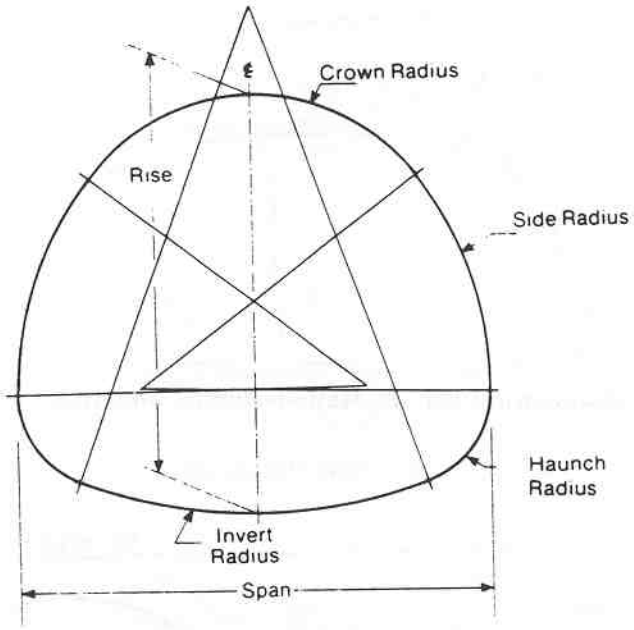
Geometric Data—Pipe Arch



Span Ft.-In.	Rise Ft.-In.	Area Sq. Ft.	Required N				Inside Radius		B	C
			Total	Crown	Invert	Haunch	Crown In.	Invert In.		
6-7	5-8	29.6	25	8	3	7	41.5	69.9	32.5	15.3
6-11	5-9	31.9	26	9	3	7	43.7	102.9	32.4	19.6
7-3	5-11	34.3	27	10	3	7	45.6	188.3	32.2	23.8
7-9	6-0	36.8	28	9	5	7	51.6	83.8	33.8	29.0
8-1	6-1	39.3	29	10	5	7	53.3	108.1	33.5	33.3
8-5	6-3	41.9	30	11	5	7	54.9	150.1	33.2	37.4
8-10	6-4	44.5	31	10	7	7	63.3	93.0	35.6	42.8
9-3	5-5	47.1	32	11	7	7	64.4	112.6	35.2	47.1
9-7	6-6	49.9	33	12	7	7	65.4	141.6	34.7	51.3
9-11	6-8	52.7	34	13	7	7	66.4	188.7	34.2	55.3
10-3	6-9	55.5	35	14	7	7	67.4	278.8	33.5	59.2
10-9	6-10	58.4	36	13	9	7	77.5	139.6	36.8	65.2
11-1	7-0	61.4	37	14	9	7	77.8	172.0	36.1	69.3
11-5	7-1	64.4	38	15	9	7	78.2	222.0	35.3	73.3
11-9	7-2	67.5	39	16	9	7	78.7	309.5	34.4	77.1
12-3	7-3	70.5	40	15	11	7	90.8	165.2	38.4	83.4
12-7	7-5	73.7	41	16	11	7	90.5	200.0	37.5	87.4
12-11	7-6	77.0	42	17	11	7	90.4	251.7	36.5	91.3
13-1	8-2	83.0	43	18	13	6	88.8	143.6	42.0	93.6
13-1	8-4	86.8	44	21	11	6	81.7	300.8	35.8	93.7
13-11	8-5	90.3	45	18	15	6	100.4	132.0	46.0	103.3
14-0	8-7	94.2	46	21	13	6	90.3	215.1	39.4	104.5
13-11	9-5	101.5	47	23	14	5	86.2	159.3	42.8	103.9
14-3	9-7	105.7	48	24	14	5	87.2	176.3	42.0	107.0
14-8	9-8	109.9	49	24	15	5	90.9	166.2	44.0	112.3
14-11	9-10	114.2	50	25	15	5	91.8	183.0	43.2	115.5
15-4	10-0	118.6	51	25	16	5	95.5	173.0	45.3	120.8
15-7	10-2	123.1	52	26	16	5	96.4	189.6	44.4	123.9
16-1	10-4	127.6	53	26	17	5	100.2	179.7	46.6	129.2
16-4	10-6	132.3	54	27	17	5	101.0	196.1	45.7	132.3
16-9	10-8	136.9	55	27	18	5	105.0	186.3	47.9	137.7
17-0	10-10	141.8	56	28	18	5	105.7	202.5	46.9	140.8
17-3	11-0	146.7	57	29	18	5	106.5	221.3	45.9	143.8
17-9	11-2	151.6	58	29	19	5	110.4	208.9	48.2	149.3
18-0	11-4	156.7	59	30	19	5	111.1	227.3	47.2	152.3
18-5	11-6	161.7	60	30	20	5	115.2	215.2	49.6	157.8
18-8	11-8	167.0	61	31	20	5	115.8	233.3	48.5	160.7
19-2	11-9	172.2	62	31	21	5	119.9	221.5	50.9	166.2
19-5	11-11	177.6	63	32	21	5	120.5	239.3	49.8	169.2
19-10	12-1	182.9	64	32	22	5	124.7	227.7	52.3	174.8
20-1	12-3	188.5	65	33	22	5	125.2	245.3	51.1	177.7
20-1	12-6	194.4	66	35	21	5	122.5	310.8	46.2	177.5
20-10	12-7	199.7	67	34	23	5	130.0	251.2	52.5	186.2
21-1	12-9	205.5	68	35	23	5	130.5	270.9	51.2	189.1
21-6	12-11	211.2	69	35	24	5	134.8	257.2	53.9	194.8

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS



Typical Section

Geometric Data—Vehicular Underpass

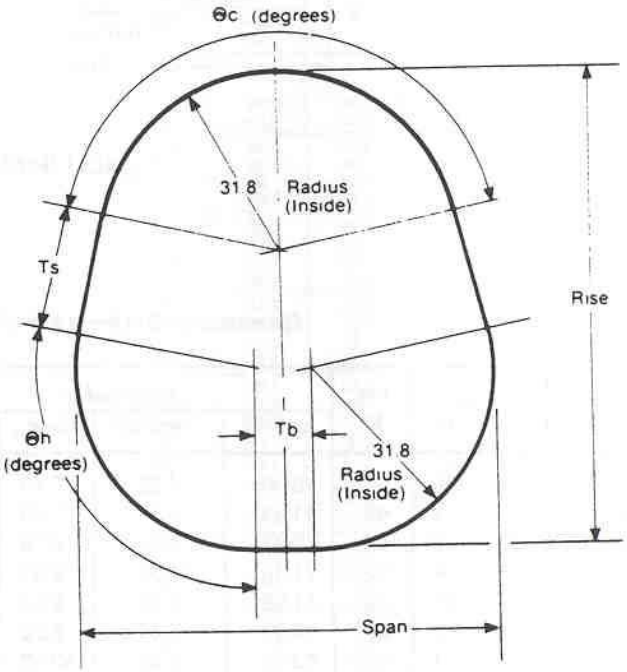
Span		Rise		Tot	Required N				Inside Radius (Inches)			
Ft	In.	Ft	In.	N	Invert	Haunch	Side	Crown	Invert	Haunch	Side	Crown
12	1	11	0	47	10.00	4.32	7.69	12.99	135.95	37.95	88.00	67.95
12	10	11	2	49	11.04	4.44	7.50	14.10	148.53	38.53	86.78	74.53
13	0	12	0	51	10.97	4.27	8.79	13.91	160.54	37.54	98.19	72.54
13	8	12	4	53	11.98	4.36	8.67	14.96	167.77	37.77	102.62	76.77
14	0	12	11	55	11.99	4.39	9.62	14.98	182.90	37.90	110.65	76.90
14	6	13	5	57	13.07	4.61	9.26	16.18	174.88	38.88	124.73	78.88
14	8	14	1	59	13.00	4.42	10.58	15.99	192.96	37.96	130.01	78.96
15	5	14	5	61	14.04	4.59	10.33	17.11	201.54	38.54	135.39	83.54
15	6	15	2	63	13.97	4.45	11.61	16.92	211.59	37.59	149.14	81.59
16	2	15	6	65	14.99	4.50	11.52	17.97	216.85	37.85	154.40	85.85
16	6	16	0	67	14.07	4.73	12.10	19.29	272.34	39.34	153.89	89.34
16	8	16	4	68	15.01	4.49	12.49	19.03	246.17	38.17	160.82	89.17
17	3	17	1	70	15.04	5.71	12.20	19.13	214.64	47.64	171.19	90.64
18	5	16	11	72	16.09	5.87	11.95	20.27	249.37	48.37	155.02	100.37
19	0	17	3	74	17.02	5.60	12.36	21.06	262.29	47.29	153.14	105.29
19	7	17	7	76	17.07	5.79	13.06	21.24	296.21	48.21	154.46	108.21
20	5	17	9	78	18.08	5.78	13.05	22.27	317.39	48.39	149.94	115.39

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Geometric Data—Pedestrian/Animal Underpass

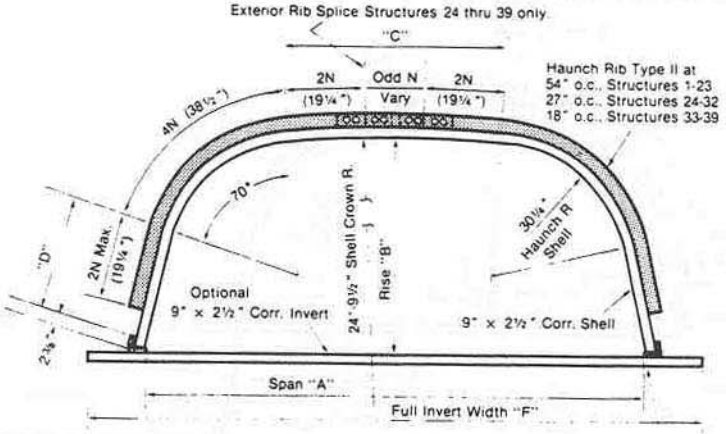
Span Ft.-In.	Rise Ft.-In.	Total N	Tb In.	Ts In.	Θc Degrees	Θh Degrees
6-1	5-9	24	9.2	7.2	100.2	129.9
6-3	6-1	25	11.1	11.0	119.3	120.4
6-3	6-6	26	11.6	15.6	136.5	111.7
6-2	7-0	27	10.2	21.1	152.2	103.9
6-3	7-4	28	11.6	25.2	153.3	103.4
6-1	7-10	29	9.8	30.9	161.7	99.2
6-3	8-2	30	11.3	35.0	161.3	99.3



Typical Section

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS
Box Culvert Geometric Data



BOX CULVERT CROSS SECTION

Side Angle "E"

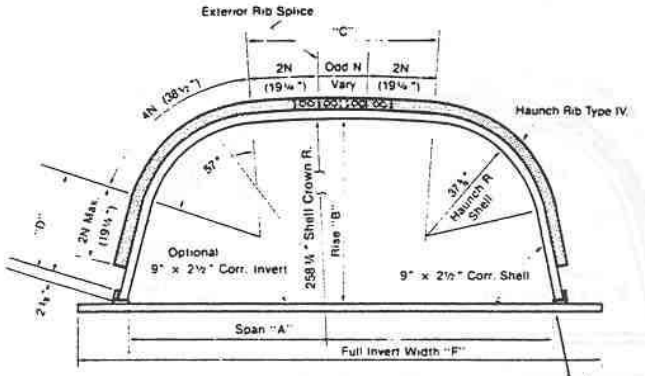
Structure Number	Span "A" (Ft.-In.)	Rise "B" (Ft.-In.)	Area (Sq. Ft.)	SHELL Crown Width "C" (N)	SHELL Leg Length "D" (N)	SHELL Side Angle "E" Deg. Min.	SHELL Total N	SHELL Haunch Plate Length (N)	SHELL Crown Plate Length (N)	SHELL Bolts/Ft.	FULL INVERT Width "F" (N)	FULL INVERT Supplemental/Stub Pl. Thick.	FULL INVERT Supplemental/Stub Pl. Width (N)	FULL INVERT Weight/Ft.	FULL INVERT Bolts/Ft.
1	8- 9	2- 6	18.4	5	.5	15-24	14	1 @ 14	—	6.67	13	—	—	23.06	5.78
2	9- 2	3- 3	25.4	5	1.5	15-24	16	2 @ 8	—	11.56	13	—	—	23.06	5.78
3	9- 7	4- 1	32.6	5	2.5	15-24	18	2 @ 9	—	12.00	14	—	—	24.44	6.00
4	10- 0	4-10	40.2	5	3.5	15-24	20	2 @ 10	—	12.44	14	—	—	24.44	6.00
5	10- 6	5- 7	48.1	5	4.5	15-24	22	2 @ 11	—	12.89	15	—	—	25.82	6.22
6	10-11	6- 4	56.4	5	5.5	15-24	24	2 @ 12	—	13.33	17	—	—	28.58	6.57
7	11- 4	7- 2	65.0	5	6.5	15-24	26	2 @ 13	—	13.78	17	—	—	28.58	6.67
8	10- 2	2- 8	23.0	7	.5	13-33	16	2 @ 8	—	12.89	15	—	—	25.82	6.22
9	10- 7	3- 5	31.1	7	1.5	13-33	18	2 @ 9	—	13.33	15	—	—	25.82	6.22
10	10-11	4- 3	39.5	7	2.5	13-33	20	2 @ 10	—	13.78	17	—	—	28.58	6.57
11	11- 4	5- 0	48.2	7	3.5	13-33	22	2 @ 11	—	14.22	17	—	—	28.58	6.57
12	11- 8	5- 9	57.2	7	4.5	13-33	24	2 @ 12	—	14.67	17	—	—	28.58	6.57
13	12- 1	6- 7	66.4	7	5.5	13-33	26	2 @ 13	—	15.11	17	—	—	28.58	6.57
14	12- 5	7- 4	76.0	7	6.5	13-33	28	2 @ 14	—	15.56	17	—	—	28.58	6.57
15	11- 7	2-10	28.1	9	0.5	11-42	18	2 @ 9	—	14.67	17	—	—	28.58	6.57
16	11-11	3- 7	37.4	9	1.5	11-42	20	2 @ 10	—	15.11	17	—	—	28.58	6.57
17	12- 3	4- 5	46.9	9	2.5	11-42	22	2 @ 11	—	15.56	17	—	—	28.58	6.67
18	12- 7	5- 2	56.6	9	3.5	11-42	24	2 @ 12	—	16.00	19	—	—	32.02	7.11
19	12-11	6- 0	66.6	9	4.5	11-42	26	2 @ 13	—	16.44	19	—	—	32.02	7.11
20	13- 3	6- 9	76.9	9	5.5	11-42	28	2 @ 14	—	16.89	19	—	—	32.02	7.11
21	13- 0	3- 0	33.8	11	0.5	9-52	20	2 @ 10	—	16.44	19	—	—	32.02	7.11
22	13- 4	3-10	44.2	11	1.5	9-52	22	2 @ 11	—	16.89	19	—	—	32.02	7.11
23	13- 7	4- 7	54.8	11	2.5	9-52	24	2 @ 12	—	17.33	19	—	—	32.02	7.11
24	13-10	5- 5	65.6	11	3.5	9-52	26	2 @ 13	—	23.11	19	—	—	32.02	7.11
25	14- 1	6- 2	76.6	11	4.5	9-52	28	2 @ 14	—	23.56	20	—	—	33.34	12.44
26	14- 5	3- 3	40.0	13	0.5	8-1	22	2 @ 11	—	22.67	20	—	—	33.34	12.44
27	14- 8	4- 1	51.5	13	1.5	8-1	24	2 @ 8	8	25.56	21	.100	2	40.23	12.37
28	14-10	4-10	63.2	13	2.5	8-1	26	2 @ 9	8	26.44	21	.100	2	40.23	12.37
29	15- 1	5- 8	75.1	13	3.5	8-1	28	2 @ 10	8	26.89	21	.100	2	40.23	12.37
30	15- 4	6- 5	87.2	13	4.5	8-1	30	2 @ 11	8	27.33	21	.100	2	40.23	12.37
31	15- 6	7- 3	99.4	13	5.5	8-1	32	2 @ 12	8	27.78	22	.100	2	41.61	12.49
32	15- 9	8- 0	111.8	13	6.5	8-1	34	2 @ 13	8	28.22	22	.100	2	41.61	12.49
33	15-10	3- 6	46.8	15	0.5	6-10	24	2 @ 8	8	32.22	22	.100	2	41.61	12.49
34	16- 0	4- 3	59.5	15	1.5	6-10	26	2 @ 9	8	33.56	22	.100	2	41.61	12.49
35	16- 2	5- 1	72.3	15	2.5	6-10	28	2 @ 10	8	34.89	23	.100	2	42.99	13.11
36	16- 4	5-11	85.2	15	3.5	6-10	30	2 @ 11	8	35.33	23	.100	3	45.75	13.11
37	16- 6	6- 8	98.3	15	4.5	6-10	32	2 @ 12	8	35.78	23	.100	3	45.75	13.11
38	16- 8	7- 6	111.5	15	5.5	6-10	34	2 @ 13	8	36.22	23	.100	3	45.75	13.11
39	16-10	8- 3	124.8	15	6.5	6-10	36	2 @ 14	8	36.67	24	.100	3	47.13	13.43

NOTES:

1) "N" equals 9.62".
2) All crowns of shells have Type IV ribs outside at 18" on centers.
3) Weights per foot listed do not include bolt weight.
4) Weight per foot of full invert includes 3½ x 3 x ¼ connecting angle and scalloped closure plate for each side. Inverts for 20N and greater are two-piece.
5) Weight per foot of footing pad includes a 3½ x 3 x ¼-in. connecting angle for each side. Optional wale beam not included.
6) Full invert plates are .100 thick. When reactions to invert require additional thickness supplemental plates of thickness and width listed are furnished to bolt between full invert and side connecting angle.
7) Width of footing pad is for each side.
8) For structures using short footing pads with leg length "D" equal to 3.5 N or more, either wale beam stiffeners should be used to avoid

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS
Box Culvert Geometric Data



BOX CULVERT CROSS SECTION

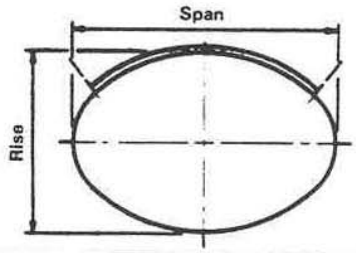
Structure Number	Span "A" (Ft.-In.)	Rise "B" (Ft.-In.)	Area (Sq. Ft.)	SHELL: Crown Width "C" (N)	SHELL: Leg Length "D" (N)	SHELL: Side Angle "E" Deg. Min.	SHELL: Total N	SHELL: Haunch Plate Length (N)	SHELL: Crown Plate Length (N)	SHELL: Bolts Per Foot	FULL INVERT: Width "F" (N)	FULL INVERT: Supplemental Stub Plates Thickness	FULL INVERT: Supplemental Stub Plates Width (N)	FULL INVERT: Weight Per Foot	FULL INVERT: Bolts Per Foot
40	17- 9	3-10	54.4	17	.5	14-54	26	8	10	33.56	25	.100	3	48.51	13.56
41	18- 2	4- 7	68.3	17	1.5	14-54	28	9	10	34.89	25	.100	3	48.51	13.56
42	18- 7	5- 4	82.5	17	2.5	14-54	30	10	10	36.22	26	.100	3	49.88	13.78
43	19- 0	6- 1	97.1	17	3.5	14-54	32	11	10	36.67	27	.100	3	51.26	14.00
44	19- 5	6-11	111.9	17	4.5	14-54	34	12	10	37.11	27	.100	3	51.26	14.00
45	19-10	7- 8	127.1	17	5.5	14-54	36	13	10	37.56	28	.100	3	52.64	14.22
46	20- 3	8- 5	142.6	17	6.5	14-54	38	14	10	38.00	28	.100	3	52.64	14.22
47	19- 1	4- 2	63.3	19	.5	12-47	28	8	12	34.89	27	.100	3	51.26	14.00
48	19- 5	4-11	78.3	19	1.5	12-47	30	9	12	36.22	27	.100	3	51.26	14.00
49	19- 9	5- 8	93.6	19	2.5	12-47	32	10	12	37.56	27	.100	3	51.26	14.00
50	20- 1	6- 6	109.2	19	3.5	12-47	34	11	12	38.00	28	.100	3	52.64	14.22
51	20- 6	7- 3	125.0	19	4.5	12-47	36	12	12	54.44	29	.125	3	56.09	14.44
52	20-10	8- 1	141.2	19	5.5	12-47	38	13	12	54.89	29	.100	3	54.02	14.44
53	21- 2	8-10	157.6	19	6.5	12-47	40	14	12	55.33	30	.150	3	59.54	14.67
54	20- 4	4- 6	73.1	21	.5	10-40	30	8	14	49.56	29	.150	3	58.16	14.44
55	20- 7	5- 3	89.2	21	1.5	10-40	32	9	14	52.22	29	.125	3	56.09	14.44
56	20-11	6- 1	105.5	21	2.5	10-40	34	10	14	54.89	29	.100	3	54.02	14.44
57	21- 3	6-10	122.1	21	3.5	10-40	36	11	14	55.33	30	.150	3	59.54	14.67
58	21- 6	7- 8	139.0	21	4.5	10-40	38	12	14	55.78	30	.125	3	57.47	14.67
59	21-10	8- 5	156.0	21	5.5	10-40	40	13	14	56.22	31	.175	3	62.99	14.89
60	22- 1	9- 3	173.3	21	6.5	10-40	42	14	14	56.67	31	.150	3	60.92	14.89
61	21- 7	4-11	83.8	23	.5	8-32	32	9	14	50.89	30	.125	3	57.47	14.67
62	21-10	5- 8	101.0	23	1.5	8-32	34	10	14	53.56	31	.175	3	62.99	14.89
63	22- 1	6- 6	118.4	23	2.5	8-32	36	11	14	56.22	31	.150	3	60.92	14.89
64	22- 3	7- 3	135.9	23	3.5	8-32	38	12	14	56.67	31	.150	4	65.05	14.89
65	22- 6	8- 1	153.7	23	4.5	8-32	40	13	14	57.11	32	.200	4	71.95	15.11
66	22- 9	8-10	171.6	23	5.5	8-32	42	14	14	57.56	32	.175	4	69.19	15.11
67	23- 0	9- 8	189.8	23	6.5	8-32	44	15	14	58.00	32	.150	4	66.43	15.11
68	22- 9	5- 4	95.5	25	.5	6-25	34	10	14	52.22	32	.175	4	69.19	15.11
69	23- 0	6- 1	113.7	25	1.5	6-25	36	11	14	54.89	32	.150	4	66.43	15.11
70	23- 2	6-11	132.1	25	2.5	6-25	38	12	14	57.56	33	.225	4	76.09	15.33
71	23- 4	7- 8	150.6	25	3.5	6-25	40	13	14	58.00	33	.200	4	73.33	15.33
72	23- 6	8- 6	169.3	25	4.5	6-25	42	14	14	58.44	33	.200	4	73.33	15.33
73	23- 8	9- 3	188.1	25	5.5	6-25	44	15	14	58.89	33	.175	4	70.57	15.33
74	23-10	10- 1	207.0	25	6.5	6-25	46	16	14	59.33	34	.250	4	80.22	15.56
75	24- 0	5- 9	108.2	27	.5	4-18	36	10	16	53.56	34	.225	4	77.46	15.56
76	24- 1	6- 6	127.5	27	1.5	4-18	38	11	16	56.22	34	.225	4	77.46	15.56
77	24- 3	7- 4	146.8	27	2.5	4-18	40	12	16	58.89	34	.200	4	74.71	15.56
78	24- 4	8- 2	166.2	27	3.5	4-18	42	13	16	59.33	34	.200	4	74.71	15.56
79	24- 5	8-11	185.7	27	4.5	4-18	44	14	16	59.78	34	.200	4	74.71	15.56
80	24- 7	9- 9	205.3	27	5.5	4-18	46	15	16	60.22	35	.300	4	87.12	15.78
81	24- 8	10- 6	225.0	27	6.5	4-18	48	16	16	60.67	35	.250	4	81.60	15.78
82	25- 2	6- 2	122.0	29	.5	2-11	38	11	16	54.89	35	.200	4	76.09	15.78
83	25- 2	7- 0	142.2	29	1.5	2-11	40	12	16	57.56	35	.200	4	76.09	15.78
84	25- 3	7- 9	162.4	29	2.5	2-11	42	13	16	60.22	36	.300	4	88.50	16.00
85	25- 4	8- 7	182.6	29	3.5	2-11	44	14	16	60.67	36	.300	4	88.50	16.00
86	25- 4	9- 5	202.9	29	4.5	2-11	46	15	16	61.11	36	.300	4	88.50	16.00
87	25- 5	10- 2	223.3	29	5.5	2-11	48	16	16	61.56	36	.300	4	88.50	16.00

NOTES:

1) "N" = 9.62"
2) All shells have Type IV ribs outside only. Both haunch and crown ribs are 18" on centers for structures 40 through 50 and 9" on centers for structures 51 through 87.
3) Weights per foot listed do not include bolt weight.
4) Weight per foot of full invert includes 3½ x 3 x ¼ connecting angle and scalloped closure plate for each side. Inverts for 20 N width and greater are two piece.
5) Full invert plates are 100" thick. When reactions to invert require additional thickness, supplemental plates of thickness and width listed are furnished to bolt between full invert and side connecting angles. When thickness listed is greater than a .250" supplemental plates will be two pieces equalling the composite thickness required.
6) Weight per foot of footing pads includes 3½ x 3 x ¼ connecting angle for each side. Optional wale beam weight is not included.
7) Width of footing pads is for each side. When thickness listed is greater than .250" the footing pads will be two pieces equalling the composite thickness required.

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS



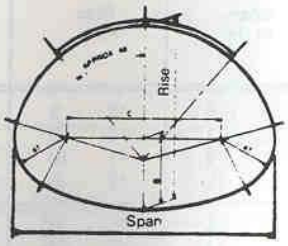
Span Ft.-in	Rise Ft-in.	Area ft²	Required N			Inside Radius	
			Crown or Invert	Haunch	Total	Crown & Invert in.	Haunch in.
19 4	12 9	191	22	10	64	150.3	53.9
20 1	13 0	202	23	10	66	157.2	53.9
20 2	11 10	183	24	8	64	164.1	42.8
20 10	12 2	193	25	8	66	171.0	42.8
21 0	15 1	248	23	13	72	157.2	70.4
21 11	13 1	220	26	9	70	177.9	48.4
22 6	15 8	274	25	13	76	171.0	70.4
23 0	14 1	249	27	10	74	184.8	53.9
23 3	15 11	288	26	13	78	177.9	70.4
24 4	16 11	320	27	14	82	184.8	75.9
24 6	14 7	274	29	10	78	198.6	53.9
25 3	14 11	287	30	10	80	205.4	53.9
25 6	16 9	330	29	13	84	198.6	70.4
26 1	18 2	369	29	15	88	198.6	81.4
26 3	15 10	320	31	11	84	212.3	59.4
27 0	16 2	334	32	11	86	219.2	59.4
27 2	19 1	405	30	16	92	205.4	86.9
27 11	19 5	421	31	16	94	212.3	86.9
28 1	17 1	369	33	12	90	226.1	64.9
28 10	17 4	384	34	12	92	233.0	64.9
29 5	19 11	455	33	16	98	226.1	86.9
30 2	20 2	472	34	16	100	233.0	86.9
30 4	17 11	415	36	12	96	246.8	64.9
31 2	21 2	513	35	17	104	239.9	92.5
31 4	18 11	454	37	13	100	253.7	70.4
32 1	19 2	471	38	13	102	260.6	70.4
32 3	22 2	555	36	18	108	246.8	98.0
33 0	22 5	574	37	18	110	253.7	98.0
33 2	20 1	513	39	14	106	267.5	75.9
34 1	23 4	619	38	19	114	260.6	103.5
34 8	20 8	548	41	14	110	281.2	75.9
35 0	21 4	574	41	15	112	281.2	81.4
35 2	24 4	666	39	20	118	267.5	109.0
35 10	25 9	719	39	22	122	267.5	120.0
36 1	22 4	620	42	16	116	288.1	86.9
36 11	25 7	736	41	21	124	281.2	114.5
37 2	22 2	632	44	15	118	301.9	81.4
38 0	26 7	786	42	22	128	288.1	120.0
38 8	28 0	844	42	24	132	288.1	131.0
40 1	29 8	928	43	26	138	295.0	142.1

SOURCE: ALUMINUM ASSOCIATION

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Geometric Data—Pipe Arch

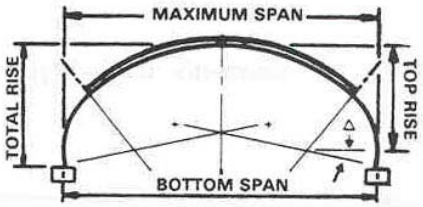


Span Ft-in	Rise Ft-in.	Area ft²	Required N				Inside Radius		B	C
			Total	Crown	Invert	Haunch	Crown in.	Invert in.		
20 1	13 11	216	68	34	20	7	122.7	224.2	62.9	146.7
20 7	14 3	229	70	36	20	7	124.9	256.4	61.4	152.8
21 5	14 7	241	72	36	22	7	131.7	237.3	65.4	163.4
21 11	14 11	254	74	38	22	7	133.7	268.8	63.8	169.4
22 8	15 3	267	76	39	23	7	138.2	274.9	65.0	177.8
23 4	15 7	281	78	40	24	7	142.7	281.0	66.3	186.1
24 3	15 10	295	80	40	26	7	150.0	262.8	70.8	196.8
24 9	16 3	309	82	42	26	7	151.7	293.0	68.9	202.9
25 5	16 7	324	84	43	27	7	156.2	299.0	70.2	211.3
26 4	16 10	339	86	43	29	7	163.9	281.3	75.0	222.1
27 0	17 2	354	88	44	30	7	168.6	287.4	76.4	230.5
27 9	17 6	369	90	45	31	7	173.3	293.5	77.9	238.9
28 5	17 10	385	92	46	32	7	178.0	299.6	79.3	247.3
29 4	18 2	401	94	46	34	7	186.6	286.7	84.6	257.9
29 10	18 6	418	96	48	34	7	187.5	311.6	82.3	264.2
30 4	18 10	435	98	50	34	7	188.6	340.1	80.0	270.2

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Geometric Data—Low Profile Arch



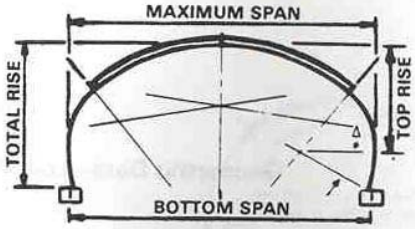
Max. Span Ft-in.	Total Rise Ft-in	Area ft²	Bottom Span Ft-in.	Top Rise Ft-in.	Required N			Inside Radius		Δ
					Crown	Side	Total	Crown In.	Side In.	Deg. Min.
20 1	7 6	120	19 10	6 6	23	6	35	157.2	54.0	12 19
19 5	6 9	105	19 2	5 10	23	5	33	157.2	43.0	15 22
21 6	7 9	133	21 4	6 9	25	6	37	171.0	54.0	12 19
22 3	7 11	140	22 1	6 11	26	6	38	177.9	54.0	12 19
23 0	8 0	147	22 10	7 1	27	6	39	184.8	54.0	12 19
23 9	8 2	154	23 6	7 2	28	6	40	191.7	54.0	12 19
24 6	8 3	161	24 3	7 4	29	6	41	198.6	54.0	12 19
25 3	8 5	168	25 0	7 5	30	6	42	205.4	54.0	12 19
26 0	8 7	175	25 9	7 7	31	6	43	212.3	54.0	12 19
27 3	10 0	217	27 1	9 0	31	8	47	212.3	76.0	8 51
28 1	9 6	212	27 11	8 7	33	7	47	226.1	65.0	10 17
28 9	10 3	234	28 7	9 3	33	8	49	226.1	76.0	8 52
28 10	9 8	220	28 8	8 8	34	7	48	233.0	65.0	10 17
30 4	9 11	237	30 2	9 0	36	7	50	246.8	65.0	10 17
31 0	10 8	261	30 10	9 8	36	8	52	246.8	76.0	8 52
31 7	12 1	309	31 2	10 4	36	10	56	246.8	87.0	14 0
31 1	10 1	246	30 10	9 1	37	7	51	253.7	65.0	10 17
32 4	12 3	319	31 11	10 6	37	10	57	253.7	87.0	14 0
31 9	10 2	255	31 7	9 3	38	7	52	260.6	65.0	10 17
33 1	12 5	330	32 8	10 8	38	10	58	260.6	87.0	14 0
33 2	11 0	289	33 0	10 1	39	8	55	267.5	76.0	8 52
34 6	13 3	367	34 1	11 6	39	11	61	267.5	98.0	12 26
34 8	11 4	308	34 6	10 4	41	8	57	281.2	76.0	8 52
37 11	15 7	478	37 8	13 10	41	14	69	281.2	131.0	9 23
35 5	11 5	318	35 3	10 6	42	8	58	288.1	76.0	8 52
38 8	15 9	491	38 4	14 0	42	14	70	288.1	131.0	9 23

See "Notes" Table 5-20A or 5-20B for rib spacing when required.

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Geometric Data—High Profile Arch



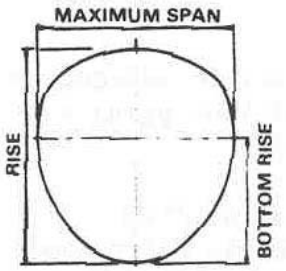
Max Span Ft.-in.	Total Rise Ft.-in.	Area ft²	Bottom Span Ft-in.	Top Rise Ft-in.	Required N				Inside Radius			Δ	
					Crown	Haunch	Side	Total	Crown in.	Haunch in.	Side in.	Deg.	Min.
20 1	9 1	152	19 6	6 6	23	5	3	39	157.2	54.0	157.2	11	40
20 9	12 1	214	18 10	7 3	23	6	6	47	157.2	65.0	157.2	22	8
21 6	11 8	215	19 10	6 9	25	5	6	47	171.0	54.0	171.0	20	20
22 10	14 6	284	19 10	8 6	25	7	8	55	171.0	76.0	171.0	26	48
22 3	11 9	224	20 7	6 11	26	5	6	48	177.9	54.0	177.9	19	33
22 11	14 0	275	20 1	7 7	26	6	8	54	177.9	65.0	177.9	25	44
23 0	11 11	234	21 5	7 1	27	5	6	49	184.8	54.0	184.8	18	49
24 4	14 10	309	21 7	8 5	27	7	8	57	184.8	76.0	184.8	24	50
23 9	12 1	244	22 2	7 2	28	5	6	50	191.7	54.0	191.7	18	8
24 6	13 8	288	21 11	7 4	29	5	8	55	198.6	54.0	198.6	23	2
25 10	15 1	334	23 3	8 9	29	7	8	59	198.6	76.0	198.6	23	6
25 3	13 1	283	23 3	7 5	30	5	7	54	205.4	54.0	205.4	19	35
26 6	15 3	347	24 0	8 10	30	7	8	60	205.4	76.0	205.4	22	19
26 0	13 3	294	24 1	7 7	31	5	7	55	212.3	54.0	212.3	18	57
27 3	15 5	360	24 10	9 0	31	7	8	61	212.3	76.0	212.3	21	36
27 5	13 6	317	25 8	7 10	33	5	7	57	226.1	54.0	226.1	17	48
29 5	16 5	412	27 1	10 0	33	8	8	65	226.1	87.0	226.1	20	18
28 2	14 5	348	25 11	8 0	34	5	8	60	233.0	54.0	233.0	19	37
30 2	18 0	466	26 8	10 2	34	8	10	70	233.0	88.0	233.0	23	51
30 4	15 5	399	28 2	9 0	36	6	8	64	246.8	65.0	246.8	18	34
31 7	18 4	497	28 5	10 4	36	8	10	72	246.8	87.0	246.8	23	3
31 1	15 7	412	29 0	9 1	37	6	8	65	253.7	65.0	253.7	18	3
31 8	17 9	483	28 7	9 10	37	7	10	71	253.7	76.0	253.7	22	25
32 4	19 11	554	27 11	10 6	37	8	12	77	253.7	87.0	253.7	26	45
31 9	17 2	469	28 9	9 3	38	6	10	70	260.6	65.0	260.6	21	47
33 1	20 1	571	28 9	10 8	38	8	12	78	260.6	87.0	260.6	26	3
32 6	17 4	484	29 6	9 4	39	6	10	71	267.5	65.0	267.5	21	14
33 10	20 3	588	29 7	10 9	39	8	12	79	267.5	87.0	267.5	25	23
34 0	17 8	514	31 2	9 8	41	6	10	73	281.2	65.0	281.2	20	11
34 8	19 10	591	30 7	10 4	41	7	12	79	281.2	76.0	281.2	24	7
35 4	21 3	645	30 7	11 0	41	8	13	83	281.2	87.0	281.2	26	6
37 3	23 4	747	32 7	13 2	41	11	13	89	281.2	120.0	281.2	26	8
34 9	17 9	529	31 11	9 9	42	6	10	74	288.1	65.0	288.1	19	42
35 5	20 0	608	31 5	10 6	42	7	12	80	288.1	76.0	288.1	23	33
36 1	21 5	663	31 5	11 2	42	8	13	84	288.1	87.0	288.1	25	28
38 0	23 6	767	33 5	13 3	42	11	13	90	288.1	120.0	288.1	25	31

See "Notes" Table 5-20A or 5-20B for rib spacing when required.

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Geometric Data—Pear Shape



Max. Span Ft.-in.	Rise Ft.-in.	Rise Bottom Ft-in.	Area ft²	Required N					Inside Radius			
				Top	Corner	Side	Bottom	Total	Bottom in.	Side in.	Corner in.	Top in.
23 7	25 6	14 10	477	25	5	24	15	98	108.31	198.07	74.07	175.07
24 0	25 10	15 1	497	22	7	22	20	100	119.07	208.07	84.07	194.07
25 4	25 11	15 10	518	27	7	20	20	102	124.23	218.24	84.24	191.24
24 10	27 7	16 9	545	27	5	25	18	105	110.90	236.21	69.21	191.21
28 10	27 3	19 8	590	32	7	27	8	110	79.61	257.96	68.96	252.96
26 8	28 3	18 0	594	28	5	30	12	110	95.45	241.24	57.24	251.24
28 0	27 10	16 9	624	27	8	22	25	112	146.38	227.72	86.72	244.72
28 7	30 7	19 7	690	32	7	24	24	118	133.13	288.45	84.45	218.45
30 0	29 7	20 0	699	32	8	23	25	119	142.41	288.26	79.26	262.26
30 0	31 2	19 11	739	34	7	24	26	122	144.43	288.58	84.58	231.58

Figure 12.4.45 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

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Section 13
Advanced Inspection Techniques

Topic 13.1 Timber

13.1.1 Introduction

At the present time, most advanced inspection techniques are used to supplement visual inspections. Advanced inspection techniques give inspectors the ability to further evaluate defects found during a visual inspection. They can also be used to perform inspections on members that are not accessible. Advanced inspection techniques usually require a professionally trained technician to perform the testing and a professional that has expertise in interpreting the advanced inspection results.

There are two main classifications of advanced inspection techniques. The first is labeled nondestructive testing. This classification pertains to all advanced inspection techniques that do not impair the usefulness of the member being tested. Destructive testing, the second main classification, covers all advanced inspection techniques that affect or destroy the structural integrity of the member being tested.

This Topic describes the different types of nondestructive and destructive test methods for timber bridge members and the general procedures for each.

13.1.2 Nondestructive Testing Methods

Pol-Tek

Pol-Tek is a sonic testing device that is used to detect rot or other low density regions in timber poles. Starting about six inches below the ground line, probes are pressed on opposite sides of the timber member. A trigger trips a hammer that sends a sound wave down one probe, through the member, and up the other probe to a dial (see Figure 13.1.1).

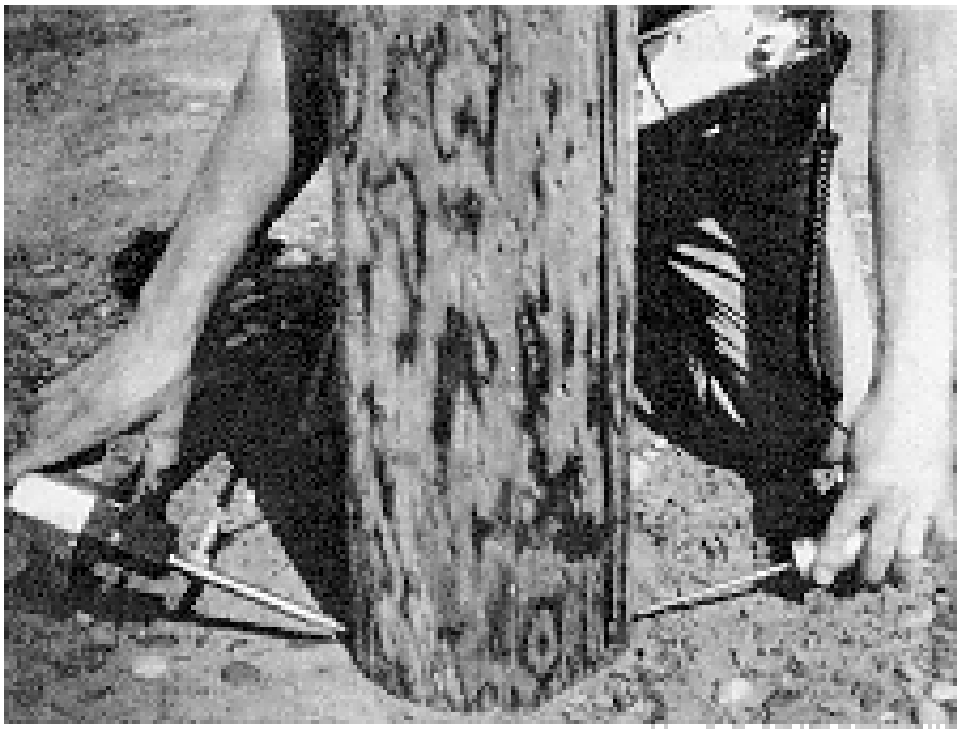


Figure 13.1.1 Pol-Tek Sonic Testing Apparatus

This method eliminates the need for making holes in good members. Members testing positive for rot are then drilled or cored to determine the nature of the defect. A dial reading that is low, compared with that of a good member of similar diameter, indicates rot or another low density region that delayed the sound wave within the member. However, several readings should be taken on the member since the readings are nearly instantaneous, and the Pol-Tek should be checked frequently for proper calibration.

Used by trained personnel, Pol-Tek works well with Douglas fir and western red cedar. However, it does not work as well with southern pine members because of the high incidence of ring shakes.

Spectral Analysis

Spectral Analysis, sometimes called stress wave, uses sonic waves to produce stress waves in a timber member. The stress waves are then used to locate decay in timber members. The stress waves travel through the timber member and reflect off the timber surface, any flaws, or joints between adjacent members at the speed of sound. It is known that stress waves travel slower in decayed members than in sound members. If the members dimensions are known, the amount of time it takes for a stress wave to travel a certain distance can prove that defects are evident due to longer stress wave timings.

Stress waves are also used to determine the in-situ strength of timber members. Sound timber members transmit waves at higher velocity than decayed wood. The velocity of the stress wave can be calculated by obtaining time of flight readings over a set length. The velocity can be converted into a dynamic modulus of elasticity, which in turn, allows professionals to estimate the strength properties of

the wood.

Ultrasonic Testing

Ultrasonic testing consists of high frequency sound waves introduced by a sending transducer. Discontinuities in the specimen interrupt the sound wave and deflect it toward a receiving transducer. The magnitude of the return signal allows a measurement of the flaw size. The distance to the flaw can be estimated from the known properties of the sound wave and of the material being tested. Ultrasonic testing can be used to detect cracks, internal flaws, discontinuities, and surface damage (see Figure 13.1.2).



Figure 13.1.2 Ultrasonic Testing Equipment

In timber bridge members, ultrasonic testing can be used to determine the in-place strength of timber bridge members, both above and below water. The load carrying capacity of the member is correlated to the member's wave velocity normal to the grain and to its in-place unit weight.

Vibration

A newer type of nondestructive testing that can determine the condition of timber bridge members deals with the use of vibrations. This nondestructive testing method is based on the philosophy that all sound timber members vibrate at a certain frequency. While testing a timber member, if the member vibrates at a different frequency than the established theoretical frequency, the member may have defects present. Vibratory testing methods in timber members are basically used to determine the members modulus of elasticity. From this, other properties of the timber member can be established.

13.1.3 Other Testing Methods

Boring or Drilling

Boring is the most dependable and widely used method for detecting internal decay in timber. Boring permits direct examination of an actual sample from a

questionable member. An increment borer is used to extract wood cores for examination (see Figure 13.1.3).



Figure 13.1.3 Increment Borer

Drilling is performed using a rechargeable drill or a brace and bit. An abrupt decrease in drilling resistance indicates either rot or a void. However, wet wood and natural voids can falsely suggest rot. While samples are generally not attainable, observation of the wood particles removed during the drilling process can provide valuable information about the member. The depth of preservative penetration, if any, can be determined, and regions of discolored wood may indicate rot.

A newer drilling technique is the use of a decay detection device. It operates upon the principle that a drill moving through sound wood will encounter more resistance that a drill moving through decayed, and/or soft wood. It records the resistance, using a pen, paper and rotary drum arrangement, so that a permanent graphic record of the test is generated. Sound wood produces a series of near vertical markings on the record, however, when decayed wood is encountered, the resistance drops and the markings assume a more horizontal or diagonal pattern. By studying the resulting record, an experienced operator can determine if decay exists and, because the record is marked in millimeters (mm) of penetration, can estimate the approximate location and size of the decayed area.



Figure 13.1.4 Decay Detection Device

The use of increment cores for assessing the presence and damage due to bacterial and fungal decay requires special care. Cleaning of the increment borer is necessary after each core extraction to eliminate transfer of organisms; trichloroethane has been found to work well. Core samples that do not show visible signs of decay can be cultured to detect the presence of potential decay hazards. Many laboratories can provide this service. Core samples are more commonly used to detect the presence of internal decay pockets and to measure the depth of preservative penetration and retention.

All bore holes can provide an entrance for bacterial and fungal decay to gain access to the member. As such, the holes must be treated with a preservative and must be plugged.

Moisture Content

Moisture meters can be used to determine moisture content in a timber member. Moisture contents exceeding 20% indicate the condition of the wood is conducive to decay. As a sliding hammer drives two electrodes into the wood, a ruler emerging from the top of the hammer measures the depth. These electrodes can measure moisture content to a depth of about 2 1/2 inches. Because the high moisture content of decaying wood causes steeper than normal moisture gradients, the meter is useful for determining the extent of decay.

Probing

Probing consists of inserting a pointed tool, such as an ice pick, into the wood and comparing its resistance with that of sound wood. Lack of resistance or excessive softness to probe penetration may reveal the presence of decay.

Two forms of probing are a pick test and a shell-thickness indicator. A pick test consists of removing a small piece of wood with a pick or pocketknife. If the wood splinters, it is probably sound wood, and if it breaks abruptly, it is probably

decayed wood.

A shell-thickness indicator is a thin, metal, hooked rod used to determine the thickness of solid, but not necessarily sound, wood. The rod is inserted into a hole made by coring or drilling and is then pulled back with pressure against the side of the hole. The hook should attach to the edge of a rot pocket, making it possible to determine the depth of the rot and the solid wood.

Shigometer

The Shigometer measures electrical resistance to detect rot in timber members. It should be used in wood with a moisture content of at least 27%, a value indicative of decaying wood. A probe is used consisting of two twisted, insulated wires with the insulation removed near the tip. This probe is inserted to various depths into a hole 3/32 inch in diameter. If the electrical resistance changes as the probe goes deeper, this indicates rot or a defect.

While this device effectively detects rot, it can also produce misleading readings on sound timber. Consequently, drilling or coring should be done on suspect members. Like the Pol-Tek, the Shigometer should be recalibrated frequently.

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Topic 13.2 Concrete

13.2.1 Introduction

Visual inspections are the most common types of inspections when dealing with bridge members. These inspections don't require sophisticated, highly technical apparatus to perform the inspection, and therefore, are the most economical. For concrete bridge members, a visual inspection can only report on the surface conditions based partly on the opinion of the inspector. Due to this fact, advanced inspection techniques are sometimes required to evaluate the underlying condition of concrete members. This topic describes and explains the common types of nondestructive and destructive testing that is available to evaluate concrete members.

13.2.2 Nondestructive Testing

Acoustic Wave Sonic/Ultrasonic Velocity Measurements

A full evaluation of concrete decks can be accomplished with sonic~ultrasonic acoustic wave velocity measurements. This method delineates areas of internal cracking (including delaminations) and deteriorated concrete, including the quantification of strength characteristics (elastic moduli values). A mobile automated data acquisition device with an impact energy source and multiple sensors is the principle part of a computer-based monitoring and recording system for detailed evaluation of bridge decks. Bridge abutments and concrete support members are tested using the same recording system with a portable, hand-held sensor array (see Figure 13.2.1 and 13.2.2). The system works directly on either bare concrete or through wearing surfaces such as asphalt. It can distinguish between debonded asphalt and delaminations, and it is effective for a detailed evaluation of large areas.



Figure 13.2.1 Portable Hand Held Sonic/Ultrasonic Testing Sensor Array System (http://www.ndtcorporation.com)

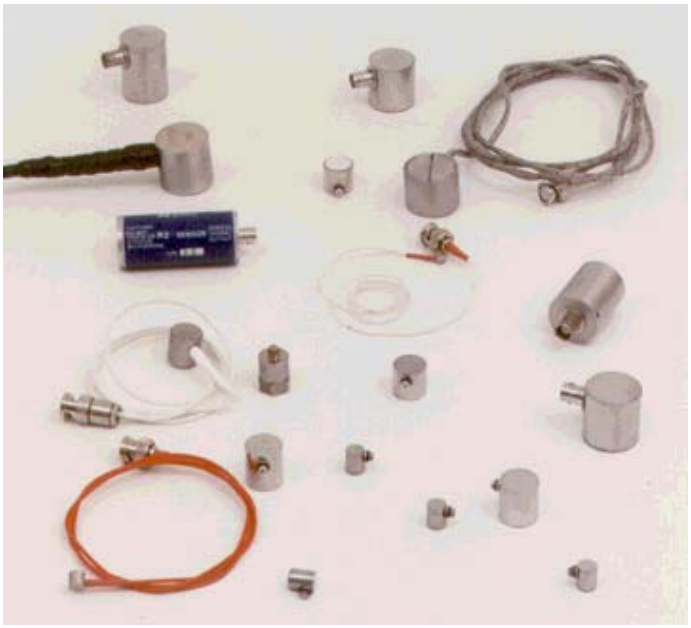


Figure 13.2.2 Acoustic Emission Sensors

Delamination Detection Machinery

Delamination detection machinery is based on sonic responses and can be used to inspect concrete decks (see Figure 13.2.3). The portable electronic instrument consists of three components: a tapping device, a sonic receiver, and a signal interpreter. The instrument is moved across the deck as acoustic signals are passed through the deck. These signals are then received and electronically interpreted, and the output is used to generate a plan of the deck showing delaminated areas. This method can be used on asphalt covered surfaces, although accuracy decreases.



Figure 13.2.3 Delamination Detection Machinery

Electrical Methods

Half Cell Potentials are used to evaluate the corrosion activity of reinforcing steel embedded in concrete. Commonly known as CSE (Copper Sulfate Electrode) tests, reinforcing bar networks are physically accessed and wired for current detection. Half cell electrical potentials of reinforcing steel are measured by moving the CSE about the concrete surface. As the CSE contacts concrete over an actively corroding rebar, voltage is registered. Measured potential values reflect levels of corrosion activity in the rebar. Higher potential measurements indicate corrosion activity. This kind of survey can be used to determine core sample locations.

Electromagnetic Methods

Advancements in ground penetrating radar have lead to the development of the High Speed Electromagnetic Roadway Measurement and Evaluation System (HERMES) Bridge Inspector. This system was built by the Lawrence Livermore National Library to detect delaminations in concrete decks caused by reinforcement corrosion. The HERMES Bridge Inspector sends high frequency electromagnetic pulses from 64 radar antennas into a bridge deck while travelling over the structure. The device is setup in a trailer mounted towing vehicle and is made up of a computer workstation, storage device, survey wheel, control electronics, and the 64 antenna modules or transceivers (see Figure 13.2.3). The system can inspect up to a 1.9 m (6'-3") width at a time with maximum speeds of

up to 60 mph. At speeds of around 20 mph, the system can sample the concrete deck every 1.5 cm (9/16") in the direction of travel. Output information can be reconstructed to show cross-sections of the deck being inspected. The depth of penetration depends on time and the material type. A 30 cm (11-13/16") penetration in concrete can be accomplished in about 6 nanoseconds. In the near future, a new system, called HERMES II, will update the original HERMES Bridge Inspector based on experience gained from the original.





Figure 13.2.4 The HERMES Bridge Inspector

Pulse Velocity

Pulse Velocity techniques are used to evaluate relative quality of concrete and estimate compressive strength. The pulses pass through the concrete and the transit time is then measured. The pulse velocity is then interpreted to evaluate the quality of the concrete and to estimate in-place concrete compressive strength.

This equipment analyzes concrete in decks by measuring velocity of sound waves. Some equipment generates sound waves by shooting BB's onto the deck. The time for the waves to return depends on the integrity of the concrete.

Flat Jack Testing

The flat jack method was originally developed to test the in situ stress and deformation of rock and is now being applied to masonry structures. A portion of the horizontal mortar joint is removed, and the flat jack (an envelope made of metal) is inserted and pressurized to determine the state of stress. For deformation testing, two flat jacks are inserted, one directly above the other and separated by five or six courses.

Ground-Penetrating Radar

Ground-penetrating radar is used to detect deterioration of bridge decks. This technique uses low-power, high-frequency pulsed radar. An important benefit of this method is the ability to measure the thickness of asphalt covering. It can also be used to examine the condition of the top flange of box beams that may otherwise be inaccessible.

Impact-Echo Testing

The impact-echo technique has proven to be successful in detecting flaws in slabs and pavements and is now being applied to assess the condition of concrete beams and columns. Testing involves introducing a stress pulse into the concrete by mechanical impact. The pulse in the concrete is reflected by cracks, voids, and the boundaries of the structure. A transducer placed near the impact point monitors surface displacements caused by the reflections. The response can then be interpreted to detect flaws within the concrete (see Figure 13.2.4).

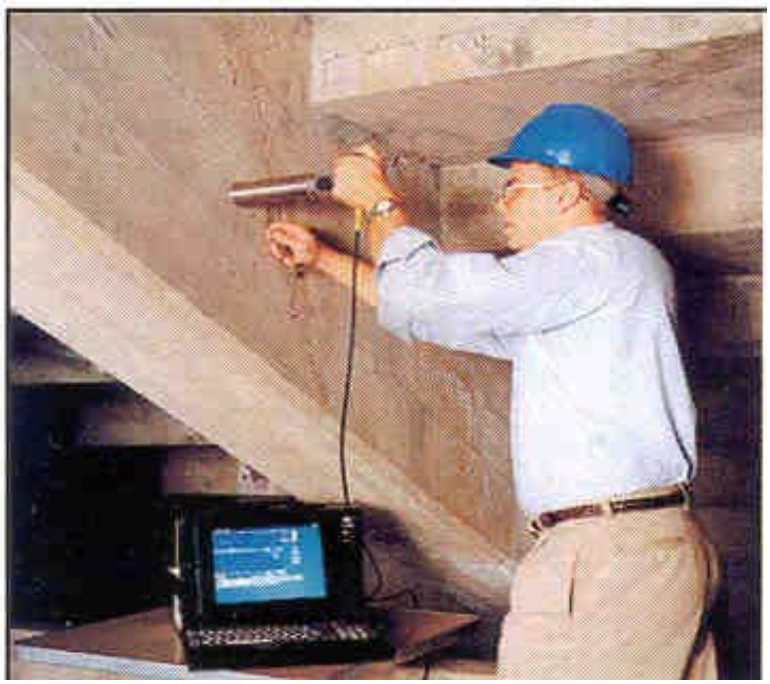


Figure 13.2.5 Impact-Echo Testing Equipment

Infrared Thermography

Infrared thermography is also used to detect deterioration of bridge decks. This technique uses an infrared camera to detect temperature differentials in a concrete surface. A "cold spot" indicates a delamination. Although subject to weather conditions, this technique is efficient for large surfaces.



Figure 13.2.6 Infrared Thermography Testing Equipment

Laser Ultrasonic Testing

Laser ultrasonic testing provides information about flaws in concrete and about the position of steel reinforcement bars, which cannot be obtained with the non-laser ultrasonic testing described in this Section. Laser-generated acoustic wave measurements with high stress amplitudes provide information about the quality of the concrete at various depths from the surface. Reinforcing steel does not cause misleading results in laser ultrasonic testing as it does in non-laser ultrasonic testing.

Magnetic Field Disturbance

Advanced inspection techniques have been developed that can evaluate fatigue damage to steel reinforcement in concrete members. The device is known as the magnetic field disturbance (MFD) system and can be used on reinforced and prestressed concrete. The system maps the magnetic field across the bottom and sides of the beam. A discontinuity in magnetized steel, such as a fracture in a rebar or a broken wire in a steel strand, produces a unique magnetic signal. While the research has been encouraging for detecting fatigue-related damage due to the significantly different magnetic signals for corroded reinforcing, MFD has not yet been demonstrated for detecting in-service corrosion damage.

Neutron Probe for Detection of Chlorides

A neutron probe can be used to detect chlorides in construction materials. The materials are bombarded with neutrons from a small portable source. Measuring the gamma rays bouncing back provides a spectrum showing different elements, one of which is chloride. A major potential application that remains to be tested is measuring chlorides in reinforced concrete to determine corrosion hazard. Another potential application includes inspecting suspension bridge cables.

Nuclear Methods

The primary use of nuclear methods is to measure the moisture content in concrete by neutron absorption and scattering techniques. These moisture measurements are then used to determine if corrosion of reinforcement is likely to occur. A more direct measurement of the rate of corrosion would be more useful to the bridge inspector, and this method is therefore more research oriented than operational

Pachometer

A pachometer is a magnetic device used in determining the position of reinforcement. Magnetic methods do not detect concrete defects or deterioration directly. However, they can detect regions of inadequate cover, which is often associated with corrosion-induced deterioration. Magnetic methods can be used to measure cover in the range of 0 to 3 inches to an accuracy of about 1/4 inch

Rebound and Penetration Methods

Rebound and penetration methods measure the hardness of concrete and can be used to predict the strength of concrete. The Schmidt hammer (also known as the Swiss hammer) is probably the most commonly used device to measure the penetration resistance of hardened concrete. A spring-loaded device strikes the surface of the concrete, and based on the response, the compressive strength of the concrete can be determined. This inspection technique can be used to compare the quality of the concrete in different parts of concrete bridge components. However, only the surface of the concrete is being tested, and the strength value is relative.

Another common penetration device is called the Windsor probe. A pistol-like driving device fires a probe into the surface of the concrete. The probe is specifically designed to crack aggregate particles and to compress the concrete being tested.

Both of these tests are considered practical primarily with concrete that is less than one year old. However, when used in conjunction with core sampling, these tests can also be used to determine significant differences in concrete strength of older bridges.

Ultrasonic Testing

Ultrasonic testing can provide valuable information regarding the condition of concrete bridge members. However, the method can be difficult to use with reinforced concrete members, and some skill is required to obtain usable results.

Large cracks and voids can be detected, since the path of the pulse will travel around any cavity in the concrete and time of transmission is therefore lengthened. The presence of steel parallel to the line of transmission provides a path along which the pulse can travel more rapidly, causing misleading results. Therefore, it is generally desirable to choose paths that avoid the influence of reinforcing steel. Refer to Topic 13.1 for further details about the principles of ultrasonic testing.

13.2.3 Other Testing

Core sampling is a destructive form of concrete inspection, and it can weaken a member. Cores can be used for many of the following destructive tests. Usable cores can normally be obtained only if the concrete is relatively sound. If possible, cores should have a diameter three times the maximum aggregate size. All core holes should be filled with non-shrink concrete grout.

Carbonation

Carbonation of concrete is the result of the reaction of carbon dioxide and other acidic gases in the air, and it can cause a loss of protection of the reinforcing steel against corrosion. The depth of carbonation in a concrete bridge member can be measured by exposing concrete samples to a solution. Uncarbonated concrete areas change color, while carbonated concrete areas remain colorless.

Concrete Permeability

Air and water permeability can be measured by drilling a small hole into the concrete, sealing the top with liquid rubber, and inserting a hypodermic needle. Air permeability can then be determined by filling the hole with water and measuring the flow into the concrete at a pressure similar to that of rainfall. This method is seldom used in bridge inspections.

Concrete Strength

Actual concrete strength and quality can be determined only by removing a concrete core and performing such laboratory tests as:

- Compressive strength
- Cement content
- Air voids
- Static modulus of elasticity
- Dynamic modulus of elasticity
- Splitting tensile strength

Endoscopes and Videoscopes

Endoscopes and videoscopes are viewing tubes that can be inserted into holes drilled into a concrete bridge member (see Figure 13.2.7). Light can be provided by glass fibers from an external source. Some applications of this method include the inspection of the inside of a box girder and the inspection of hollow posttensioning ducts. Although this is a viewing method, it is considered to be a destructive method because some destruction is necessary for its proper use in concrete.



Figure 13.2.7 Remote Video Inspection Device

Moisture Content

Moisture content in concrete serves as an indicator of corrosion activity. Moisture content can be determined using nuclear methods (refer to Topic 13.2.2) or from concrete samples taken from the bridge and oven dried in a laboratory

Petrographic Examination

Laboratory technique for determining various characteristics of hardened concrete, which are useful in determining the existing condition and predicting future performance. Will detect Alkali-Silica Reaction (ASR) products.

Reinforcing Steel Strength

The actual properties of reinforcing steel can only be determined by removing test samples. Such removal of reinforcing steel can be detrimental to the capacity of the bridge and should be done only when such data is essential.

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Topic 13.3 Steel

13.3.1 Introduction

During a visual inspection of steel bridge members, the bridge inspector may come across a crack or area that can only be evaluated with the aid of advanced inspection techniques. Professional technicians typically perform these advanced inspection techniques, but bridge inspectors should have an understanding of the various options available. Many of the advanced inspection techniques used for steel can also be used for other materials. This Topic describes and explains the common types of nondestructive and destructive testing that is available for steel bridge members.

13.3.2 Nondestructive Testing

Acoustic Emissions Testing (American Society of Nondestructive Testing Designation (ASNT) - AE)

Acoustic emissions can be used to identify growing cracks. When cracks grow, they emit minute "sounds" which propagate outward from the source. Sensors placed on the surface of the member "listen" for these sounds. The bridge must be loaded so as to produce stress and cracking in the members when this test is used.

A device known as Local Area Monitoring (LAM) can be used to monitor areas that already are cracked or cracked areas that have been retrofit. The device is a portable, modular eight-channel system that can be mounted close to the area being monitored (see Figure 13.3.1). The system can be directly connected to a computer or it can be accessed through wired or wireless modems for data collection.



Figure 13.3.1 LAM System Showing Eight Sensors and Holding Magnet

Computer Programs

Computer programs have been developed to maximize the value of bridge inspections. In the pre-inspection routine, the inspector enters data on the bridge design and previously detected flaws. The computer responds with a customized checklist for the inspector, flagging critical areas of the structure. In the post-inspection routine, the inspector enters data about the flaws encountered in the field, and the computer responds with information about if the crack is likely to propagate and how to repair the crack. This procedure allows the inspector to detect flaws early and to judge which ones need immediate repair.

Computer Tomography

Computer tomography uses X-ray and gamma radiation to visualize the interior defects of a steel member. The image is captured by a detector array, it is processed by a computer, and it is then reconstructed. This method is similar in many ways to medical CAT scans, and it has great potential for locating discontinuities of all types in steel members (as well as concrete members).

Corrosion Sensors

Corrosion sensors are being developed that use environmental variables such as dirt and duration of wetness to indicate the degree of corrosion of a steel structure.

Dye Penetrant (ASNT Designation - PT)

A dye penetrant can be used to define the extent and size of surface flaws in steel members (see Figure 13.3.2). The test area is cleaned to bare metal, a dye is applied and allowed to penetrate the surface, and excess penetrant is removed. A developer is then applied, which draws the dye out of the irregularities and defines the extent and size of surface flaws. Bridge inspectors commonly use this method since it does not require extensive training or expensive equipment. A limitation of this method, however, is that it reveals neither the depth of cracks nor any subsurface flaws. Another important factor when performing dye penetrant testing is the penetrant dwell time. This is the amount of time that the penetrant is allowed to remain on the surface before the excess is wiped off. Factors that effect the dwell time include:

- Temperature of the member being tested and the penetrant
- Ambient air temperature (higher temperatures require shorter dwell times)
- Humidity (low humidity causes penetrant to dry out rapidly)
- Size and shape of the discontinuity (hairline cracks need more time than large ones)
- Material type
- Penetrant removal type and manufacturer's recommendations

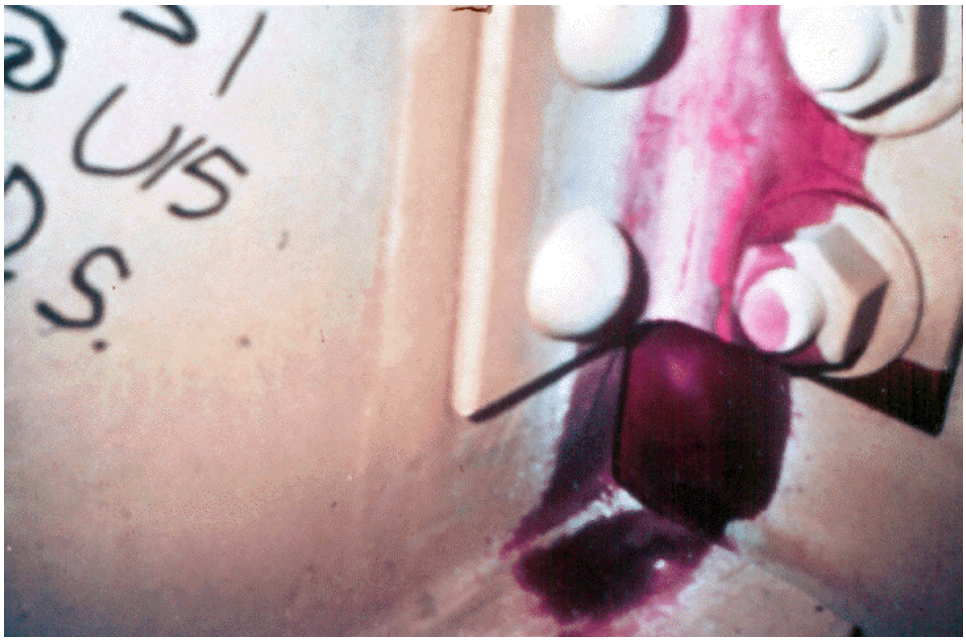


Figure 13.3.2 Detection of a Crack Using Dye Penetrant

Magnetic Particle (ASNT Designation - MT)

Magnetic particle or magnetic flux leakage is useful in detecting surface gouges, cracks, and holes in ferromagnetic materials. It can also detect subsurface defects, such as voids, inclusions, and cracks, which lie near the surface. A magnetic field is induced into the member, and cracks or other irregularities in the surface of the member cause irregularities in the magnetic field (see Figure 13.3.3). This technique is also referred to as magnetic field disturbance.

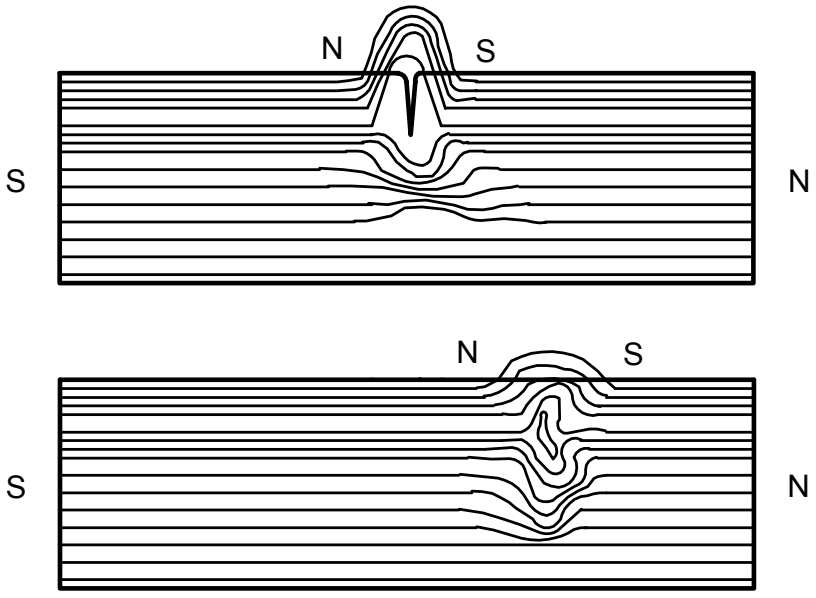


Figure 13.3.3 Schematic of Magnetic Field Disturbance

Radiographic Testing (ASNT Designation - RT)

Radiographic testing is used to detect and locate surface and subsurface cracks, voids, and inclusions. X-rays or gamma rays are passed through the member and are absorbed differently by the various flaws. When a piece of film is exposed to the rays, the defects appear as shadows on the film. This type of advanced inspection is typically used for full penetration groove welds during fabrication and construction.

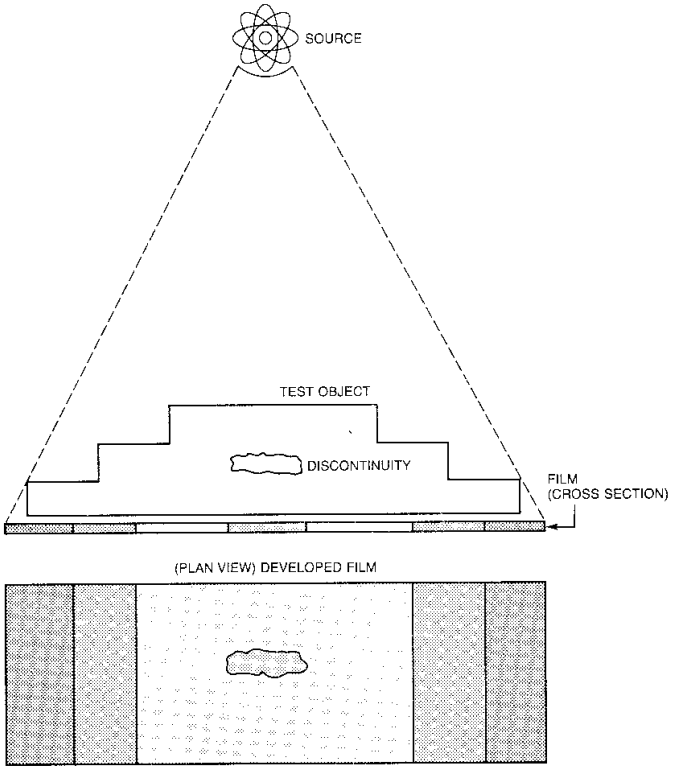


Figure 13.3.4 Radiographic Testing

Robotic Inspection

Several companies are currently developing and marketing a system which uses high-resolution video cameras on robotic arms attached to permanent falsework underneath the bridge. By remote telescanning, details can be visually monitored, with magnification if needed, without the inspector having to climb to gain access to a detail each time an inspection is desired. While the primary material application for robotic inspection is steel, it can also be used on timber and concrete bridges.

In recent years, the California Department of Transportation (Caltrans) has been working on an aerial robotic inspection system (see Figure 13.3.5). This system, in the testing and development stage, can allow bridge inspectors to view elevated bridge members from the ground. It is controlled by a remote control that is connected to the system through a 100 ft (30 m) electrical cord. A fiber optics cable transfers information and images from the aerial device to the ground station. This type of inspection may reduce traffic delays and increase the level of safety for motorists and bridge inspectors.



Figure 13.3.5 Robotic Inspection

Ultrasonic Testing (ASNT Designation - UT)

Ultrasonic testing is frequently used in steel applications and can be used to detect cracks in flat, relatively smooth members, as well as pins (see Figure 13.3.6). It can also be used to measure the thickness of steel members, providing detailed information concerning loss of cross section. Ultrasonic testing also has many applications in the inspection of welds, detecting porosity, voids, inclusions, corrosion, cracks, and other discontinuities. Refer to Topic 13.1 for further details about the principles of ultrasonic testing.



Figure 13.3.6 Ultrasonic Testing of a Pin in a Moveable Bridge

Eddy Current (ASNT Designation - ET)

This type of electromagnetic technique uses AC currents. Eddy current testing can only be performed on conductive materials and is capable of detecting cracks and flaws as well as member dimensions and variations. The system works by monitoring the voltage across a coil that has an AC current flowing through it. When the coil is placed next to the conductive member, the member produces eddy currents that flow opposite to the direction of flow from the coils. Defects in the member disturb the eddy currents, which, in turn, effect the induced current. The effected induced current is monitored through the voltage across the coil. Eddy current testing devices can be hand held devices (see Figure 13.3.7).



Figure 13.3.7 Hand Held Eddy Current Instrument

Information concerning non-destructive testing can be found on the American Society of Non-destructive Testing website: www.asnt.org.

13.3.3 Other Testing

Strength tests are normally considered destructive tests since they usually involve tests conducted on pieces of steel removed from the bridge. Small steel pieces cut out of steel members are called test "coupons." The removal technique and coupon size must be suitable for the planned tests. If a coupon is required, consult the bridge engineer to determine the most suitable area of removal. For instance, an inspector should not remove a coupon from the web area over a bearing. An inspector also should not remove a coupon from the bottom flange at midspan. Destructive tests may be necessary to determine the strength or other properties of existing iron or steel on bridges for which the steel type is unknown.

The following tests can be conducted only by the destructive technique of removing a sample and evaluating it in a laboratory.

Brinell Hardness Test

The Brinell hardness test measures the resistance to penetration of the steel. A hardened steel ball is pressed into the test coupon by a machine-applied load. The applied load and the surface area of the indentation are used to calculate the hardness of the steel. For a steel that has not been hardened by cold work, its hardness is directly related to its ultimate tensile strength.

Charpy Impact Test

An impact test determines the amount of energy required to fracture a specimen. A common impact test for steel coupons is the Charpy V-notch test (see Figure 13.3.8). A notched test coupon is placed in a vise, and a hammer is then released from an elevated position, swinging down and hitting the coupon. Since the force of the hammer is concentrated in a notch in the coupon, the stress goes into fracturing the specimen and not into strain. The energy required for fracture is determined based on the mass of the hammer and the distance that it fell. This test can be performed at different temperatures to determine if the steel is susceptible to brittle failure.

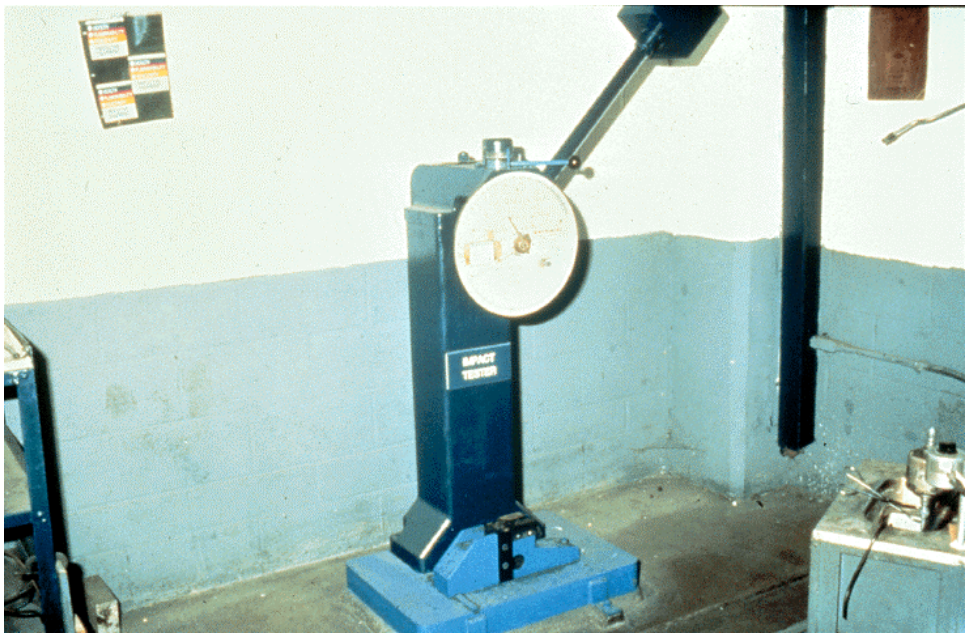


Figure 13.3.8 Charpy V-Notch Test

Chemical Analysis

The chemical composition of the steel is an important indication of whether a weld will crack, either from cold cracking or hot cracking. Tests can be performed on coupons to determine the chemical composition of the steel.

Cold, or delayed, cracking can be approximated using a carbon equivalent (C.E.) equation that is based on the chemical composition of the steel. One such equation, based on the relative proportions of various elements in the steel, is presented in the ASTM A706 rebar specification:

$$C.E. = C\% + \frac{Mn\%}{6} + \frac{Cu\%}{40} + \frac{Ni\%}{20} + \frac{Cr\%}{10} - \frac{Mo\%}{50} - \frac{V\%}{10}$$

When the C.E. is below 0.55, the steel is generally not susceptible to cold cracking, and no special precautions are required for welding. However, when the C.E. is above 0.55, the steel is susceptible to cold cracking, and special precautions are required for welding.

Hot cracking occurs as the weld begins to solidify. Hot cracks have almost been eliminated today due to modern welding material formulation.

Tensile Strength Test

The tensile strength is the highest stress that can be applied to the coupon before it ruptures. Once the yield strength has been exceeded, the coupon begins to elongate or "neck down" and eventually breaks if the load is not removed. The tensile strength of the steel can be easily determined. See Section P.1, Bridge Mechanics.

The ends of the test coupon are placed in vises on a testing machine. The machine then applies a tensile load to the ends of the coupon. The machine measures the load at which the coupon fails or ruptures. This load and the cross-sectional area of the coupon determine the tensile strength of the steel (See Figures 13.3.9 and 13.3.10).



Figure 13.3.9 Brittle Failure of Cast Iron Specimen



Figure 13.3.10 Ductile Failure of Cold Rolled Steel

13.3.4 Instrumentation

Detection and Warning of Bridge Collapse

Sophisticated sensing devices and electronic equipment can be used to warn motorists of an imminent bridge collapse. Such warning equipment includes radar monitoring, accelerometers attached to the bridge, frequency transmitters and receivers, and a mechanical springboard device.

Laser Beam Line-of-Sight Detectors

When structures suffer fractures or cracks, the weak point sags more than allowed in the design. A laser beam can be directed towards a sensor strategically placed on the structure. As the weak point sags, the sensor moves. When this displacement is greater than the level of tolerance, an alarm is activated. A computer can record the magnitude and rate of change of the deformation. This method can be used to detect potential structural failures.

Strain Gauges

Strain gauges can be used to monitor the response of a member to a known live load. Foil mounted gauges can be used in the axial direction of flat members, and single wire filament, paper mounted gauges can be used on cables. Portable strain reading instruments can be used to monitor all gauges from a central location on or near the bridge.

Locations for strain gauges should be selected based on the condition of individual members, accessibility, and the objectives of the load-testing program. Well-designed strain gauge instrumentation can provide valuable information about:

- The actual transverse load distribution through the deck system
- The load sharing between elements of a multi-element member
- The effectiveness of the various members of the primary structural system

- The influence of deteriorated or defective members

Strain gauge instrumentation data has been interpreted to provide the weights of the vehicles crossing the bridge. This is known as a weighing-in-motion system.

System Identification

In recent years, an increasing number of bridges have been evaluated using measured response data. These have provided useful information and, in some instances, have revealed bridges which needed to be closed or restricted.

Using structural response data, the properties of the structure (e.g., areas and moments of inertia of structural members) can be calculated. The process of determining a structural model from response data is called system identification. The primary use of system identification in structural engineering has been for earthquake engineering research. The accuracy achieved indicates that system identification can also provide a valuable tool for detecting structural flaws.

System identification can be performed using a variety of response data, such as modal and time history response. For modal response, the frequencies and mode shapes of the structure are obtained either from ambient vibration data or from the results of harmonic excitation. A time history response is the response (i.e., displacements or acceleration) of one or more points on the structure as a function of time due to a known loading function. For either type of response data, the results are used to determine structural parameters representing the structural integrity of the bridge.

Initially, system identification is used to create a structural model, which more accurately represents the structure than a model used for design. Subsequent analyses are then performed to determine which parameters are changing. Since the parameters represent structural properties (e.g., areas and moments of inertia), the changes are indicative of structural deterioration.

Since bridge inspections focus on individual members, and system identification considers the entire structure, they are complementary processes. Therefore, system identification can be used to more accurately define the structural integrity of bridges. Using ambient vibrations to perform system identification creates the possibility of continuously monitoring bridges, even from a remote location.

Three-Dimensional Displacements and Strains

Current strain measurements are limited to point-determination or two-dimensional geometries. Research is being conducted to develop optical techniques to measure displacements and strains in three dimensions. The three-dimensional measurements will give a more accurate description of the interaction of bridge elements. The current testing involves global bridge measurement using coherent laser radar (CLR) and global bridge monitoring with wireless transponders. The global measurement using coherent laser radar system was adapted from a rapid inspection system developed for NASA. CLR is portable and can measure bridge deflection with sub-millimeter precision and is available commercially. The global bridge monitoring with wireless transponders system has many transponder/sensor modules that are battery powered. Once installed at a bridge site, these modules transfer information to a local controller using spread spectrum radio. The different types of modules available can measure strain, rotation, or displacement.

New technology is making the use of these highly technical systems economically feasible for bridge inspection. From this fact, advanced inspection techniques are becoming more popular for the routine inspection of all bridge members. Current and future studies have been, and will be focusing directly on relating results from advanced inspection techniques into Bridge Management Systems ratings.

Appendix

PORT AUTHORITY OF ALLEGHENY COUNTY
PITTSBURGH, PENNSYLVANIA

REPORT ON THE
INITIAL NBIS INSPECTION
OF
CHARTIERS CREEK BRIDGE

BMS No. 02 7421 0000 9061

Submitted By:

Michael Baker Jr., Inc.
Airport Office Park, Building 3
Coraopolis, Pennsylvania
15108

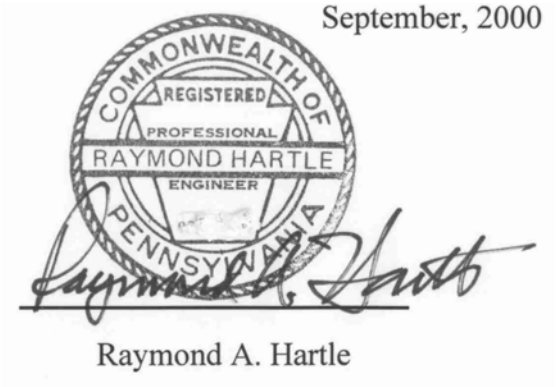
September, 2000

STRUCTURE B.M.S. NUMBER:	02 7421 0000 9061
BRIDGE NAME:	Chartiers Creek Bridge
LOCATION:	Crafton, Pennsylvania
INSPECTION DATE:	June 23, 2000
INSPECTED BY:	Michael Baker Jr., Inc. Patrick A. Leach, P.E. Charles L. Molnar
PREPARED FOR:	Port Authority of Allegheny County
PREPARED BY:	Michael Baker Jr., Inc. Written By: Joseph E. Salvadori, E.I.T. Reviewed By: Raymond A. Hartle, P.E.
PORT AUTHORITY AGREEMENT NUMBER:	92-08
OWNER OF BRIDGE:	Port Authority of Allegheny County

COST INFORMATION:

Inspection & Report	$3,108.00
Rigging	$1,560.00
Traffic Control	$ 0
Railroad	$ 0
Insurance	$ 0

DATE SUBMITTED: September, 2000



Raymond A. Hartle

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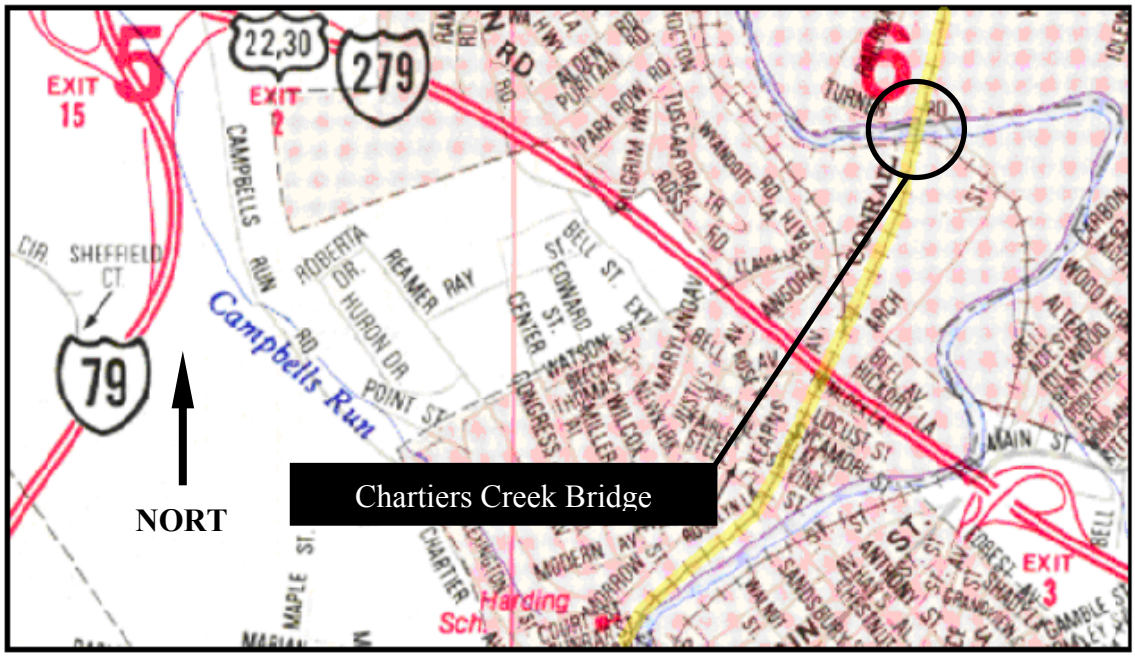
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- Photographs
- Drawings *(Note – Drawings for this structure are not included here.)*
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Location Map
(No Scale)

REPORT ON THE INITIAL NBIS INSPECTION
OF
CHARTIERS CREEK BRIDGE

PORT AUTHORITY OF ALLEGHENY COUNTY

II. INTRODUCTION:

- Location

  Located in the Borough of Crafton, the Chartiers Creek Bridge carries two (2) lanes of the Port Authority of Allegheny County's West Busway over Chartiers Creek, and the Pittsburgh Industrial Railroad, Inc.

- Year Built

  The approximate date of the original construction of the Chartiers Creek Bridge is 1948. The structure was built by the Pennsylvania Railroad Company. Rehabilitation was completed in July 1997.

- Load Posting

  None required.

- Description

  The Chartiers Creek Bridge is a three (3) span, non-composite, riveted and bolted built-up plate girder bridge with a total length of 253'- 11" (see photo no. 1). The 3 spans consist of one (1) main simple span 124'-0", one (1) simple south end span 55'-3", and one (1) simple north end span 68'-3" long. The span lengths are measured between centerline of bearings. The skew angle measured between the centerline of the abutment and West Busway is 90^0. There are AT&T conduits mounted under the deck, and light poles mounted on top of the concrete parapets (see photo no.'s 8 & 2, respectively).

The superstructure consists of four girders spaced at 7'-0" – 6'-0" – 7'-0" on centers, are laterally restrained with angle cross framing, and support an 8 1/2" reinforced concrete deck. The deck thickness includes a 1/2" integral-wearing surface. The deck measures 28'-0" between the reinforced concrete parapets present on both sides of the structure. Galvanized stay-in-place deck forms are present on the underside of the deck (see photo no. 8).

Span 1 girders are made up of a 5'-11" deep by 1/2" thick web plates, and 18" wide by 3/4" thick top and bottom flange plates (see photo no. 8). The main span consists of a 10'- 4 1/2" deep by 1/2" thick web plate, and top and bottom flange plates varying from 20" wide by 7/8" thick, to 20" wide by 1" thick (see photo no. 9). Span 3 girders are made up of a 6'- 10 1/2" deep by 1/2" thick web plate, and 18" wide by 3/4" and 7/8" thick top and bottom flange plates (see photo no. 10). New knee brackets, bolted to the fascia girders, measure 4'-9" wide, from the centerline of existing fascia girders to the centerline of the new W24x55 fascia stringers, with 1/2" thick web plates, and 6" wide by 1/2" thick top and bottom flange plates (see photo no. 4). Lateral bracing and diaphragms consist of angles, and angle x-bracing, respectively. Laminated elastomeric bearing pads are present at the girder ends.

The main span vertical underclearance, from the existing concrete channel bottom, at the centerline of the railroad measures 60'-9" and 36'-7" in span 1.

Gravity type substructures consist of a combination of original stone construction with newly constructed reinforced concrete abutment backwalls and pier caps (see photo no.'s 4 to 7).

III. INSPECTION FINDINGS:

Michael Baker Jr., Inc. performed this initial inspection, which follows NBIS procedures, on June 23, 2000, via a UB-40 underbridge inspection crane. In general, the structure was in good condition with a few minor problems. Several conduits at the south abutment and in span 1 have severely buckled segments, and broken couplers and/or adapters (see photo no.'s 12 & 13). In addition, a conduit in span 3 is split and leaking water (see photo no. 14). These problems are due to the junction boxes being allowed to fill with rainwater during construction.

Approach

The north and south approach roadway and slabs are newly constructed with no deficiencies noted.

Deck

No deficiencies noted – new construction (see photo no. 11). All PennDOT Type 1 scuppers are in excellent condition. A few scuppers exhibit minor debris accumulation but are fully functional (see photo no. 15). Random hairline (< 0.01") shrinkage cracks along the length of the concrete parapets are present (see photo no. 16). Deck expansion joints consist of strip seals in good condition with minor debris accumulation (see photo no. 17).

Superstructure

The superstructure has no visible structural deficiencies. Girders, fascia stringers, knee brackets, and lateral bracing are newly painted. The paint shows no visual defects, but the girders and bracing exhibit evidence of prior minor section loss and member pitting. Fascia stringers and knee brackets are in new condition with no deficiencies noted (see photo no. 4). Diaphragms are in good condition, but show areas of freckled surface rust under the broken

conduit in span 1. Approximately 50% of lateral bracing connections between girders 3 & 4, in span 2, were not painted with final paint coat (see photo no. 18). Laminated elastomeric bearing pads are functioning properly with no problems noted.

Substructure

The north and south abutments are in good condition, with a few minor problems noted. Both abutments have newly constructed reinforced concrete backwalls, bridge seats, and wingwalls with no visual deficiencies noted (see photo no.'s 4 & 5). The stem tops consist of new reinforced concrete construction, also with no visual deficiencies noted, and are attached to the existing stone masonry bases. Some locations of the stone masonry show minor cracking and loosening of mortar.

Piers 1 & 2 are in good condition with minor cracking and loosening of mortar on the existing stone masonry portion of the stems. The bridge seats, caps, and stem tops are newly constructed reinforced concrete with no visual deficiencies noted (see photo no.'s 6 & 7).



Photo No. 1 General Elevation (Upstream)



Photo No.2 South Approach (near)



Photo No.3 North Approach (far)



Photo No.4 South Abutment (near) - Elevation



Photo No.5 North Abutment (far) - Elevation



Photo No.6 Pier 1 - North Face (Looking South)



Photo No.7 Pier 2 - North Face (Looking South), note electrical lines



Photo No.8 General Underside View – Span 1



Photo No.9 General Underside View – Span 2



Photo No. 10 General Underside View – Span 3



Photo No. 11 General Deck View



Photo No. 12 Conduit, Span 1 – note longitudinal crack/split



Photo No. 13 Conduit and Couplers, Span 1 – note bend in conduit, and coupler separation



Photo No.14 Conduit , Span 3 – note conduit is split and leaking water



Photo No. 15 Typical PennDOT Type 1 Scupper

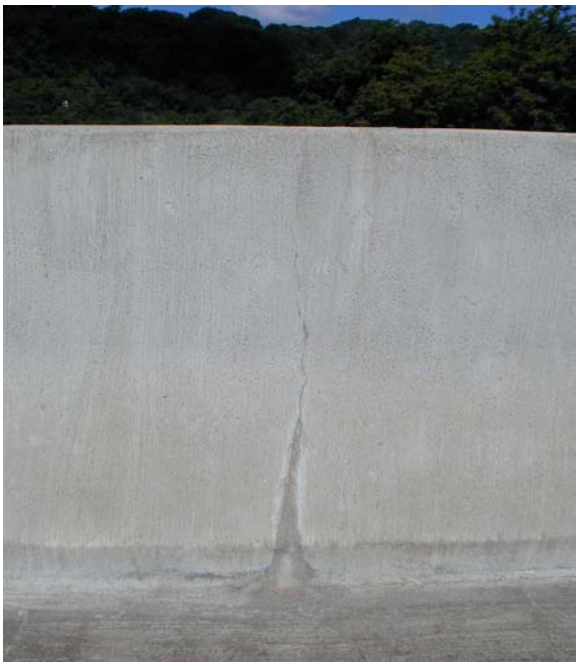


Photo No.16 Typical parapet crack



Photo No. 17 Strip Seal at North Abutment (typ.) – note minor debris accumulation



Photo No.18 Lateral bracing connection between beam #3 and #4, in span 2 – note no final paint coat, and rust freckles

IV. STRUCTURAL ANALYSIS:

Bridge Load Ratings (Tons)

LOAD FACTOR	H	HS	ML	P
Inventory w/o F.W.S	115	159	152	---
Inventory w/ F.W.S	112	155	148	---
Operating w/o F.W.S	191	265	253	346
Operating w/ F.W.S	187	259	247	338

Note: 1) Critical rating is for a beam controlled by shear in span 3
2) Due to no analysis being performed as part of the inspection, the above table is reproduced from contract drawings.

V. RECOMMENDATIONS AND COST ESTIMATE:

Repairs

Item	Estimated Quantity	Unit Cost	Total Cost
Drain junction boxes, and conduits filled with water. Repair bent conduits, and broken couplers/adapters.	N/A	Lump Sum	$5,000.00
Paint locations requiring final paint coat between girders 3 & 4 in span 2.	20 SF	Lump Sum	$1,000.00

TOTAL COST $6,000.00

Note: The above costs are only for the items listed and do not include additional costs which would be incurred when the work is performed, such as mobilization, maintenance and traffic protection, engineering, etc.

PDT Form D-450A (DEC 1996)

Site Data

BRIDGE MANAGEMENT SYSTEM
BRIDGE INSPECTION REPORT

BMS Updated by _____ Date _____

A01: 027421 0000 9061

C05 Structure Type (Dept.): Main — STL. RIVETED I-BEAM — 19110

CHARTIERS CREEK BRIDGE Over CHARTIERS CREEK Approach

Inspection Date E06: 062300

Name of Consultant and/or Inspectors E12: MICHAEL BAKER JR., INC.

Inspection Type E07: 1

Inspected by E08: 8

Hired by E13: 8

Time started: 7:30 A.M.

Weather Conditions: MOSTLY SUNNY Temp: 84

Time completed: 4:30 P.M.

CRAFTON

☐ City ☒ Borough ☐ Township

Optional Reminder: Check boxes if Maintenance Activities are needed --> X

Bridge Signing Verification

BMS Item	Type of Sign	Required Sign	SIGNING IN FIELD: Near Advance	Bridge Site: Near	Bridge Site: Far	Far Advance	Comments
D15	Bridge Weight Limit	N/A T					NONE POSTED
D15	Except Combination	N/A T					
D14	One Truck at a Time	Yes /(No)					
B22/B23	Vert. Clearance - On	N/A					See Sketch
B22/B23	Vert. Clearance - Und	N/A					See Sketch
	One Lane Bridge	Yes /(No)	(Opt)			(Opt)	
	Narrow Bridge	Yes /(No)	(Opt)			(Opt)	
	Hazard Clearance	Yes /(No)					
	Other						
(Opt)	Other						

Key --> OK: Signs properly installed M : Signs missing D : Signs damaged / incorrect New Wearing Surface Under Bridge: YES ☐ NO ☐

Notes

Vert. Clear. Sign **On Feature:** B01 = ___ B31 = ___ **Under Feature:** B01 = ___ B31 = ___

E26 Underclearance Appraisal 5 Controlling: Lateral 12'-2" Vertical 36'-7"

E28-A Traffic Safety Features (Subfields shown vertically) Posted Speed Limit ___ mph

6 Bridge Railing: PARAPET - JERSEY BARRIER. (GOOD CONDITION - MINOR CRACKING THROUGHOUT)

8 Transition: PARAPET EXTENSIONS.

8 Approach Guiderail: ON RIGHT - CONTINUOUS NJ BARRIER - GOOD. W-BEAM AND STL. POSTS ON NEAR LT. AND FAR LT.

6 Approach Rail Ends: FLARED AND TURNED DOWN W-BEAM ON NEAR LT. AND FAR LT.

E28 Approach Alignment 8: NO SPEED REDUCTION. GOOD SIGHT DISTANCE.

E15 Approach Roadway 8: NEW PAVEMENT GOOD CONDITION.

Pavement: GOOD

Drainage: GOOD (ALL NEW CONSTRUCTION)

Shoulders: GOOD

E14 Approach Slab 8: NEW CONSTRUCTION.

Bump at Bridge Yes ☐ No ☒

C19 Relief Joint 1

PDT Form D-450B (DEC 1996)

Bridge 1 Data

Inspection Date

A01: 02 7421 0000 9061 — E06: 062300

For Non-State Roadways

B01 Ref	B27 ADT	B28 ADT YR	B30A ADTT %

For State highways, data from RMS will be used.

E25 Deck Geometry: 6 — Table ____ Controlling Values: B27 / B34 / B22 ____ A31 / A31 / B18 ____

Design Exception granted ? ____

E16 Deck Wearing Surface: 9 NEW CONSTRUCTION (CONCRETE INTEGRAL)

C10 Wearing Surface Type: 101 — C10A Wearing Surface Thickness: 05

E17 Deck: 9 — Estimated Spall or Delamination ____ % — Est. Chloride Content ____

Top: EXCELLENT CONDITION - NEW CONSTRUCTION.

Underside: STAY IN PLACE FORMS (NO RUSTING NOTED) GALVANIZED AND IN GOOD CONDITION.

Exp Joint No. 4 — C22 Exp Jt Types: M B G

GOOD CONDITION - SOME MINOR DIRT BUILD UP. (STRIP SEALS)

Deck Drainage: GOOD - SOME SCUPPERS HAVE DEBRIS BUT NOT IN THE DOWNSPOUT.

E18 Superstructure: 7 — See Sheet ____ for Additional Details. — Form 491-J attached for FCM details Yes/No ☐

Girders / Beams: GOOD CONDITION - SUPERSTRUCTURE HAS BEEN RECONSTRUCTED FOR NEW BUSWAY BRIDGE. NEW PAINT/COATING OVER PREVIOUS PITTING/MORE SECTION LOSS. ALSO, SOME AREAS OVER LIGHT SURFACE RUST ON BOTTOM FLANGE. (THROUGHOUT)

Floorbeams: N/A

Stringers: NEW (FASCIA STRINGERS) W24 X 55 EXCELLENT CONDITION.

Diaphrams: GOOD CONDITION. FEW AREAS OF FRECKLED SURFACE RUST UNDER BROKEN CONDUIT IN SPAN 1.

Truss Members: N/A

Portals / Bracing: FEW AREAS OF FRECKLED SURFACE RUST UNDER BROKEN CONDUIT IN SPAN 1. SEVERAL AREAS BETWEEN G3 AND G4 IN SPAN 2 WERE NOT PAINTED WITH FINAL COAT.

Bearings: GOOD CONDITION. (LAMINATED ELASTOMERIC)

Drainage System (Below Deck): EXCELLENT CONDITION. (TYPE 1 SCUPPERS)

PDT Form D-450C (DEC 1996)

Abutment Data

Inspection Date

A01	0	2	7	4	2	1	0	0	0	0	9	0	6	1	E06	0	6	2	3	0	0

E20 Substructure 7 Details on Sheet

NAB - Near Abutment (Use same notation as W09)

Backwall GOOD CONDITION - NEW CONSTRUCTION.

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION. VERY MINOR DEBRIS.

Cheekwalls

Stem GOOD CONDITION - NEW CONCRETE CONSTRUCTION AT TOP ON EXISTING STONE MASONRY BASE. SOME LOCATIONS HAVE MINOR CRACKING AND LOOSENING OF MORTAR.

Wings GOOD CONDITION - NEW CONSTRUCTION.

Footing NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

ABUTMENT IS NOT IN CHANNEL. ALSO, CHANNEL IS CONCRETE LINED.

Settlement NONE NOTED.

Embank-Slope-Wall GOOD CONDITION - HEAVY VEGETATION.

Wall Drainage

FAB - Far Abutment (Use same notation as W09)

Backwall GOOD CONDITION - NEW CONSTRUCTION.

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION. MINOR DEBRIS.

Cheekwalls

Stem GOOD CONDITION - SAME AS NEAR ABUTMENT.

Wings GOOD CONDITION - NEW CONSTRUCTION.

Footing NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

ABUTMENT IS NOT IN THE CHANNEL.

Settlement NONE NOTED.

Embank-Slope-Wall HEAVY VEGETATION.

Wall Drainage

PDT Form D-450D (DEC 1996)

Pier Data

A01	0 2	7 4 2 1	0 0 0 0	9 0 6 1	Inspection Date E06	0 6 2 3 0 0

Substructure (Cont.)

Pier / Bent Number 1 (Use same notation as W09)

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION.

Caps GOOD CONDITION - NEW CONSTRUCTION.

Cheekwalls

Columns/Stems GOOD CONDITION - NEW CONSTRUCTION ON TOP OF EXISTING STONE MASONRY BASE. MINOR CRACKING AND LOOSE MORTAR.

Footings NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

NOT IN CHANNEL - CHANNEL IS CONCRETE LINED.

Settlement NONE NOTED.

Pier / Bent Number 2 (Use same notation as W09)

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION.

Caps GOOD CONDITION - NEW CONSTRUCTION.

Cheekwalls

Columns/Stems GOOD CONDITION - SAME AS PIER 1.

Footings NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

CHANNEL IS CONCRETE LINED.

Settlement NONE NOTED.

PDT Form D-450E
(DEC 1996)

Waterway 1 Data

BRIDGE MANAGEMENT SYSTEM
BRIDGE INSPECTION REPORT

BMS Updated by _____ Date _____

A01: 02 7421 0000 9061 — U.W. Inspection Date W01-A: _____

Over _____ Weather Conditions _____

Inspection Type W02: N — U.W. Inspection Type W02-A: _____ — Regular U.W. Insp. Freq. W03: _____ — Interim U.W. Inps. Freq. W04: _____

Time started _____ Time completed _____

Name of Consultant and/or Inspectors W16: _____ — Hired by W17: _____ — Inspection Cost W15: _____

Scour Critical Rating E29A: W06: 9 based on: [X] Observed Scour [ ] Scour Calculation

No. of Units Inspected W14: _____

Streambed Material W07: C 8 _____ (36 SPACES) CONCRET LINED CHANNEL.

E21 Channel/Channel Protection - Cond. Rating: 7 Details on Sheet _____

Channel: CHANNEL IS LINED WITH CONCRETE.

Banks: GOOD CONDITION - HEAVY VEGETATION.

Streambed Movements: NONE NOTED.

Debris, Vegetation: SOME DEBRIS IN CHANNEL.

River (Stream) Control Devices: N/A

Embankment / Streambed Controls: N/A

Drift, Other: NONE NOTED.

E27 Waterway Adequacy: 9

Risk of Overtopping [X] Remote [ ] Slight [ ] Occasional [ ] Frequent

Traffic Delay [X] Insignificant [ ] Significant [ ] Severe **B18 - Functional Class.** _____

High Water Mark: ELEV : _____ DATE (mmyyyy) _____ [ ] New HW Mark [ ] HW since last inspecion

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures
NAB	P	00	9	E	000	

Findings: ABUTMENT OUT OF FLOOD PLANE.

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures
P01	P	00	9	E	000	

Findings: _____

Form D-450F
(DEC 1996)

Waterway 2 Data

A01	02	7421	0000	9061	W01-A	U.W. Inspection Date

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures
P02	P	00	9	E	000	

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures
FAB	P	00	9	E	000	

Findings: ABUTMENT OUT OF FLOOD PLANE.

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

PDT Form D-450G (DEC 1996)

Waterway 3 Data

U.W. Inspection Date

A01	0	2	7	4	2	1	0	0	0	0	9	0	6	1	W01-A						

OBSERVED SCOUR RATING GUIDE

Rating	ITEM NUMBER								Rating
	1	2	3	4	5	6	7	8	
	Change Since Last Inspection	Scour Hole	Debris Potential	Substructure Scourability	Opening Adequacy/ Channel	Sediment	Alignment	Velocity/ Stream Slope	
9	None	None	None	NF/P9/R9	Good	None	Good	Low	9
8	None	Minor	None	P8/C8/R8	Good	Minor	Good	Low	8
7	Minor	Minor	Minor	P7/C7/R7	Fair	Minor	Good	Medium	7
6	Minor	Advanced	Medium*	A6	Fair	Medium	Medium	Medium	6
5	Medium*	Advanced	High*	A5	Fair	High	Medium	High	5
4	Medium	Serious*	High	R4*/A4*	Poor*	High	Poor*+	High	4
3	High*	Serious*	Present*	A3	Overtop*	High	Poor	High	3
2	Bridge is scour critical, IMMEDIATE action is required *								2
1	Bridge is scour critical, bridge is CLOSED *								1
0	Bridge has failed due to scour *								0

NOTES:

Rating considerations given in highest to lowest level of importance from left to right.

* If an item is so marked, it cannot be given a higher ranking.

s founded on competent rock and no problems exist.

C = Effective Countermeasures

P = Pile Supported Substructures

DETERMINATION OF RATING FOR BMS ITEM W11-A

Substructure Unit	1	2	3	4	5	6	7	8	**W11-A**
	Change Since Last Inspection	Scour Hole	Debris Potential	Scourability	Opening Adequacy/ Channel	Sediment	Alignment	Velocity/ Stream Slope	Overall Observed Scour Rating
P02	9	9	8	8	9	7	9	7	9

If Underwater Inspection only

Signatures and Date:

PDT Form D-450K (DEC 1996) **Bridge 2 Data**

Inspection Date

A01 02 | 7421 | 0000 | 9061 | E06 062300

E19 Paint Condition 88 New Paint Y / N If Yes: [ ] Spot [ ] Zone [X] Full [ ] Revise item G08-G17 [ ]

Interior Beam / Girder: VERY GOOD - RECENTLY REPAINTED.

Fascias: VERY GOOD - NEW.

Splash Zone: Truss / Girder:

Truss:

Bearings: VERY GOOD.

Other:

E23 Est. Remaining Life BMS to Calculate Yes/No 34 Comments:

Recalculate IR/OR: Yes [ ] Due to: Deterioration [ ] New Wearing Surf. [ ] Other [ ]

No [X] Previous Rating Dated _____ is still valid

	H	HS	ML-80	Other	Other	HS Load Factor
E30 Inventory Rating	198	298	898			298
E31 Operating Rating	198	298	898			298

E32 Rate Meth 2S E33 Typ Mem 1 AASHTO E37 Spec 94 E38 Manual 94

E29 Bridge Post 9 CONTROLLING: H ____ HS ____ ML80 X Engineering Judgement ____

E24 Structural Condition Appraisal 7 Based upon [ ] Table 1 B27-ADT ____ B30-IR ____

or E18-Super 7 E20-Sub 7, E22-Culvert ____,

E01 Next Insp. Freq. 24 E03 Equip. Next Insp. B SNOOPER TRUCK (UB-40)

E04 Spec. Insp. Type [ ] E05 By Date [ ]

Is bridge over water? [X] Yes. E22 = N Complete Forms D-450E through G

[ ] No. E22 = N E21 = N E27 = N E29A = N

Notes: ONE SPAN IS OVER WATER AND ONE SPAN IS OVER RAILROAD.
HAD RAILROAD REPRESENTITIVE ON SITE. CREW WAS OUT OF SPAN 1 (RR LOCATION) BY TIME REQUIRED. (9 A.M.)
INSPECTION WAS FIRST ON NEWLY CONSTRUCTED BUSWAY BRIDGE.WHICH USED AN EXISTING RR BRIDGE.
CONDUITS ON BRIDGE WERE BUSTED AT ADAPTERS AT ABUTMENT 1. ALSO, ONE EXPANSION COUPLER WAS BROKEN AND NEEDED REPLACED. SEVERAL CONDUIT SEGMENTS IN SPAN 1 WERE SEVERELY BUCKLED AND NEEDED REPLACED.

Signatures and Date: PATRICK LEACH, P.E. - 6/23/00
CHARLES MOLNAR - 6/23/00

PDT Form D-450M **Maintenance Needs Data**

(DEC 1996)

A01	02	7421	0000	9061

Inspection Date: E06 | 062300

Approach Roadway Work

	Item #	Location		Quantity	PR	D/C
Pavement (Patch/Raise)	RDPAVMT	L N R L F R	SY			
Pavement Relief Jt (Rep/Repl)	RDRLFJT	L N R L F R	SY			
Shoulders (Repair/Reconstr)	RDSHLDR	L N R L F R	SY			
Drainage-Off Bridge (Improve)	RDDRAIN	L N R L F R	EA			
GR/Trans/End (Rep/Repl/Imp)	RDGDERL	L N R L F R	EA			
Load Limit Signs (Replace)	RDLDSGN	L N R L F R	EA			
Clearance Signs (Replace)	RDCLSGN	L N R L F R	EA			
Cut Brush to Clear Signs	RDBRUSH	L N R L F R	EA			
Approach Slab (Replace)	A744201	L N R L F R	SY			

Cleaning - Flushing

Deck	A743101	-- --	EB			
Scupper/Down Spouting	B743101	1 2 3 4 5 O	EB			
Bearing/Bearing Seat	C743102	1 2 3 4 5 O	EB			
Steel-Horizontal Surfaces	D743102	1 2 3 4 5 O	EB			

Deck

Bitum Deck W Surf (Rep/Repl)	BITWRGS	1 2 3 4 5 O	SY			
Timber Deck (Rep/Repl)	B744301	1 2 3 4 5 O	SY			
Open Steel Grid (Rep/Repl)	C744302	1 2 3 4 5 O	SY			
Concrete Deck (Repair)	D744303	1 2 3 4 5 O	SY			
Concrete Sidewalk (Repair)	E744303	1 2 3 4 5 O	SY			
Concrete Curb/Parapet (Rep)	F744303	1 2 3 4 5 O	SY			

Deck Joints - Expansion Joints

Reseal	A743301	N 1 2 3 O F	LF			
Repair/Reseal	A744101	N 1 2 3 O F	LF			
Compression Seal (Rep/Rehab)	B744102	N 1 2 3 O F	LF			
Modular Dam (Rep/Rehab)	C744102	N 1 2 3 O F	LF			
Steel Dams (Rep/Rehab)	D744102	N 1 2 3 O F	LF			
Other Types (Rep/Rehab)	E744102	N 1 2 3 O F	LF			

Bridge Railings - Parapets

Bridge Parapet (Rep/Repl)	RLGBRPR	N 1 2 3 O F	LF			
Struct Mount GR (Rep/Repl)	RLGSTRM	N 1 2 3 O F	LF			
Pedestrian (Rep/Repl)	RLGPEDN	N 1 2 3 O F	LF			
Median Barrier (Rep/Repl)	RLGMEDB	1 2 3 4 5 O	LF			

Deck Drainage

Scupper Grate (Replace)	DRNGRAT	1 2 3 4 5 O	EA			
Drain/Scupper (Install)	B744401	1 2 3 4 5 O	EA			
Downspouting (Rep/Repl)	C744402	N 1 2 3 O F	EA			

Bearings

Lubricate	A743501	N 1 2 3 O F	EA			
Steel (Rep/Rehab)	A744501	N 1 2 3 O F	EA			
Steel (Replace)	B744501	N 1 2 3 O F	EA			
Expansion (Reset)	C744502	N 1 2 3 O F	EA			
Pedestal/Seat (Reconstruct)	D744503	N 1 2 3 O F	EA			

Timber

Stringer (Rep/Repl)	A744601	1 2 3 4 5 O	EA			
Other Members (Rep/Repl)	B744601	1 2 3 4 5 O	EA			

Steel

	Item #	Location		Quantity	PR	D/C
Stringer (Rep/Repl)	A744602	1 2 3 4 5 O	EA			
Floorbeam (Rep/Repl)	B744602	1 2 3 4 5 O	EA			
Girder (Repair)	C744602	1 2 3 4 5 O	EA			
Diaph/Lat. Bracing (Rep/Repl)	D744602	1 2 3 4 5 O	EA			

Reinforced, PS, PC, and PT Concrete

Stringer (Rep/Repl)	A744603	1 2 3 4 5 O	EA			
Diaphragm (Rep/Repl)	B744603	1 2 3 4 5 O	EA			
Other Members (Rep/Repl)	C744603	1 2 3 4 5 O	EA			

Truss

Members (Strengthen/Rep/Repl)	A744701	1 2 3 4 5 O	EA			
Portal (Modify)	B744701	1 2 3 4 5 O	EA			
Members(Tighten/Flameshorten)	C744702	1 2 3 4 5 O	EA			

Painting

Superstructure - Spot	A743201	1 (2) 3 4 5 O	EB	1	4	
Substructure - Spot	B743201	N 1 2 3 O F	EB			
Superstructure - Full	C743201	1 2 3 4 5 O	EB			
Substructure - Full	D743201	N 1 2 3 O F	EB			

Abutment - Wings - Piers

Backwall (Rep/Repl)	A744801	L N R L F R	CY			
Abutments (Repair)	B744802	L N R L F R	CY			
Wing (Rep/Repl)	C744802	L N R L F R	CY			
Piers (Repair)	D744802	1 2 3 4 5 O	CY			
Footing (Underpin)	E744803	N 1 2 3 O F	CY			
Masonry (Repoint)	F744804	N 1 2 3 O F	LF			
Abut Slopewall (Rep/Repl)	A745101	L N R L F R	SY			
Abut Slopewall (Construct New)	B745102	L N R L F R	CY			
Pile Repair	A745901	N 1 2 3 O F	EA			

Scour - Erosion Control

Streambed Paving (Rep/Constr)	A745301	UP UN DN	CY			
Rock Protection	B745301	UP UN DN	CY			
Scour Hole (Backfill)	C745301	UP UN DN	CY			
Stream Deflector (Rep/Constr)	D745302	UP UN DN	CY			
Vegetation/Debris (Remove)	ECREMVG	UP UN DN	CY			
Deposition (Remove)	ECREMDP	UP UN DN	CY			

Culvert

Headwall/Wings (Rep/Repl)	A745201	IN OUT	SY			
Apron/Cutoff Wall (Rep/Repl)	B745202	IN OUT	SY			
Barrel (Repair)	C745203	-- --	SY			

FOR COMPLETION BY REVIEW ENGINEER

Apply Protective Coating

Deck/Parapet/Sidewalk	A743401	DK PARA SW	SY			
Substructure	B743401	N 1 2 3 O F	SY			

Construct Temporary

Support Pier	A745401	N 1 2 3 O F	EA			
Pipe/Culvert Crossing	B745401	LT CL RT	EB			
Bridge	C745401	LT CL RT	EB			

REP..... Repair
N..... Near
F..... Far
O..... Other
IN..... Inlet

REPL..... Replace
UP..... Upstream
DN..... Downstream
UN..... Under
OUT ..... Outlet

IMP..... Improve
LNR..... Near Left/Right
LFR..... Far Left/Right
1,2,3, etc..... Span/Pier No.
EB..... Each Bridge (site)

PR - PRIORITY CODE

0 - Prompt action required. (Inform Bridge Engineer before updating BMS)
1 - High Priority, as soon as work can be scheduled.
2 - Priority, review work plan, adjust schedule if needed.
3 - Add to scheduled work.
4 - Routine structural, can be delayed until funds are available.
5 - Routine non-structural, can be delayed until programmed,

MAJOR IMPROVEMENT NEEDS

F01 **Year Needed**

F02 **Type Work**

F10 **Future ADT**

F04 **Improvement Length**

F06 **Bridge Width**

F11 **Future ADT Year**

Reviewed On: .....

By: .....

Note: The Appendix section for this report is not included here. The BMS 491 Forms for PENNDOT are that state's version of the FHWA SI&A sheet with additional state items. The documents included in the report are typically red marked revisions to the file copy and reflect changes identified during the inspection.

Subpart C - National Bridge Inspection Standards

§ 650.301 Application of standards.

The National Bridge Inspection Standards in this part apply to all structures defined as bridges located on all public roads. In accordance with the AASHTO (American Association of State Highway and Transportation Officials) Transportation Glossary, a bridge is defined as a structure including supports erected over a depression or an obstruction, such as water, high-way, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

[44 FR 25435, May 1. 1979. as amended at 51 FR 16834. May 7. 19863]

Federal Highway Administration, DOT

§ 650.303 Inspection procedures.

(a) Each highway department shall include a bridge inspection organization capable of performing inspections, preparing reports, and determining ratings in accordance with the provisions of the AASHTO Manual[1] and the Standards contained herein.

(b) Bridge inspectors shall meet the minimum qualifications stated in § 650.307.

(c) Each structure required to be inspected under the Standards shall be rated as to its safe load carrying capacity in accordance with section 4 of the AASHTO Manual. If it is determined under this rating procedure that the maximum legal load under State law exceeds the load permitted under the Operating Rating, the bridge must be posted in conformity with the AASHTO Manual or in accordance with State law.

(d) Inspection records and bridge inventories shall be prepared and maintained in accordance with the Standards.

(e) The individual in charge of the organizational unit that has been dele-gated the responsibilities for bridge inspection, reporting and inventory shall determine and designate on the individual inspection and inventory records and maintain a master list of the following:

(1) Those bridges, which contain fracture critical members, the location and description of such members on the bridge and the inspection frequency and procedures for inspection of such members. (Fracture critical members are tension members of a bridge whose failure will probably cause a portion of or the entire bridge to collapse.)

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[1] The AASHTO Manual referred to In this part Is the Manual for Maintenance Inspection of Bridges 19,q3 together with subsequent interim changes or the most recent version of the AASHTO Manual published by the American Association of State Highway and Transportation Officials. A copy of the Manual may be examined during normal business hours at the office of each Division Administrator of the Federal Highway Administration, at the office of each Regional Federal Highway Administrator, and at the Washington Headquarters of the Federal Highway Administration. The addresses of those document inspection facilities are set forth in appendix D to part 7 of the regulations of the Office of the Secretary (49 CFR part 7). In addition, a copy of the Manual may be secured upon payment in advance by writing to the American Association of State Highway and Transportation Officials, 444 N. Capitol Street, NW., Suite 225. Washington, DC 20001.

(2) Those bridges with underwater members which cannot be visually evaluated during periods of low flow or examined by feel for condition, integrity and safe load capacity due to excessive water depth or turbidity. These members shall be described, the inspection frequency stated, not to exceed five years, and the inspection procedure specified.

(3) Those bridges, which contain unique or special features requiring additional attention during inspection to ensure the safety of such bridges and the inspection frequency and procedure for inspection of each such feature.

(4) The date of last inspection of the features designated in paragraphs (e)(1) through (3) of this section and a description of the findings and follow-up actions, if necessary, resulting from the most recent inspection of fracture critical details, underwater members or special features of each so designated bridge.

[36 FR 7851, Apr. 27, 1971. Re-designated at 39 FR 10430, Mar. 20, 1974, as amended at 44 FR 25435. May 1, 1979; 53 FR 32W. Aug. 26. 1988]

§ 650.305 Frequency of inspections.

(a) Each bridge is to be inspected at regular intervals not to exceed 2 years in accordance with section 2.3 of the AASHTO Manual.

(b) Certain types or groups of bridges will require inspection at less than 2-year intervals. The depth and frequency to which bridges are to be inspected will depend on such factors as age, traffic characteristics, state of maintenance, and known deficiencies. The evaluation of these factors will be the responsibility of the individual in charge of the inspection program.

(c) The maximum inspection interval may be increased for certain types or groups of bridges where past inspection reports and favorable experience and analysis justify the increased interval of inspection. If a State proposes to inspect some bridges at greater than the specified two-year interval, the State shall submit a detailed proposal and supporting data to the Federal High-way Administrator for approval. The maximum time period between inspections shall not exceed four years.

[36 FR 7851, Apr. 27, 1971. Re-designated at 39 FR 10430, Mar. 20, 1974, as amended at 39 FR 29590, Aug. 16, 1974; 53 FR 32616, Aug. 26, 1988; 57 FR 53281, Nov. 9, 1992]

§ 650.307 Qualifications of personnel.

(a) The individual in charge of the organizational unit that has been delegated the responsibilities for bridge inspection, reporting, and Inventory shall possess the following minimum qualifications:

(1) Be a registered professional engineer; or

(2) Be qualified for registration as a professional engineer under the laws of the State; or

(3) Have a minimum of 10 years experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the "Bridge Inspector's Training Manual,"[2] which has been developed by a joint Federal-State task force, and subsequent additions to the manual.[3]

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[2] The "Bridge Inspector's Training Manual" may be purchased from the Superintendent of Documents. U.S. Government Printing Office, Washington, DC 20402.

[3] The following publications are supplemental to the "Bridge Inspector's Training Manual": "Bridge Inspector's Manual for Movable Bridges." 1977, GPO Stock No. 050-002-00103-5; "Culvert Inspector's Training Manual," July 1986, GPO Stock No. 050-001-0030-7; and "Inspection of Fracture Critical Bridge Members," 1986, GPO Stock No. 050-001-00302-3.

(b) An individual in charge of a bridge inspection team shall possess the following minimum qualifications:

(1) Have the qualifications specified in paragraph (a) of this section; or

(2) Have a minimum of 5 years experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the "Bridge Inspector's Training Manual," which has been developed by a joint Federal-State task force.

(3) Current certification as a Level Ill or IV Bridge Safety Inspector under the National Society of Professional Engineer's program for National Certification in Engineering Technologies (NICET)[4] is an alternate acceptable means for establishing that a bridge in section team leader is qualified.

[36 FR 7851. Apr. 27, 1971. Re-designated at 39 FR 10430, Mar. 20. 1974. as amended at 44 FR 25435, May 1, 1979; 53 FR 32616. Aug. 26, 1988]

§ 650.309 Inspection report.

The findings and results of bridge inspections shall be recorded on standard forms. The data required to complete the forms and the functions which must be performed to compile the data are contained in section 3 of the AASHTO Manual.

[139 FR 29590, Aug. 16, 1974]

§ 650.311 Inventory.

(a) Each State shall prepare and maintain an inventory of all bridge structures subject to the Standards. Under these Standards, certain structure inventory and appraisal data must be collected and retained within the various departments of the State organization for collection by the Federal Highway Administration as needed. A tabulation of this data is contained in the structure inventory and appraisal sheet distributed by the Federal High-way Administration as part of the Re-cording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide) In January of 1979. Reporting procedures have been developed by the Federal Highway Administration.

(b) Newly completed structures, modification of existing structures which would alter previously recorded data on the Inventory forms or placement of load restriction signs on the approaches to or at the structure itself shall be entered in the State's inspection reports and the computer inventory file as promptly as practical, but no later than 90 days after the change in the status of the structure for bridges directly under the State's jurisdiction and no later than 180 days after the change in status of the structure for all other bridges on public roads within the State.

[44 FR 25435. May 1. 1979. as amended at 53 FR 32617. Aug. 26. 19881]

[4] For Information on NICET program certification contact: National Institute for Certification in Engineering Technologies. 1420 King Street. Alexandria, Virginia 22314. Attention: John D. Antrim. P.E. Phone (703) 684-2835.

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GLOSSARY

A

AASHTO - American Association of State Highway and Transportation Officials, name changed from AASHO (American Association of State Highway Officials) in 1973

abrasion - wearing or grinding away of material by friction; usually caused by sand, gravel, or stones, carried by wind or water

absorption - the process of a liquid being taken into a permeable solid (e.g., the wetting of concrete)

abutment - part of bridge substructure at either end of bridge which transfers loads from superstructure to foundation and provides lateral support for the approach roadway embankment

ADT - Average Daily Traffic

ADTT - Average Daily Truck Traffic

admixture - an ingredient added to concrete other than cement, aggregate or water (e.g., air entraining agent)

aggradation - progressive raising of a streambed by deposition of sediment

aggregate - hard inert material such as sand, gravel, or crushed rock that may be combined with a cementing material to form mortar or concrete

air entrainment - the addition of air into a concrete mixture in order to increase the durability and resist thermal forces

alignment - the relative horizontal and vertical positioning between components, such as the bridge and its approaches

alignment bearing - a bearing embedded in a bridge seat to prevent lateral movements (see BEARING)

alligator cracking - cracks initiated by inadequate base support or drainage that form on the surface of a road in adjacent, rectangular shapes (like the skin of an alligator)

alloy - two or more metals, or metal and non-metal, intimately combined, usually by dissolving together in a molten state to form a new base metal

anchorage - the complete assemblage of members and parts, embedded in concrete, rock or other fixed material, designed to hold a portion of a structure in correct position

anchor bolt - a metal rod or bar commonly threaded and fitted with a nut and washer at one end only, used to secure in a fixed position upon the substructure the bearings of a bridge, the base of a column, a pedestal, shoe, or other member of a structure

anchor span - the span that counterbalances and holds in equilibrium the cantilevered portion of an adjacent span; also called the back span; see CANTILEVER BEAM, GIRDER, or TRUSS

angle - a basic member shape, usually steel, in the form of an "L"

anisotropy - the property of certain materials, such as crystals, that exhibits different strengths in different directions

anode - the positively charged pole of a corrosion cell at which oxidation occurs

anti-friction bearing - a ball or roller-type bearing; a bearing that reduces transfer of horizontal loads between components

appraisal rating - a judgment of a bridge component's adequacy in comparison to current standards

approach - the part of the roadway immediately before and after the bridge structure

approach pavement - an approach which has a cross section that is either the same as or slightly wider than the bridge deck width

approach slab - a reinforced concrete slab placed on the approach embankment adjacent to and usually resting upon the abutment back wall; the function of the approach slab is to carry wheel loads on the approaches directly to the abutment, thereby transitioning any approach roadway misalignment due to approach embankment settlement

appurtenance - an element that contributes to the general functionality of the bridge site (e.g., lighting, signing)

apron - a form of scour (erosion) protection consisting of timber, concrete, riprap, paving, or other construction material placed adjacent to abutments and piers to prevent undermining

arch - a curved structure element primarily in compression that transfers vertical loads through inclined reactions to its end supports

arch barrel - a single arch member that extends the width of the structure

arch rib - the main support element used in open spandrel arch construction; also known as arch ring

armor - a secondary steel member installed to protect a vulnerable part of another member, e.g., steel angles placed over the edges of a joint; also scour protection such as rip rap

as-built plans - plans made after the construction of a project, showing all field changes to the final design plans (i.e.. showing how the bridge was actually built)

asphalt - a brown to black bituminous substance that is found in natural beds and is also obtained as a residue in petroleum refining and that consists chiefly of hydrocarbons; an asphaltic composition used for pavements and as a waterproof cement

ASTM - American Society for Testing and Materials

auger - a drill with a spiral channel used for boring

axial - in line with the longitudinal axis of a member

axle load - the load borne by one axle of a traffic vehicle, a movable bridge, or other motive equipment or device and transmitted through a wheel or wheels

B

back - see EXTRADOS

backfill - material, usually soil or coarse aggregate, used to fill the unoccupied portion of a substructure excavation such as behind an abutment stem and backwall

backstay - cable or chain attached at the top of a tower and extending to and secured upon the anchorage to resist overturning stresses exerted upon the tower by a suspended span

backwall - the topmost portion of an abutment above the elevation of the bridge seat, functioning primarily as a retaining wall with a live load surcharge; it may serve also as a support for the extreme end of the bridge deck and the approach slab

backwater - the back up of water in a stream due to a downstream obstruction or constriction

bank - sloped sides of a waterway channel or approach roadway, short for embankment

bascule bridge - a bridge over a waterway with one or two leaves which rotate from a horizontal to a near-vertical position, providing unlimited overhead clearance

base course - a layer of compacted material found just below the wearing course that supports the pavement

base metal - the surface metal of a steel element to be incorporated in a welded joint; also known as structure metal, parent metal

base plate - steel plate, whether cast, rolled or forged, connected to a column, bearing or other member to transmit and distribute its load to the substructure

batten plate - a plate with two or more fasteners at each end used in lieu of lacing to tie together the shapes comprising a built-up member

batter - the inclination of a surface in relation to a horizontal or a vertical plane; commonly designated on bridge detail plans as a ratio (e.g., 1:3, H:V); see RAKE

battered pile - a pile driven in an inclined position to resist horizontal forces as well as vertical forces

bay - the area of a bridge floor system between adjacent multi-beams or between adjacent floor beams

beam - a linear structural member designed to span from one support to another and support vertical loads

bearing - a support element transferring loads from superstructure to substructure while permitting limited movement capability

bearing capacity - the load per unit area which a structural material, rock, or soil can safely carry

bearing failure - crushing of material under extreme compressive load

bearing pile - a pile which provides support through the tip (or lower end) of the pile

bearing plate - a steel plate, which transfers loads from the superstructure to the substructure

bearing pressure - the bearing load divided by the area to which it is applied

bearing seat - a prepared horizontal surface at or near the top of a substructure unit upon which the bearings are placed

bearing stiffener - a vertical web stiffener at the bearing location

bearing stress - see BEARING PRESSURE

bedding - the soil or backfill material used to support pipe culverts

bedrock - the undisturbed rock layer below the surface soil

bench mark - an established reference point with known elevation and coordinates, used to document dimensions, elevations or position movement

bending moment - the internal force within a beam resulting from transverse loading

bent - a substructure unit made up of two or more column or column-like members connected at their top-most ends by a cap, strut, or other member holding them in their correct positions

berm - the line that defines the location where the top surface of an approach embankment or causeway is intersected by the surface of the side slope

beveled washer - a wedge-shaped washer used in connections incorporating members with sloped flange legs, e.g., channels and S-beams

bitumen - a black sticky mixture of hydrocarbons obtained from natural deposits or from distilling petroleum; tar

bituminous concrete - a mixture of aggregate and liquid asphalt or bitumen, which is compacted into a dense mass

blanket - a streambed protection against scour placed adjacent to abutments and piers

BMS - Bridge Management System

bolt - a mechanical fastener with machine threads at one end to receive a nut, and an integral head at the other end

bolster - a block-like member used to support a bearing on top of a pier cap or abutment bridge seat; see PEDESTAL

bond - in reinforced concrete, the grip of the concrete on the reinforcing bars, which prevents slippage of the bars relative to the concrete mass

bond stress - a term commonly applied in reinforced concrete construction to the stress developed by a force tending to produce movement or slippage at the interface between the concrete and the reinforcement bars

bowstring truss - a general term applied to a truss of any type having a polygonal arrangement of its top chord members conforming to or nearly conforming to the arrangement required for a parabolic truss; a truss with a curved top chord

box beam - a hollow structural beam with a square, rectangular, or trapezoidal cross-section that supports vertical loads and provides torsional rigidity

box culvert - a culvert of rectangular or square cross-section

box girder - a hollow, rectangular or trapezoidal shaped girder, a primary member along the longitudinal axis of the bridge, which provides good torsional rigidity

bracing - a system of secondary members that maintains the geometric configuration of primary members

bracket - a projecting support fixed upon two intersecting members to strengthen and provide rigidity to the connection

breastwall - the portion of an abutment between the wings and beneath the bridge seat; the breast wall supports the superstructure loads, and retains the approach fill; see STEM

bridge - a structure spanning and providing passage over a river, chasm, road, or similar obstacle

bridge deficiency - a defect in a bridge component or member that makes the bridge less capable or less desirable for use

bridge pad - the raised, leveled area upon which the pedestal, masonry plate or other corresponding element of the superstructure bears on the substructure; also called bridge seat bearing area

bridge seat - the top surface of an abutment or pier upon which the superstructure span is placed and supported; for an abutment it is the surface forming the support for the superstructure and from which the backwall rises; for a pier it is the entire top surface

bridge site - the position or location of a bridge and its surrounding area

bridging - a carpentry term applied to the cross–bracing fastened between timber beams to increase the rigidity of the floor construction, limit differential deflection and minimize the effects of impact and vibration

brittle fracture - the failure of a steel member occurring without warning, prior to plastic deformation

brush curb - a narrow curb, 9 inches or less in width, which prevents a vehicle from brushing against the railing or parapet

buckle - to fail by an inelastic change in alignment (deflection) as a result of compression in axial loaded members

buckle plate - an obsolete style of steel deck using dished steel plates as structural members

built-up member - a column or beam composed of plates and angles or other structural shapes united by bolting, riveting or welding to enhance section properties

bulb t-girder - a t-shaped concrete girder with a bulb shape at the bottom of the girder cross section

bulkhead - a retaining wall-like structure commonly composed of driven sheet piles or a barrier of wooden timbers or reinforced concrete members

buoyancy - upward pressure exerted by the fluid in which an object is immersed

butt joint - a joint between two pieces of metal that have been connected in the same plane

buttress - a bracket-like wall, of full or partial height, projecting from another wall; the buttress strengthens and stiffens the wall against overturning forces; all parts of a buttress act in compression

buttressed wall - a retaining wall designed with projecting buttresses to provide strength and stability

butt weld - a weld joining two plates or shapes end to end; also splice weld

C

cable - a tension member comprised of numerous individual steel wires or strands twisted and wrapped in such a fashion to form a rope of steel; see SUSPENSION BRIDGE

cable band - a steel casting with clamp bolts which fixes a floor system suspender cable to the catenary cable of a suspension bridge

cable-stayed bridge - a bridge in which the superstructure is directly supported by cables, or stays, passing over or attached to towers located at the main piers

caddisfly - a winged insect closely related to the moth and butterfly whose aquatic larvae seek shelter by digging small shallow holes into submerged timber elements

caisson - a rectangular or cylindrical chamber for keeping water or soft ground from flowing into an excavation

camber - the slightly arched or convex curvature provided in beams to compensate for dead load deflection; in general, a structure built with perfectly straight lines appears slightly sagged

cantilever - a structural member that has a free end projecting beyond a support; length of span overhanging the support

cantilever abutment - an abutment that resists lateral earth pressure through the opposing cantilever action of a vertical stem and horizontal footing

cantilever bridge - a general term applying to a bridge having a superstructure incorporating cantilever design

cantilever span - a superstructure span composed of two cantilever arms, or of a suspended span supported by one or two cantilever arms

cap - the topmost portion of a pier or a pile bent serving to distribute the loads upon the columns or piles and to hold them in their proper relative positions; see PIER CAP, PILE CAP

cap beam - the top member in a bent that ties together the supporting members

capstone - the topmost stone of a masonry pillar, column or other structure requiring the use of a single capping element

carbon steel - steel (iron with dissolved carbon) owing its properties principally to its carbon content; ordinary, unalloyed steel

cast-in-place (C.I.P.) - the act of placing and curing concrete within formwork to construct a concrete element in its final position

cast iron - relatively pure iron, smelted from iron ore, containing 1.8 to 4.5% free carbon and cast to shape

catch basin - a receptacle, commonly box shaped and fitted with a grilled inlet and a pipe outlet drain, designed to collect the rainwater and floating debris from the roadway surface and retain the solid material so that it may be periodically removed

catchment area - see DRAINAGE AREA

catenary - the curve obtained by suspending a uniformly loaded rope or cable between two points

cathode - the negatively charged pole of a corrosion cell that accepts electrons and does not corrode

cathodic protection - a means of preventing metal from corroding by making it a cathode through the use of impressed direct current or by attaching a sacrificial anode

catwalk - a narrow walkway for access to some part of a structure

causeway - an elevated roadway crossing a body of water

cellular abutment - an abutment in which the space between wings, abutment stem, approach slab, and footings is hollow. Also known as a vaulted abutment

cement mortar - a mixture of sand and cement with enough water to make it plastic

cement paste - the plastic combination of cement and water that supplies the cementing action in concrete

centerline of bearings - a horizontal line that passes through the centers of the bearings, used in abutment/pier layout and beam erection

center of gravity - the point at which the entire mass of a body acts; the balancing point of an object

centroid - that point about which the static moment of all the elements of area is equal to zero

chain drag - a chain or a series of short medium weight chains attached to a T-shaped handle; used as a preliminary technique for sounding a large deck area for delamination

chamfer - an angled edge or corner, typically formed in concrete

channel - a waterway connecting two bodies of water or containing moving water; a rolled steel member having a C-shaped cross section

channel profile - a longitudinal section of a channel along its centerline

check - a crack in wood occurring parallel with the grain and through the rings of annual growth

cheek wall - see KNEE WALL

chipping hammer - hammer such as a geologist's pick or masonry hammer used to remove corrosion from steel members and to sound concrete for delamination; a welder's tool for cleaning slag from steel after welding

chloride - an ingredient in deicing agents that can damage concrete and steel bridge elements

chord - a generally horizontal member of a truss

circular arch - an arch in which the intrados surface has a constant radius

clearance - the unobstructed vertical or horizontal space provided between two objects

clear headroom - the vertical clearance beneath a bridge structure available for navigational use

clear span - the unobstructed space or distance between support elements of a bridge or bridge member

clip angle - see CONNECTION ANGLE

closed spandrel arch - a stone, brick or reinforced concrete arch span having spandrel walls to retain the spandrel fill or to support either entirely or in part the floor system of the structure when the spandrel is not filled

coarse aggregate - aggregate that stays on a sieve of 5 mm (¼") square opening

coating - a material that provides a continuous film over a surface in order to protect or seal it; a film formed by the material

coefficient of thermal expansion - the unit change in dimension produced in a material by a change of one degree in temperature

cofferdam - a temporary dam-like structure constructed around an excavation to exclude water; see SHEET PILE COFFERDAM

cold chisel - short bar with a sharp end used for cold-cutting soft metals when struck with a hammer

column - a general term applying to a vertical member resisting compressive stresses and having, in general, a considerable length in comparison with its transverse dimensions

column bent - a bent shaped pier that uses columns incorporated with a cap beam

compaction - the process by which a sufficient amount of energy (compressive pressure) is applied to soil or other material to increase its density

component - a general term reserved to define a bridge deck, superstructure or substructure

composite action - the contribution of a concrete deck to the moment resisting capacity of the superstructure beam when the superstructure beams are not the same material as the deck

composite construction - a method of construction whereby a cast-in-place concrete deck is mechanically attached to superstructure members by shear connectors

compression - a type of stress involving pressing together; tends to shorten a member; opposite of tension

compression failure - buckling, crushing, or collapse caused by compression stress

compression flange - the part of a beam that is compressed due to a bending moment

compression seal joint - a joint consisting of a neoprene elastic seal squeezed into the joint opening

concentrated load - a force applied over a small contact area; also known as point load

concrete - a stone-like mass made from a mixture of aggregates and cementing material, which is moldable prior to hardening; see BITUMINOUS CONCRETE and PORTLAND CEMENT CONCRETE

concrete beam - a structural member of reinforced concrete designed to carry bending loads

concrete pile - a pile constructed of reinforced concrete either precast and driven into the ground or cast-in-place in a hole bored into the ground

concrete tee team - "T" shaped section of reinforced concrete; cast-in-place monolithic deck and beam system

condition rating - a judgment of a bridge component condition in comparison to its original as-built condition

conductor - a material that is suitable for carrying electric current

connection angle - a piece of angle serving to connect two elements of a member or two members of a structure; also known as clip angle

consolidation - the time dependent change in volume of a soil mass under compressive load caused by water slowly escaping from the pores or voids of the soil

construction joint - a pair of adjacent surfaces in reinforced concrete where two pours have met, reinforcement steel extends through this joint

continuous beam - a general term applied to a beam that spans uninterrupted over one or more intermediate supports

continuous bridge - a bridge designed to extend without joints over one or more interior supports

continuous footing - a common footing that is underneath a wall, or columns

continuous span - spans designed to extend without joints over one or more intermediate supports

continuous truss - a truss without hinges having its chord and web members arranged to continue uninterrupted over one or more intermediate points of support

continuous weld - a weld extending throughout the entire length of a connection

contraction - the thermal action of the shrinking of an object when cooled; opposite of expansion

coping - a course of stone laid with a projection beyond the general surface of the masonry below it and forming the topmost portion of a wall; a course of stone capping the curved or V-shaped extremity of a pier, providing a transition to the pier head proper, when so used it is commonly termed the "starling coping," "nose coping," the "cutwater coping" or the "pier extension coping"

corbel - a piece constructed to project from the surface of a wall, column or other portion of a structure to serve as a support for another member

core - a cylindrical sample of concrete or timber removed from a bridge component for the purpose of destructive testing to determine the condition of the component

corrosion - the general disintegration of metal through oxidation

corrugated - an element with alternating ridges and valleys

counter - a truss web member that undergoes stress reversal and resists only live load tension; see WEB MEMBERS

counterfort - a bracket-like wall connecting a retaining wall stem to its footing on the side of the retained material to stabilize the wall against overturning; a counterfort, as opposed to a buttress, acts entirely in tension

counterforted abutment - an abutment that develops resistance to bending moment in the stem by use of counterforts. This permits the breast wall to be designed as a horizontal beam or slab spanning between counterforts, rather than as a vertical cantilever slab

counterforted wall - a retraining wall designed with projecting counterforts to provide strength and stability

counterweight - a weight which is used to balance the weight of a movable member; in bridge applications counterweights are used to balance a movable span so that it rotates or lifts with minimum resistance. Also sometimes used in continuous structures to prevent uplift

couplant - a viscous fluid material used with ultrasonic gages to enhance transmission of sound waves

couple - two forces that are equal in magnitude, opposite in direction, and parallel with respect to each other

coupon - a sample of steel taken from an element in order to test material properties

course - a horizontal layer of bricks or stone

cover - the clear thickness of concrete between a reinforcing bar and the surface of the concrete; the depth of backfill over the top of a pipe or culvert

covered bridge - an indefinite term applied to a wooden bridge having its roadway protected by a roof and enclosing sides

cover plate - a plate used in conjunction with a flange or other structural shapes to increase flange section properties in a beam, column, or similar member

crack - a break without complete separation of parts; a fissure

cracking (reflection) - visible cracks in an overlay indicating cracks in the concrete underneath

crack initiation - the beginning of a crack usually at some microscopic defect

crack propagation - the growth of a crack due to energy supplied by repeated stress cycles

creep - an inelastic deformation that occurs under a constant load, below the yield point, and increases with time

creosote - an oily liquid obtained by the distillation of coal or wood tar and used as a wood preservative

crib - a structure consisting of a foundation grillage combined with a superimposed framework providing compartments or coffers which are filled with gravel, concrete or other material satisfactory for supporting the structure to be placed thereon

cribbing - a construction consisting of wooden, metal or reinforced concrete units so assembled as to form an open cellular-like structure for supporting a superimposed load or for resisting horizontal or overturning forces acting against it.

cribwork - large timber cells that are submerged full of concrete to make an underwater foundation

cross - transverse bracings between two main longitudinal members; see DIAPHRAGM, BRACING

cross frame - steel elements placed in "X" shaped patterns to act as stiffeners between the main carrying superstructure members

cross girders - transverse girders, supported by bearings, which support longitudinal beams or girders

cross-section - the shape of an object cut transversely to its length

cross-sectional area - the area of a cross-section

crown - the highest point of the transverse cross section of a roadway, pipe or arch; also known as soffit or vertex

crown of roadway - the vertical dimension describing the total amount the surface is convexed or raised from gutter to centerline; this is sometimes termed the cross fall or cross slope of roadway

culvert - a drainage structure beneath an embankment (e.g., corrugated metal pipe, concrete box culvert)

curb - a low barrier at the side limit of the roadway used to guide the movement of vehicles

curb inlet - see SCUPPER

curtain wall - a term commonly applied to a thin wall between main columns designed to withstand only secondary loads. Also the wall portion of a buttress or counterfort abutment that spans between the buttresses or counterforts

curvature - the degree of curving of a line or surface

curved girder - a girder that is curved in the horizontal plane in order to adjust to the horizontal alignment of the bridge

cutoff wall - vertical wall at the end of an apron or slab to prevent scour undermining

cutwater - a sharp-edged structure, facing the water channel current, built around a bridge pier to protect if

from the flow of water and debris in the water

cyclic stress - stress that varies with the passage of live loads; see STRESS RANGE

D

dead load - a static load due to the weight of the structure itself

debris - material including floating wood, trash, suspended sediment or bed load moved by a flowing stream

deck - that portion of a bridge which provides direct support for vehicular and pedestrian traffic, supported by a superstructure

deck arch - an arch bridge with the deck above the top of the arch

deck bridge - a bridge in which the supporting members are all beneath the roadway

decking - bridge flooring installed in panels, e.g., timber planks

deck joint - a gap allowing for rotation or horizontal movement between two spans or an approach and a span

deficiency - see BRIDGE DEFICIENCY

deflection - elastic movement of a structural member under a load

deformation - distortion of a loaded structural member; may be elastic or inelastic

deformed bars - concrete reinforcement consisting of steel bars with projections or indentations (deformations) to increase the mechanical bond between the steel and concrete

degradation - general progressive lowering of a stream channel by scour

delamination - surface separation of concrete into layers; separation of glulaminated timber plies

design load - the force for which a structure is designed; the most severe combination of loads

deterioration - decline in quality over a period of time due to chemical or physical degradation

diagonal - a sloping structural member of a truss or bracing system

diagonal stay - a cable support in a suspension bridge extending diagonally from the tower to the roadway to add stiffness to the structure and diminish the deformations and undulations resulting from traffic service

diagonal tension - the tensile force due to horizontal and vertical shear in a beam

diaphragm - a transverse member placed within a member or superstructure system to distribute stresses and improves strength and rigidity; see BRACING

diaphragm wall - a wall built transversely to the longitudinal centerline of a spandrel arch serving to tie together and reinforce the spandrel walls, together with providing a support for the floor system in conjunction with the spandrel walls; also known as cross wall

differential settlement - uneven settlement of individual or independent elements of a substructure; tilting in the longitudinal or transverse direction due to deformation or loss of foundation material

dike - an earthen embankment constructed to retain or redirect water; when used in conjunction with a bridge, it prevents stream erosion and localized scour and/or so directs the stream current such that debris does not accumulate; see SPUR

discharge - the volume of fluid per unit of time flowing along a pipe or channel

displacement induced stress - stresses caused by differential deflection of adjacent parts

distributed load - a load uniformly applied along the length of an element or component of a bridge

ditch - a trough-like excavation made to collect water

diver - a specially trained individual who inspects the underwater portion of a bridge substructure and the surrounding channel

dolphin - a group of piles driven close together or a caisson placed to protect portions of a bridge exposed to possible damage by collision with river or marine traffic

double movable bridge - a bridge in which the clear span over the navigation channel is produced by joining the arms of two adjacent swing spans or the leaves of two adjacent bascule spans at or near the center of the navigable channel; see MOVABLE BRIDGE

dowel - a length of bar embedded in two parts of a structure to hold the parts in place and to transfer stress

drainage - a system designed to remove water from a structure

drainage area - an area in which surface run-off collects and from which it is carried by a drainage system; also known as catchment area

drain hole - hole in a box shaped member or a wall to provide means for the exit of accumulated water or other liquid; also known as drip hole; see WEEP HOLE

drain pipes - pipes that carry storm water

drawbridge - a general term applied to a bridge over a navigable body of water having a movable superstructure span of any type

drift bolt - a short length of metal bar used to connect and hold in position wooden members placed in contact; similar to a dowel

drift pin - tapered steel rod used by ironworkers to align bolt holes

drip notch - a recess cast on the underside of an overhang that prevents water from following the concrete surface onto the supporting beams

drop inlet - a type of inlet structure that conveys the water from a higher elevation to a lower outlet elevation smoothly without a free fall at the discharge

duct - the hollow space where a prestressing tendon is placed in a post-tensioned prestressed concrete girder

ductile - capable of being molded or shaped without breaking; plastic

ductile fracture - a fracture characterized by plastic deformation

ductility - the ability to withstand non–elastic deformation without rupture

dumbbell pier - a pier consisting of two cylindrical or rectangular shaped piers joined by an integral web

dummy member - truss member that carries no primary loads; may be included for bracing or for appearance

E

E - modulus of elasticity of a material; Young's modulus; the stiffness of a material

efflorescence - a deposit on concrete or brick caused by crystallization of carbonates brought to the surface by moisture in the masonry or concrete

elastic - capable of sustaining deformation without permanent loss of shape

elastic deformation - non-permanent deformation; when the stress is removed, the material returns to its original shape

elasticity - the property whereby a material changes its shape under the action of loads but recovers its original shape when the loads are removed

elastomer - a natural or synthetic rubber-like material

elastomeric pad - a synthetic rubber pad used in bearings that compresses under loads and accommodates horizontal movement by deforming

electrolyte - a medium of air, soil, or liquid carrying ionic current between two metal surfaces, the anode and the cathode

electrolytic cell - a device for producing electrolysis consisting of the electrolyte and the electrodes

electrolytic corrosion - corrosion of a metal associated with the flow of electric current in an electrolyte

elevation view - a drawing of the side view of a structure

elliptic arch - an arch in which the intrados surface is a full half of the surface of an elliptical cylinder; this terminology is sometimes incorrectly applied to a multicentered arch

elongation - the elastic or plastic extension of a member

embankment - a mound of earth constructed above the natural ground surface to carry a road or to prevent water from passing beyond desirable limits; also known as bank

end block - in a prestressed concrete I-beam, the widened beam web at the end to provide adequate anchorage bearing for the post tensioning steel and to resist high shear stresses; similarly, the solid end diaphragm of a box beam

end post - the end compression member of a truss, either vertical or inclined in position and extending from top chord to bottom chord

end section - a concrete or steel appurtenance attached to the end of a culvert for the purpose of hydraulic efficiency, embankment retention or anchorage

end span - a span adjacent to an abutment

epoxy - a synthetic resin which cures or hardens by chemical reaction between components which are mixed together shortly before use

epoxy coated reinforcement - reinforcement steel coated with epoxy; used to prevent corrosion

equilibrium - in statics, the condition in which the forces acting upon a body are such that no external effect (or movement) is produced

equivalent uniform load - a load having a constant intensity per unit of its length producing an effect equal to that of a live load consisting of vehicle axle or wheel concentrations spaced at varying distances

erosion - wearing away of soil by flowing water not associated with a channel; see SCOUR

expansion - an increase in size or volume

expansion bearing - a bearing designed to permit longitudinal or lateral movements resulting from temperature changes and superimposed loads with minimal transmission of horizontal force to the substructure; see BEARING

expansion dam - the part of an expansion joint serving as an end form for the placing of concrete at a joint; also applied to the expansion joint device itself; see EXPANSION JOINT

expansion joint - a joint designed to permit expansion and contraction movements produced by temperature changes, loadings or other forces

expansion rocker - a bearing device at the expansion end of a beam or truss that allows the longitudinal

movements resulting from temperature changes and superimposed loads through a tilting motion

expansion roller - a cylinder so mounted that by revolution it facilitates expansion, contraction or other movements resulting from temperature changes, loadings or other forces

expansion shoe - expansion bearing, generally of all metal construction

exterior girder - an outermost girder supporting the bridge floor

extrados - the curve defining the exterior (upper) surface of an arch; also known as back

eyebar - a member consisting of a rectangular bar with enlarged forged ends having holes for engaging connecting pins

F

failure - a condition at which a structure reaches a limit state such as cracking or deflection where it is no longer able to perform its usual function; collapse; fracture

falsework - a temporary wooden or metal framework built to support the weight of a structure during the period of its construction and until it becomes self-supporting

fascia - an outside, covering member designed on the basis of architectural effect rather than strength and rigidity, although its function may involve both

fascia girder - an exposed outermost girder of a span sometimes treated architecturally or otherwise to provide an attractive appearance

fatigue - the tendency of a member to fail at a stress below the yield point when subjected to repetitive loading

fatigue crack - any crack caused by repeated cyclic loading at a stress below the yield point

fatigue damage - member damage (crack formation) due to cyclic loading

fatigue life - the length of service of a member subject to fatigue, based on the number of cycles it can undergo

fender - a structure that acts as a buffer to protect the portions of a bridge exposed to floating debris and water-borne traffic from collision damage; sometimes called an ice guard in regions with ice floes

fender pier - a pier-like structure which performs the same service as a fender but is generally more substantially built; see GUARD PIER

field coat - a coat of paint applied after the structure is assembled and its joints completely connected; quite commonly a part of the field erection procedure; field painting

fill - material, usually earth, used to change the surface contour of an area, or to construct an embankment

filler - a piece used primarily to fill a space beneath a batten, splice plate, gusset, connection angle, stiffener or other element; also known as filler plate

filler metal - metal prepared in wire, rod, electrode or other form to be fused with the structure metal in the formation of a weld

filler plate - see FILLER

fillet - a curved portion forming a junction of two surfaces that would otherwise intersect at an angle

fillet weld - a weld of triangular or fillet shaped cross–section between two pieces at right angles

filling - see FILL

fine aggregate - sand or grit for concrete or mortar that passes a No. 4 sieve (4.75 mm)

finger dam - expansion joint in which the opening is spanned by meshing steel fingers or teeth

fish belly - a term applied to a girder or a truss having its bottom flange or its bottom chord constructed either haunched or bow–shaped with the convexity downward; see LENTICULAR TRUSS

fixed beam - a beam with a fixed end

fixed bearing - a bearing that allows only rotational movement; see BEARING

fixed bridge - a bridge having constant position, i.e., without provision for movement to create increased navigation clearance

fixed end - movement is restrained

fixed-ended arch - see VOUSSOIR ARCH

fixed span - a superstructure span having its position practically immovable, as compared to a movable span

fixed support - a support that will allow rotation only, no longitudinal movement

flange - the (usually) horizontal parts of a rolled I-shaped beam or of a built-up girder extending transversely across the top and bottom of the web

flange angle - an angle used to form a flange element of a built-up girder, column, strut or similar member

floating bridge - see PONTOON BRIDGE

floating foundation - used to describe a soil-supported raft or mat foundation with low bearing pressures; sometimes applied to a "foundation raft" or "foundation grillage"

flood frequency - the average time interval in years in which a flow of a given magnitude will recur

flood plain - area adjacent to a stream or river subject to flooding

floor - see DECK

floorbeam - a primary horizontal member located transversely to the general bridge alignment

floor system - the complete framework of members supporting the bridge deck and the traffic loading

flow capacity - maximum flow rate that a channel, conduit, or culvert structure is hydraulically capable of carrying

flux - a material that protects the weld from oxidation during the fusion process

footbridge - a bridge designed and constructed to provide means of traverse for pedestrian traffic only; also known as pedestrian bridge

footing - the enlarged, lower portion of a substructure, which distributes the structure load either to the earth or to supporting piles; the most common footing is the concrete slab; footer is a colloquial term for footing

foot wall - see TOE WALL

force - an influence that tends to accelerate a body or to change its movement

forms - the molds that hold concrete in place while it is hardening; also known as form work, shuttering; see LAGGING, STAY-IN-PLACE FORMS

form work - see FORMS

foundation - the supporting material upon which the substructure portion of a bridge is placed

foundation excavation - the excavation made to accommodate a footing for a structure; also known as foundation pit

foundation failure - failure of a foundation by differential settlement or by shear failure of the soil

foundation grillage - a construction consisting of steel, timber, or concrete members placed in layers; each layer is perpendicular to those above and below it and the members within a layer are generally parallel, producing a crib or grid-like effect. Grillages are usually placed under very heavy concentrated loads

foundation load - the load resulting from traffic, superstructure, substructure, approach embankment, approach causeway, or other incidental load increment imposed upon a given foundation area

foundation pile - see PILE

foundation pit - see FOUNDATION EXCAVATION

foundation seal - a mass of concrete placed underwater within a cofferdam for the base portion of structure to close or seal the cofferdam against incoming water; see TREMIE

fracture - see BRITTLE FRACTURE

fracture critical member - a member in tension or with a tension element whose failure would probably cause a portion of or the entire bridge to collapse

frame - a structure which transmits bending moments from the horizontal beam member through rigid joints to vertical or inclined supporting members

framing - the arrangement and connection of the component members of a bridge superstructure

free end - movement is not restrained

friction pile - a pile that provides support through friction resistance between the pile and the surrounding earth along the lateral surface of the pile

friction roller - a roller placed between members intended to facilitate change in their relative positions by reducing the frictional resistance to translation movement

frost heave - the upward movement of, or force exerted by, soil due to freezing of retained moisture

frost line - the depth to which soil may be frozen

G

gabion - rock filled wire baskets used to retain earth and provide erosion control

galvanic action - electrical current between two unlike metals

galvanize - to coat with zinc

gauge - the distance between parallel lines of rails, rivet holes, etc; a measure of thickness of sheet metal or wire; also known as gage

geometry - shape or form; relationship between lines or points

girder - a horizontal flexural member that is the main or primary support for a structure; any large beam, especially if built up

girder bridge - a bridge whose superstructure consists of two or more girders supporting a separate floor system as differentiated from a multi-beam bridge or a slab bridge

girder span - a span in which the major longitudinal supporting members are girders

glue laminated - a member created by gluing together two or more pieces of lumber

grade - the fall or rise per unit horizontal length; see GRADIENT

grade crossing - a term applicable to an intersection of two highways, two railroads or a railroad and a highway at a common grade or elevation; now commonly accepted as meaning the last of these combinations

grade intersection - the location where two roadway slopes meet in profile; to provide a smooth transition from one to the other they are connected by a vertical curve and the resulting profile is a sag or a crest

grade separation - roadways crossing each other at different elevations; see OVERPASS, UNDERPASS

gradient - the rate of inclination of the roadway and/or sidewalk surface(s) from the horizontal, applying to a bridge and its approaches; it is commonly expressed as a percentage relation (ratio) of horizontal to vertical dimensions

gravity abutment - a thick abutment that resists horizontal earth pressure through its own dead weight

gravity wall - a retaining wall that is prevented from overturning or sliding by its own dead weight

grid flooring - a steel floor system comprising a lattice pattern that may or may not be filled with concrete

grillage - assembly of parallel beams, usually steel or concrete, placed side by side, often in layers with alternating directions; see FOUNDATION GRILLAGE

groin - a wall built out from a river bank to check scour

grout - mortar having a sufficient water content to render it free-flowing, used for filling (grouting) the joints in masonry, for fixing anchor bolts and for filling cored spaces; usually a thin mix of cement, water and sometimes sand or admixtures

grouting - the process of filling in voids with grout

guard pier - a pier-like structure built to protect a swing span in its open position from collision with passing vessels or water-borne debris; may be equipped with a rest pier upon which the swing span in its open position may be latched; see FENDER PIER

guardrail - a safety feature element intended to redirect an errant vehicle

guide rail - see GUARDRAIL

gunite - the process of blowing Portland cement mortar or concrete onto a surface using compressed air

gusset plate - a plate that connects the members of a structure and holds them in correct position at a joint

gutter - a paved ditch; area adjacent to a roadway curb used for drainage

guy - a cable member used to anchor a structure in a desired position

H

H Loading - a combination of loads used to represent a two-axle truck developed by AASHTO

hairline cracks - very narrow cracks that form in the surface of concrete due to tension caused by loading

hammer - hand tool used for sounding and surface inspection

hammerhead pier - a pier with a single cylindrical or rectangular shaft and a relatively long, transverse cap; also known as a tee pier or cantilever pier

hand hole - hole provided in component plate of built-up box section to permit access to the interior for construction and maintenance purposes

hand rail - commonly applies only to sidewalk railing presenting a latticed, barred, balustered or other open web construction

hands-on access - close enough to the member or component so that it can be touched with the hands and inspected visually

hanger - a tension member serving to suspend an attached member; allows for expansion between a cantilevered and suspended span

haunch - an increase in the depth of a member usually at points of support; the outside areas of a pipe between the spring line and the bottom of the pipe

haunched girder - a horizontal beam whose cross sectional depth varies along its length

H-beam - a rolled steel member having an H-shaped cross-section (flange width equals beam depth) commonly used for piling; also H-pile

head - a measure of water pressure expressed in terms of an equivalent weight or pressure exerted by a column of water; the height of the equivalent column of water is the head

head loss - the loss of energy between two points along the path of a flowing fluid due to fluid friction; reported in feet of head

headwall - a concrete structure at the ends of a culvert to retain the embankment slopes, anchor the culvert, and prevent undercutting

headwater - the source or the upstream waters of a stream

heat treatment - any of a number of various operations involving controlled heating and cooling that are used to impart specific properties to metals; examples are tempering, quenching, and annealing

heave - the upward motion of soil caused by outside forces such as excavation, pile driving, moisture or soil expansion; see FROST HEAVE

heel - the portion of a footing behind the stem

helical - having the form of a spiral

high carbon steel - carbon steel containing 0.5 to 1.5% dissolved carbon

high strength bolt - bolt and nut made of high strength steel, usually A325 or A490

hinge - a point in a structure at which a member is free to rotate

hinged joint - a joint constructed with a pin, cylinder segment, spherical segment or other device permitting rotational movement

honeycomb - an area in concrete where mortar has separated and left spaces between the coarse aggregate, usually caused by improper vibration during concrete construction

horizontal alignment - a roadway's centerline or baseline alignment in the horizontal plane

horizontal curve - a roadway baseline or centerline alignment defined by a radius in the horizontal plane

Howe truss - a truss of the parallel chord type with a web system composed of vertical (tension) rods at the panel points with an X pattern of diagonals

HS Loading - a combination of loads developed by AASHTO used to represent a truck and trailer

hybrid girder - a girder whose flanges and web are made from steel of different grades

hydraulics - the mechanics of fluids

hydrology - study of the accumulation and flow of water from watershed areas

hydroplaning - loss of contact between a tire and the roadway surface when the tire planes or glides on a film of water

I

I-beam - a structural member with a cross–sectional shape similar to the capital letter "I"

ice guard - see FENDER

impact - A factor that describes the effect on live load due to dynamic and vibratory effects of a moving load; in bridge design, a load based on a percentage of live load to include dynamic and vibratory effects; in fracture mechanics, a rapidly applied load, such as a collision or explosion

incomplete fusion - a weld flaw where the weld metal has not combined metallurgically with the base metal

indeterminate stress - stress in a structural member which cannot be calculated directly; it is computed by the iterative application of mathematical equations, usually with an electronic computer; indeterminate stresses arise in continuous span and frame type structures

individual column footing - footing supporting one column

inelastic compression - compression beyond the yield point

inlet - an opening in the floor of a bridge leading to a drain; roadway drainage structure which collects surface water and transfers it to pipes

inspection frequency - the frequency with which the bridge is inspected -- normally every two years

integral abutment - an abutment cast monolithically with the end diaphragm of the deck; such abutments usually encase the ends of the deck beams and are pile supported

integral deck - a deck which is monolithic with the superstructure; concrete tee beam bridges have integral decks

intercepting ditch - a ditch constructed to prevent surface water from flowing in contact with the toe of an embankment or causeway or down the slope of a cut

interior girder - any girder between exterior or fascia girders

interior span - a span of which both supports are intermediate substructure units

intermittent weld - a noncontinuous weld commonly composed of a series of short welds separated by spaces of equal length

intrados - the curve defining the interior (lower) surface of the arch; also known as soffit

inventory item - data contained in the structure file pertaining to bridge identification, structure type and material, age and service, geometric data, navigational data, classification, load rating and posting, proposed improvements, and inspections

inventory rating - the capacity of a bridge to withstand loads under normal service conditions based on 55% of yield strength

invert elevation - the bottom or lowest point of the internal surface of the transverse cross section of a pipe or culvert

iron - a metallic element used in cast iron, wrought iron and steel

isotropic - having the same material properties in all directions, e.g., steel

J

jack arch - a deck support system comprised of a brick or concrete arch springing from the bottom flanges of adjacent rolled steel beams

jacking - the lifting of elements using a type of jack (e.g., hydraulic), sometimes acts as a temporary support system

jack stringer - the outermost stringer supporting the bridge floor in a panel or bay

jacket - a protective shell surrounding a pile made of fabric, concrete or other material

jersey barrier - a concrete barrier with sloping front face that was developed by the New Jersey Department of Transportation

joint - in masonry, the space between individual stones or bricks; in concrete, a division in continuity of the concrete; in a truss, point at which members of a truss are joined

K

keeper plate - a plate, which is connected to a sole plate, designed to prohibit a beam from becoming dislodged from the bearing

key - a raised portion of concrete on one face of a joint that fits into a depression on the adjacent face

keystone - the symmetrically shaped, wedge–like stone located in a head ring course at the crown of an arch; the final stone placed, thereby closing the arch

king-post - the vertical member in a "king-post" type truss; also known as king rod

king-post truss - two triangular panels with a common center vertical; the simplest of triangular system trusses

kip - a kilo pound (1000 lb.); convenient unit for structural calculations

knee brace - a short member engaging at its ends two other members that are joined to form a right angle or a near-right angle to strengthen and stiffen the connecting joint

knee wall - a return of the abutment backwall at its ends to enclose the bridge seat on three of its sides; also called cheek wall

knife edge - a condition in which corrosion of a steel member has caused a sharp edge

knuckle - an appliance forming a part of the anchorage of a suspension bridge main suspension member permitting movement of the anchorage chain

K-truss - a truss having a web system wherein the diagonal members intersect the vertical members at or near

the mid-height; the assembly in each panel forms a letter "K"

L

L-abutment - a cantilever abutment with the stem flush with the toe of the footing, forming an "L" in cross section

laced column - a riveted, steel built-up column of usually four angles or two channels tied together laterally with lacing

lacing - small flat plates, usually with one rivet at each end, used to tie individual sections of built up members; see LATTICE

lagging - horizontal members spanning between piles to form a wall; forms used to produce curved surfaces; see FORMS

lamellar tear - incipient cracking parallel to the face of a steel member

laminated timber - timber planks glued together face to face to form a larger member; see GLUE LAMINATED

lane loading - a design loading which represents a line of trucks crossing over a bridge

lap joint - a joint between two members in which the end of one member overlaps the end of the other

lateral - a member placed approximately perpendicular to a primary member

lateral bracing - the bracing assemblage engaging a member perpendicular to the plane of the member; intended to resist transverse movement and deformation; also keeps primary parallel elements in truss bridges and girder bridges aligned; see BRACING

lattice - a crisscross assemblage of diagonal bars, channels, or angles on a truss; also known as latticing, lacing

lattice truss - in general, a truss having its web members inclined but more commonly the term is applied to a truss having two or more web systems composed entirely of diagonal members at any interval and crossing each other without reference to vertical members

leaching - the action of removing substances from a material by passing water through it

lead line - a weighted cord incrementally marked, used to determine the depth of a body of water; also known as sounding line

leaf - the movable portion of a bascule bridge that forms the span of the structure

lenticular truss - a truss having parabolic top and bottom chords curved in opposite directions with their ends meeting at a common joint; also known as a fish belly truss

levee - an embankment built to prevent flooding of low-lying land

leveling course - a layer of bituminous concrete placed to smooth an irregular surface

light-weight concrete - concrete of less than standard unit weight; may be no-fines concrete, aerated concrete, or concrete made with lightweight aggregate

link - a hanger plate in a pin and hanger assembly whose shape is similar to an eyebar, e.g., the head (at the pinhole) is wider than the shank

link and roller - a movable bridge element consisting of a hinged strut–like link fitted with a roller at its bottom end, supported upon a shoe plate or pedestal and operated by a thrust strut serving to force it into a vertical position and to withdraw it therefrom; when installed at each outermost end of the girders or the trusses of a swing span their major function is to lift them to an extent that their camber or droop will be removed and the arms rendered free to act as simple spans; when the links are withdrawn to an inclined position fixed by the operating mechanism the span is free to be moved to an open position

live load - a temporary dynamic load such as vehicular traffic that is applied to a structure; also accompanied by vibration or movement affecting its intensity

load - a force carried by a structure component

load factor design - a design method used by AASHTO, based on limit states of material and arbitrarily increased loads

load indicating washer - a washer with small projections on one side, which compress as the bolt is tightened; gives a direct indication of the bolt tension that has been achieved

load rating - calculation of the amount of load a bridge can carry based on actual member size and condition

load and resistance factor design (LRFD) - design method used by AASHTO, based on limit states of material with increased loads and reduced member capacity based on statistical probabilities

local buckling - localized buckling of a beam's plate element, can lead to failure of member

longitudinal bracing - bracing that runs lengthwise with a bridge and provides resistance against longitudinal movement and deformation of transverse members

loss of prestress - loss of prestressing force due to a variety of factors, including shrinkage and creep of the concrete, creep of the prestressing tendons, and loss of bond

low-carbon steel - steel with 0.04 to 0.25% dissolved carbon; also called mild steel

lower chord - the bottom horizontal member of a truss

luminaire - a lighting fixture

M

macadam - roadway pavement made with crushed stone aggregate, of coarse open gradation, compacted in place; asphaltic macadam included aspahlt as a binder

main beam - a horizontal structural member which supports the span and bears directly on a column or wall

maintenance - basic repairs performed on a facility to keep it at an adequate level of service

maintenance and protection of traffic - the management of vehicular and pedestrian traffic through a construction zone to ensure the safety of the public and the construction workforce; MPT; TRAFFIC PROTECTION

marine borers - mollusks and crustaceans that live in water and destroy wood by digesting it

masonry - that portion of a structure composed of stone, brick or concrete block placed in courses and usually cemented with mortar

masonry cement - Portland cement and lime used to make mortar for masonry construction

masonry plate - a steel plate placed on the substructure to support a superstructure bearing and to distribute the load to the masonry beneath

mattress - a flexible scour protection blanket composed of interconnected timber, gabions, or concrete units.

meander - a twisting, winding action from side to side; characterizes the serpentine curvature of a narrow, slow flowing stream in a wide flood plain

median - separation between opposing lanes of highway traffic; also known as median strip

member - an individual angle, beam, plate, or built component piece intended ultimately to become an integral part of an assembled frame or structure

metal corrosion - oxidation of metal by electro-galvanic action involving an electrolyte (moisture), an anode (the metallic surface where oxidation occurs), a cathode (the metallic surface that accepts electrons and does not corrode), and a conductor (the metal piece itself)

midspan - a reference point half–way between the supports of a beam or span

mild steel - steel containing from 0.04 to 0.25% dissolved carbon; see LOW CARBON STEEL

military loading - a loading pattern used to simulate heavy military vehicles passing over a bridge

mill scale - dense iron oxide on iron or steel that forms on the surface of metal that has been forged or hot worked

modular joint - a bridge joint designed to handle large movements consisting of an assembly of several strip

or compression seals

moisture content - the amount of water in a material expressed as a percent by weight

moment - the couple effect of forces about a given point; see BENDING MOMENT

monolithic - forming a single mass without joints

mortar - a paste of portland cement, sand, and water laid between bricks, stones or blocks

movable bridge - a bridge having one or more spans capable of being raised, turned, lifted, or slid from its normal service location to provide a clear navigation passage; see BASCULE BRIDGE, VERTICAL LIFT BRIDGE, PONTOON BRIDGE, RETRACTILE DRAW BRIDGE, ROLLING LIFT BRIDGE, and SWING BRIDGE

movable span - a general term applied to a superstructure span designed to be swung, lifted or otherwise moved longitudinally, horizontally or vertically, usually to provide increased navigational clearance

moving load - a live load which is moving, for example, vehicular traffic

MPT - see MAINTENANCE AND PROTECTION OF TRAFFIC

MSE - mechanically stabilized earth; see REINFORCED EARTH

multi-centered arch - an arch in which the intrados surface is outlined by two or more arcs symmetrically arranged and having different radii that intersect tangentially

N

nail laminated - a laminated member produced by nailing two or more pieces of timber together face to face

NBIS - National Bridge Inspection Standards, first established in 1971 to set national policy regarding bridge inspection frequency, inspector qualifications, report formats, and inspection and rating procedures

NCHRP - National Cooperative Highway Research Program

NDE - nondestructive evaluation

NDT - nondestructive testing; any testing method of checking structural quality of materials that does not damage them

necking - the elongation and contraction in area that occurs when a ductile material is stressed

negative bending - bending of a member that causes tension in the surface adjacent to the load, e.g., moment at interior supports of a span or at the joints of a frame

negative moment - bending moment in a member such that tension stresses are produced in the top portions

of the member; typically occurs in continuous beams and spans over the intermediate supports

neoprene - a synthetic rubber-like material used in expansion joints and elastomeric bearings

neutral axis - the internal axis of a member in bending along which the strain is zero; on one side of the neutral axis the fibers are in tension, on the other side the fibers are in compression

nose - a projection acting as a cut water on the upstream end of a pier; see STARLING

notch effect - stress concentration caused by an abrupt discontinuity or change in section

O

offset - a horizontal distance measured at right angles to a survey line to locate a point off the line

on center - a description of a typical dimension between the centers of the objects being measured

open spandrel arch - a bridge that has open spaces between the deck and the arch members allowing "open" visibility through the bridge

open spandrel ribbed arch - a structure in which two or more comparatively narrow arch rings, called ribs, function in the place of an arch barrel; the ribs are rigidly secured in position by arch rib struts located at intervals along the length of the arch; the arch ribs carry a column type open spandrel construction which supports the floor system and its loads

operating rating - the capacity of a bridge to withstand loads based on 75% of yield strength

operator's house - the building containing control devices required for opening and closing a movable bridge span

orthotropic - having different properties in two or more directions at right angles to each other (e.g., wood); see ANISOTROPY

outlet - in hydraulics, the discharge end of drains, sewers, or culverts

out-of-plane distortion - distortion of a member in a plane other than that which the member was designed to resist

overlay - see WEARING SURFACE

overload - a weight greater than the structure is designed to carry

overpass - bridge over a roadway or railroad

overturning - tipping over; rotational movement

oxidation - the chemical breakdown of a substance due to its reaction with oxygen from the air

oxidized steel - rust

P

pack - a steel plate inserted between two others to fill a gap and fit them tightly together; also known as packing; fill; filler plate

pack rust - rust forming between adjacent steel surfaces in contact which tends to force the surfaces apart due to the increase in material volume

paddleboard - striped, paddle-shaped signs or boards placed on the roadside in front of a narrow bridge as a warning of reduced roadway width

panel -the portion of a truss span between adjacent points of intersection of web and chord members

panel point - the point of intersection of primary web and chord members of a truss

parabolic arch - an arch in which the intrados surface is a segment of a symmetrical parabolic surface (suited to concrete arches)

parabolic truss - a polygonal truss having its top chord and end post vertices coincident with the arc of a parabola, its bottom chord straight and its web system either triangular or quadrangular; also known as a parabolic arched truss

parapet - a low wall along the outmost edge of the roadway of a bridge to protect vehicles and pedestrians

pedestal - concrete or built-up metal member constructed on top of a bridge seat for the purpose of providing a specific bearing seat elevation

pedestal pier - one or more piers built in block-like form that may be connected by an integrally built web between them; when composed of a single, wide block–like form, it is called a wall or solid pier

pedestrian bridge - see FOOT BRIDGE

penetration - when applied to creosoted lumber, the depth to which the surface wood is permeated by the creosote oil; when applied to pile driving; the depth a pile tip is driven into the ground

physical testing - the testing of bridge members in the field or laboratory

pier - a substructure unit that supports the spans of a multi-span superstructure at an intermediate location between its abutments

pier cap - the topmost horizontal portion of a pier that distributes loads from the superstructure to the vertical pier elements

pile - a shaft-like linear member which carries loads to underlying rock or soil strata

pile bent - a row of driven or placed piles extending above the ground surface supporting a pile cap; see BENT

pile bridge - a bridge carried on piles or pile bents

pile cap - a slab or beam which acts to secure the piles in position laterally and provides a bridge seat to receive and distribute superstructure loads

pile foundation - a foundation supported by piles in sufficient number and to a depth adequate to develop the bearing resistance required to support the substructure load

pile pier - see PILE BENT

piling - collective term applied to group of piles in a construction; see PILE, SHEET PILES

pin - a cylindrical bar used to connect elements of a structure

pin-connected truss - a general term applied to a truss of any type having its chord and web members connected at each panel point by a single pin

pin and hanger - a hinged connection detail designed to allow for expansion and rotation between a cantilevered and suspended span at a point between supports.

pin joint - a joint in a truss or other frame in which the members are assembled upon a single cylindrical pin

pin packing - arrangement of truss members on a pin at a pinned joint

pin plate - a plate rigidly attached upon the end of a member to develop the desired bearing upon a pin or pin-like bearing, and secure additional strength and rigidity in the member; doubler plate

pintle - a relatively small steel pin engaging the rocker of an expansion bearing, in a sole plate or masonry plate, thereby preventing sliding of the rocker

pipe - a hollow cylinder used for the conveyance of water, gas, steam etc.

piping - removal of fine particles from within a soil mass by flowing water

plain concrete - concrete with no structural reinforcement except, possibly, light steel to reduce shrinkage and temperature cracking

plan and profile - a drawing that shows both the roadway plan view and profile view in the same scale; see PLAN VIEW, PROFILE

plan view - drawing that represents the top view of the road or a structure

plastic deformation - permanent deformation of material beyond the elastic range

plate - a flat sheet of metal which is relatively thick; see SHEET STEEL

plate girder - a large I-shaped beam composed of a solid web plate with flange plates attached to the web plate by flange angles or fillet welds

plug weld - a weld joining two members produced by depositing weld metal within holes cut through one or more of the members; also known as slot weld

plumb bob - a weight hanging on a cord used to provide a true vertical reference

plumb line - a true vertical reference line established using a plumb bob

pneumatic caisson - an underwater caisson in which the working chamber is kept free of water by compressed air at a pressure nearly equal to the water pressure outside it

pointing - the compacting of the mortar into the outermost portion of a joint and the troweling of its exposed surface to secure water tightness or desired architectural effect; replacing deteriorated mortar

ponding - accumulation of water

pontoon bridge - a bridge supported by floating on pontoons moored to the riverbed; a portion may be removable to facilitate navigation

pony truss - a through truss without top chord lateral bracing

pop-out - conical fragment broken out of a concrete surface by pressure from reactive aggregate particles

portable bridge - a bridge that may be readily erected for a temporary communication-transport service and disassembled and reassembled at another location

portal - the clear unobstructed space of a through truss bridge forming the entrance to the structure

portal bracing - a system of sway bracing placed in the plane of the end posts of the trusses

portland cement - a fine dry powder made by grinding limestone clinker made by heating limestone in a kiln; this material reacts chemically with water to produce a solid mass

portland cement concrete - a mixture of aggregate, portland cement, water, and usually chemical admixtures

positive moment - a force applied over a distance that causes compression in the top fiber of a beam and tension in the bottom fiber

post - a member resisting compressive stresses, located vertical to the bottom chord of a truss and common to two truss panels; sometimes used synonymously for vertical; see COLUMN

posting - a limiting dimension, speed, or loading indicating larger dimensions, higher speeds, or greater loads cannot be safely taken by the bridge

post-stressing - see POSTTENSIONING

posttensioning - a method of prestressing concrete in which the tendons are stressed after the concrete has been cast and hardens

pot bearing - a bearing type that allows for multi-dimensional rotation by using a piston supported on an elastomer contained on a cylinder ("pot"), or spherical bearing element

pot holes - irregular shaped, disintegrated areas of bridge deck or roadway pavement caused by the failure of the surface material

Pratt truss - a truss with parallel chords and a web system composed of vertical posts with diagonal ties inclined outward and upward from the bottom chord panel points toward the ends of the truss; also known as N-truss

precast concrete - concrete members that are cast and cured before being placed into their final positions on a construction site

prestressed concrete - concrete with strands, tendons, or bars that are stressed before the live load is applied

prestressing - applying forces to a structure to deform it in such a way that it will withstand its working loads more effectively; see POSTTENSIONING, PRETENSIONING

pretensioning - a method of prestressing concrete in which the strands are stressed before the concrete is placed; strands are released after the concrete has hardened, inducing internal compression into the concrete

primary member - a member designed to resist flexure and distribute primary live loads and dead loads

priming coat - the first coat of paint applied to the metal or other material of a bridge; also known as base coat, or primer

probing - investigating the location and condition of submerged foundation material using a rod or shaft of appropriate length; checking the surface condition of a timber member for decay using a pointed tool, e.g., an ice pick

profile - a section cut vertically along the center line of a roadway or waterway to show the original and final ground levels

programmed repair - those repairs that may be performed in a scheduled program

protective system - a system used to protect bridges from environmental forces that cause steel and concrete to deteriorate and timber to decay, typically a coating system

PS&E - Plans, Specifications, and Estimate; the final submission of the designers to the owner

punching shear - shear stress in a slab due to the application of a concentrated load

Q

quality assurance - an independent evaluation of a product or activity (e.g., an inspection) to establish that a predescribed level of quality has been met

quality control - checks necessary to maintain a uniform level of quality

queen-post truss - a parallel chord type of truss having three panels with the top chord occupying only the length of the center panel

R

railing - a fence-like construction built at the outermost edge of the roadway or the sidewalk portion of a bridge to protect pedestrians and vehicles; see HANDRAIL

rake - an angle of inclination of a surface in relation to a vertical plane; also known as batter

ramp - an inclined traffic-way leading from one elevation to another

range of stress - the algebraic difference between the minimum and maximum stresses in a member

raveling - the consistent loss of aggregate from a pavement resulting in a poor riding surface

reaction - the resistance of a support to a load

rebar - see REINFORCING BAR

redundancy - a structural condition where there are more elements of support than are necessary for stability so that, if a structural member fails, the remaining element can sustain the load

redundant member - a member in a bridge which renders it a statically indeterminate structure; the structure would be stable without the redundant member whose primary purpose is to reduce the stresses carried by the determinate structure

rehabilitation - significant repair work to a structure

reinforced concrete - concrete with steel reinforcing bars embedded in it to supply increased tensile strength and durability

reinforced concrete pipe - pipe manufactured of concrete reinforced with steel bars or welded wire fabric

Reinforced Earth - proprietary retaining structure made of earth and steel strips connected to concrete facing; the steel strips are embedded in backfill and interlock with the facing; see MSE

reinforcement - rods or mesh embedded in concrete to strengthen it

reinforcing bar - a steel bar, plain or with a deformed surface, which bonds to the concrete and supplies tensile strength to the concrete

relaxation - a decrease in stress caused by creep

residual stress - a stress that is trapped in a member after it is formed into its final shape

resistivity of soil - an electrical measurement in ohm-cm that estimates the corrosion activity potential of a given soil

resurfacing - a layer of wearing surface material that is put over the approach or deck surface in order to create a more uniform riding surface

Retained Earth - proprietary retaining structure made of weld wire fabric strips connected to concrete facing; see MSE

retaining wall - a structure designed to restrain and hold back a mass of earth

retractile draw bridge - a bridge with a superstructure designed to move horizontally, either longitudinally or diagonally, from "closed" to "open" position, the portion acting in cantilever being counterweighted by that supported on rollers; also known as traverse draw bridge

rib - curved structural member supporting a curved shape or panel

rigger - an individual who erects and maintains scaffolding or other access equipment such as that used for bridge inspection

rigid frame - a structural frame in which bending moment is transferred between horizontal and vertical or inclined members by joints

rigid frame bridge - a bridge with moment resisting joints between the horizontal portion of the superstructure and vertical or inclined legs

rigid frame pier - a pier with two or more columns and a horizontal beam on top constructed monolithically to act like a frame

rip-rap - stones, blocks of concrete or other objects placed upon river and stream beds and banks, lake, tidal or other shores to prevent scour by water flow or wave action

rivet - a one-piece metal fastener held in place by forged heads at each end

riveted joint - a joint in which the assembled members are fastened by rivets

roadway - the portion of the road intended for the use of vehicular traffic

roadway shoulder - drivable area immediately adjoining the traveled roadway

rocker bearing - a bridge support that accommodates expansion and contraction of the superstructure through a tilting action

rocker bent - a bent hinged or otherwise articulated at one or both ends to provide the longitudinal movements resulting from temperature changes and superimposed loads

rolled shape - forms of rolled steel having "I", "H", "C", "Z" or other cross sectional shapes

rolled-steel section - any hot-rolled steel section including wide flange shapes, channels, angles, etc.

roller - a steel cylinder intended to provide longitudinal movements by rolling contact

roller bearing - a single roller or a group of rollers so installed as to permit longitudinal movement of a structure

roller nest - a group of steel cylinders used to facilitate the longitudinal movements resulting from temperature changes and superimposed loads

rolling lift bridge - a bridge of bascule type devised to roll backward and forward upon supporting girders when operated through an "open and closed" cycle

rubble - irregularly shaped pieces of stone in the undressed condition obtained from a quarry and varying in size

runoff - the quantity of precipitation that flows from a catchment area past a given point over a certain period

S

sacrificial anode - the anode in a cathodic protection system

sacrificial coating - a coating over the base material to provide protection to the base material; examples include galvanizing on steel and aluclading on aluminum

sacrificial protection - see CATHODIC PROTECTION

sacrificial thickness - additional material thickness provided for extra service life of a member in an aggressive environment

saddle - a member located upon the topmost portion of the tower of a suspension bridge which acts as a bearing surface for the catenary cable passing over it

safe load - the maximum load that a structure can support with an appropriate factor of safety

safety belt - a belt worn in conjunction with a safety line to prevent falling a long distance when working at heights; no longer acceptable as fall protection under OSHA rules

safety curb - a curb between 9 inches and 24 inches wide serving as a limited use refuge or walkway for pedestrians crossing a bridge

safety factor - the difference between the ultimate strength of a member and the maximum load it is expected to carry

safety harness - harness with shoulder, leg, and waist straps of approved OSHA design used as personal fall protection in conjunction with appropriate lanyards and tie off devices

sag - to sink or bend downward due to weight or pressure

scab - a plank bolted over the joint between two timber members to hold them in correct alignment and strengthen the joint; a short piece of I–beam or other structural shape attached to the flange or web of a metal pile to increase its resistance to penetration; also known as scab piece

scaling - the gradual disintegration of a concrete surface due to the failure of the cement paste caused by chemical attack or freeze/thaw cycles

scour - removal of a streambed or bank area by stream flow

scour protection - protection of submerged material by steel sheet piling, rip rap, concrete lining, or combination thereof

scuba - self-contained underwater breathing apparatus; a portable breathing device for free swimming divers

scupper - an opening in the deck of a bridge to provide means for water accumulated upon the roadway surface to drain

seam weld - a weld joining the edges of two members placed in contact; in general, it is not a stress-carrying weld

seat - a base on which an object or member is placed

seat angle - a piece of angle attached to the side of a member to provide support for a connecting member either temporarily during its erection or permanently; also known as a shelf angle

secondary member - a member that does not carry calculated live loads; bracing members

section loss - loss of a member's cross sectional area usually by corrosion or decay

section view - an internal representation of a structure element as if a slice was made through the element

seepage - the slow movement of water through a material

segmental - constructed of individual pieces or segments which are collectively joined to form the whole

segmental arch - a circular arch in which the intrados is less than a semi-circle

segregation - in concrete construction, the separation of large aggregate from the paste during placement

seismic - a term referring to earthquakes (e.g., seismic forces)

semi-stub abutment - cantilever abutment founded part way up the slope, intermediate in size between a full height abutment and a stub abutment

service load design - AASHTO's description for Working Stress Design

settlement - the movement of substructure elements due to changes in the soil properties

shear - the load acting across a beam near its support

shear connectors - devices that extend from the top flange of a beam and are embedded in the above concrete slab, forcing the beam and the concrete to act as a single unit

shear spiral - a coil-shaped component welded to the top flange of a beam, as a shear connector

shear stress - the shear force per unit of cross–sectional area; also referred to as diagonal tensile stress

shear stud - a type of shear connector in the form of a road with a head that is attached to a beam with an automatic stud-welding gun

sheet pile cofferdam - a wall-like barrier composed of driven piling constructed to surround the area to be occupied by a structure and permit dewatering of the enclosure so that the excavation may be performed in the open air

sheet piles - flattened Z-shaped interlocking piles driven into the ground to keep earth or water out of an excavation or to protect an embankment

sheet piling - a general or collective term used to describe a number of sheet piles installed to form a crib, cofferdam, bulkhead, etc.; also known as sheeting

sheet steel - steel in the form of a relatively thin sheet or plate; for flat rolled steel, specific thicknesses vs. widths are classified by AISI as bar, strip, sheet or plate

shelf angle - see SEAT ANGLE

shim - a thin plate inserted between two elements to fix their relative position and to transmit bearing stress

shoe - a steel or iron member, usually a casting or weldment, beneath the superstructure bearing that transmits and distributes loads to the substructure bearing area

shop - a factory or workshop

shop drawings - detailed drawings developed from the more general design drawings used in the manufacture or fabrication of bridge components

shoring - a strut or prop placed against or beneath a structure to restrain movement; temporary soil retaining structure

shoulder abutment - a cantilever abutment extending from the grade line of the road below to that of the road overhead, usually set just off the shoulder; see FULL HEIGHT ABUTMENT

shoulder area - see ROADWAY SHOULDER

shrinkage – a reduction in volume caused by moisture loss in concrete or timber while drying

sidewalk - the portion of the bridge floor area serving pedestrian traffic only

sidewalk bracket - frame attached to and projecting from the outside of a girder to serve as a support for the sidewalk stringers, floor and railing or parapet

sight distance - the length of roadway ahead that is easily visible to the driver; required sight distances are defined by AASHTO's "A Policy on Geometric Design of Highways and Streets"

silt - very finely divided siliceous or other hard rock material removed from its mother rock through erosive action rather than chemical decomposition

simple span - beam or truss with two unrestraining supports near its ends

S-I-P forms - see STAY-IN-PLACE FORMS, FORMS

skew angle - the angle produced when the longitudinal members of a bridge are not perpendicular to the substructure; the skew angle is the acute angle between the alignment of the bridge and a line perpendicular to the centerline of the substructure units

skewback - the inclined support at each end of an arch

skewback shoe - the member transmitting the thrust of an arch to the skewback course or cushion course of an abutment or piers; also known as skewback pedestal

slab - a wide beam, usually of reinforced concrete, which supports load by flexure

slab bridge - a bridge having a superstructure composed of a reinforced concrete slab constructed either as a single unit or as a series of narrow slabs placed parallel with the roadway alignment and spanning the space between the supporting substructure units

slide - movement on a slope because of an increase in load or a removal of support at the toe; also known as landslide

slip form - to form concrete by advancing a mold

slope - the inclination of a surface expressed as a ratio of one unit of rise or fall for so many horizontal units

slope protection - a thin surfacing of stone, concrete or other material deposited upon a sloped surface to

prevent its disintegration by rain, wind or other erosive action; also known as slope pavement

slot weld - see PLUG WELD

slump - a measurement taken to determine the stiffness of concrete; the measurement is the loss in height after a cone-shaped mold is lifted

soffit - underside of a bridge deck; also see INTRADOS

soldier beam - a steel pile driven into the earth with its projecting butt end used as a cantilever beam

soldier pile wall - a series of soldier beams supporting horizontal lagging to retain an excavated surface; commonly used in limited right-of-way applications

soil interaction structure - a subsurface structure that incorporates both the strength properties of a flexible structure and the support properties of the soil surrounding the structure

sole plate - a plate attached to the bottom flange of a beam that distributes the reaction of the bearing to the beam

solid sawn beam – a section of tree cut to the desired size at a saw mill

sounding - determining the depth of water by an echo-sounder or lead line; tapping a surface to detect delaminations (concrete) or decay (timber)

spall - depression in concrete caused by a separation of a portion of the surface concrete, revealing a fracture parallel with or slightly inclined to the surface

span - the distance between the supports of a beam; the distance between the faces of the substructure elements; the complete superstructure of a single span bridge or a corresponding integral unit of a multiple span structure; see CLEAR SPAN

spandrel - the space bounded by the arch extrados and the horizontal member above it

spandrel column - a column constructed on the rib of an arch span and serving as a support for the deck construction of an open spandrel arch; see OPEN SPANDREL ARCH

spandrel fill - the fill material placed within the spandrel space of a closed spandrel arch

spandrel tie - a wall or a beam-like member connecting the spandrel walls of an arch and securing them against bulging and other deformation; in stone masonry arches the spandrel tie walls served to some extent as counterforts

spandrel wall - a wall built on the extrados of an arch filling the space below the deck; see TIE WALLS

specifications - a detailed description of requirements, materials, tolerances, etc., for construction which are not shown on the drawings; also known as specs

spider - inspection access equipment consisting of a bucket or basket which moves vertically on wire rope, driven by an electric or compressed air motor

spillway - a channel used to carry water away from the top of a slope to an adjoining outlet

splice - a structural joint between members to extend their effective length

spread footing - a foundation, usually a reinforced concrete slab, which distributes load to the earth or rock below the structure

spring line - the horizontal line along the face of an abutment or pier at which the intrados of an arch begins

spur - a projecting jetty-like construction placed adjacent to an abutment or embankment to prevent scour

stage - inspection access equipment consisting of a flat platform supported by horizontal wire-rope cables; the stage is then slid along the cables to the desired position; a stage is typically 20 inches wide, with a variety of lengths available

staged construction - construction performed in phases, usually to permit the flow of traffic through the site

statics - the study of forces and bodies at rest

station - 100 feet (U.S. customary); 100 meters (metric)

stationing - a system of measuring distance along a baseline

stay-in-place forms - a corrugated metal sheet for forming deck concrete that will remain in place after the concrete has set; the forms do not contribute to deck structural capacity after the deck has cured; see FORMS, S.I.P FORMS

stay plate - a tie plate or diagonal brace to prevent movement

steel - an alloy of iron, carbon, and various other elements

stem - the vertical wall portion of an abutment retaining wall, or solid pier; see BREASTWALL

stiffener - a small member attached to another member to transfer stress and to prevent buckling

stiffening girder - a girder incorporated in a suspension bridge to distribute the traffic loads uniformly among the suspenders and reduce local deflections

stiffening truss - a truss incorporated in a suspension bridge to distribute the traffic loads uniformly among the suspenders and reduce local deflections

stirrup - U-shaped bar used as a connection device in timber and metal bridges; U-shaped bar placed in concrete to resist diagonal tension (shear) stresses

stone masonry - the portion of a structure composed of stone, generally placed in courses with mortar

straight abutment - an abutment whose stem and wings are in the same plane or whose stem is included within a length of retaining wall

strain - the change in length of a body produced by the application of external forces, measured in units of length; this is the proportional relation of the amount of change in length divided by the original length

strand - a number of wires grouped together usually by twisting

strengthening - adding to the capacity of a structural member

stress - the force acting across a unit area in a solid material

stress concentration - local increases in stress caused by a sudden change of cross section in a member

stress range - the variation in stress at a point with the passage of live load, from initial dead load value to the maximum additional live load value and back

stress raiser - a detail that causes stress concentration

stress reversal - change of stress type from tension (+) to compression (−) or vice versa

stress sheet - a drawing showing all computed stresses resulting from the application of a system of loads together with the design composition of the individual members resulting from the application of assumed unit stresses for the material to be used in the structure

stress-laminated timber – consists of multiple planks mechanically clamped together to perform as one unit

stringer - a longitudinal beam spanning between transverse floorbeams and supporting a bridge deck

strip seal joint - a joint using a relatively thin neoprene seal fitted into the joint opening

structural analysis - engineering computation to determine the carrying capacity of a structure

structural member - an individual piece, such as a beam or strut, which is an integral part of a structure

structural redundancy - the ability of an interior continuous span to resist total collapse by cantilever action in the event of a fracture

structural shapes - the various types of rolled iron and steel having flat, round, angle, channel, "I", "H", "Z" and other cross-sectional shapes adapted to heavy construction

structural stability - the ability of a structure to maintain its normal configuration, not collapse or tip in any way, under existing and expected loads

structural tee - a tee-shaped rolled member formed by cutting a wide flange longitudinally along the centerline of web

structure - something, such as a bridge, that is designed and built to sustain a load

strut - a member acting to resist axial compressive stress; usually a secondary member

stub abutment - an abutment within the topmost portion of an embankment or slope having a relatively small vertical height and usually pile supported; stub abutments may also be founded on spread footings

subbase - a layer of material placed between the base course and the subgrade within a flexible pavement structure

subgrade - natural earth below the roadway pavement structure

sub-panel - a truss panel divided into two parts by an intermediate web member, generally a subdiagonal or a hanger

substructure - the abutments and piers built to support the span of a bridge superstructure

superelevation - the difference in elevation between the inside and outside edges of a roadway in a horizontal curve; required to counteract the effects of centrifugal force

superimposed dead load - dead load that is applied to a compositely designed bridge after the concrete deck has cured; for example, the weight of parapets or railings placed after the concrete deck has cured

superstructure - the entire portion of a bridge structure that primarily receives and supports traffic loads and in turn transfers these loads to the bridge substructure

surface corrosion - rust that has not yet caused measurable section loss

suspended span - a simple span supported from the free ends of cantilevers

suspender - a vertical wire cable, metal rod, or bar connecting the catenary cable of a suspension bridge or an arch rib to the bridge floor system, transferring loads from the deck to the main members

suspension bridge - a bridge in which the floor system is supported by catenary cables that are supported upon towers and are anchored at their extreme ends

suspension cable - a catenary cable which is one of the main members upon which the floor system of a suspension bridge is supported; a cable spanning between towers

swale - a drainage ditch with moderately sloping sides

sway anchorage - a guy, stay cable or chain attached to the floor system of a suspension bridge and anchored upon an abutment or pier to increase the resistance of the suspension span to lateral movement; also known as sway cable

sway bracing - diagonal brace located at the top of a through truss, transverse to the truss and usually in a vertical plane, to resist transverse horizontal forces

sway frame - a complete panel or frame of sway bracing

swedged anchor bolt - anchor bolt with deformations to increase bond in concrete; see ANCHOR BOLT

swing span bridge - a movable bridge in which the span rotates in a horizontal plane on a pivot pier, to permit passage of marine traffic

T

tack welds - small welds used to hold member elements in place during fabrication or erection

tail water - water ponded below the outlet of a waterway, thereby reducing the amount of flow through the waterway; see HEADWATER

tape measure - a long, flexible strip of metal or fabric marked at regular intervals for measuring

tee beam - a rolled steel section shaped like a "T"; reinforced concrete beam shaped like the letter "T"

temperature steel - reinforcement in a concrete member to prevent cracks due to stresses caused by temperature changes

temporary bridge - a structure built for emergency or interim use, intended to be removed in a relatively short time

tendon - a prestressing cable, strand, or bar

tensile force - a force caused by pulling at the ends of a member; see TENSION

tensile strength - the maximum tensile stress at which a material fails

tension - stress that tends to pull apart material

thermal movement - contraction and expansion of a structure due to a change in temperature

three-hinged arch - an arch that is hinged at each support and at the crown

through arch - an arch bridge in which the deck passes between the arches

through girder bridge - normally a two-girder bridge where the deck is between the supporting girders

tie - a member carrying tension

tie plate - relatively short, flat member carrying tension forces across a transverse member; for example, the plate connecting a floor beam cantilever to the main floor beam on the opposite side of a longitudinal girder; see STAY PLATE

tie rod - a rod-like member in a frame functioning to transmit tensile stress; also known as tie bar

tie walls - one of the walls built at intervals above an arch ring connecting and supporting the spandrel walls; any wall designed to serve as a restraining member to prevent bulging and distortion of two other walls connected thereby; see DIAPHRAGM WALL

timber - wood suitable for construction purposes

toe - the front portion of a footing from the intersection of the front face of the wall or abutment to the front edge of the footing; the line where the side slope of an embankment meets the existing ground

toe of slope - the location defined by the intersection of the embankment with the surface existing at a lower elevation; also known as toe

toe wall - a relatively low retaining wall placed near the "toe–of–slope" location of an embankment to protect against scour or to prevent the accumulation of stream debris; also known as footwall

ton - a unit of weight equal to 2,000 pounds

torque - the angular force causing rotation

torque wrench - a hand or power tool used to turn a nut on a bolt that can be adjusted to deliver a predetermined amount of torque

torsion - twisting about the longitudinal axis of a member

torsional rigidity - a beam's capacity to resist a twisting force along the longitudinal axis

tower - a pier or frame supporting the catenary cables of a suspension bridge

traffic control - modification of normal traffic patterns by signs, cones, flagmen, etc.

transducer - a device that converts one form of energy into another form, usually electrical into mechanical or the reverse; the part of ultrasonic testing device which transmits and receives sound waves

transverse bracing - the bracing assemblage engaging the columns of bents and towers in planes transverse to the bridge alignment that resists the transverse forces tending to produce lateral movement and deformation of the columns

transverse girder - see CROSS GIRDER

travel way - the roadway

tremie - a piece of construction equipment (e.g., pipe or funnel) used to place concrete underwater

trestle - a bridge structure consisting of spans supported on braced towers or frame bents

truck loading - a combination of loads used to simulate a single truck passing over a bridge

truss - a jointed structure made up of individual members primarily carrying axial loads arranged and connected in triangular panels

truss bridge - a bridge having a pair of trusses for a superstructure

trussed beam - a beam stiffened to reduce its deflection by a steel tie-rod that is held at a short distance from the beam by struts

truss panel - see PANEL

tubular sections - structural steel tubes, rectangular, square or circular; also known as hollow sections

tubular truss - a truss whose chords and struts are composed of pipes or cylindrical tubes

tunnel - an underground passage, open to daylight at both ends

turnbuckle - a long, cylindrical, internally threaded nut with opposite hand threads at either end used to connect the elements of adjustable rod and bar members

two-hinged arch - a rigid frame that may be arch-shaped or rectangular with hinges at both supports

U

U-bolt - a bar bent in the shape of the letter "U" and fitted with threads and nuts at its ends

ultimate strength - the highest stress that a material can withstand before breaking

ultrasonic thickness gage - an instrument used to measure the thickness of a steel element using a probe which emits and receives sound waves

ultrasonic testing - nondestructive testing of a material's integrity using sound waves

underpass - the lowermost feature of a grade separated crossing; see OVERPASS

uniform load - a load of constant magnitude along the length of a member

unit stress - the force per unit of surface or cross–sectional area

uplift - a negative reaction or a force tending to lift a beam, truss, pile, or any other bridge element upwards

upper chord - the top longitudinal member of a truss

V

vertical - describes the axis of a bridge perpendicular to the underpass surface

vertical alignment - a roadway's centerline or baseline alignment in the vertical plane

vertical clearance - the distance between the structure and the underpass

vertical curve - a sag or crest in the profile of a roadway, usually in the form of a parabola, to transition between grades

vertical lift bridge - a bridge in which the span moves up and down while remaining parallel to the roadway

viaduct - a series of spans carried on piers at short intervals

vibration - the act of vibrating concrete to compact it

Vierendeel truss - a truss with only chords and verticals joined with rigid connections designed to transfer moment

voided slab - a precast concrete deck unit cast with cylindrical voids to reduce dead load

voids - an empty or unfilled space in concrete

Voussoir - one of the truncated wedge-shaped stones composing a ring course in a stone arch; also known as ring stone

voussoir arch - an arrangement of wedge shaped blocks set to form an arched bridge

W

wale, waler - horizontal bracing running along the inside walls of a sheeted pit or cofferdam

Warren truss - a triangular truss consisting of sloping members between the top and bottom chords and no verticals; members form the letter W

washer - a small metal ring used beneath the nut or the head of a bolt to distribute the load or reduce galling during tightening

water/cement ratio - the weight of water divided by the weight of portland cement in concrete; this ratio is a major factor in the strength of concrete

waterproofing membrane - an impervious layer placed between the wearing surface and the concrete deck, used to protect the deck from water and corrosive chemicals that could damage it

waterway opening - the available width for the passage of water beneath a bridge

wearing surface - the topmost layer of material applied upon a roadway to receive the traffic loads and to resist the resulting disintegrating action; also known as wearing course

web - the portion of a beam located between and connected to the flanges; the stem of a dumbbell type pier

web crippling - damage caused by high compressive stresses resulting from concentrated loads

web members - the intermediate members of a truss, not including the end posts, usually vertical or inclined

web plate - the plate forming the web element of a plate girder, built-up beam or column

web stiffener - a small member welded to a beam web to prevent buckling of the web

weephole - a hole in a concrete retaining wall to provide drainage of the water in the retained soil

weld - a joint between pieces of metal at faces that have been made plastic and caused to flow together by heat or pressure

weldability - the degree to which steel can be welded without using special techniques, such as pre-heating

welded bridge structure - a structure whose metal elements are connected by welds

welded joint - a joint in which the assembled elements and members are connected by welds

welding - the process of making a welded joint

weld layer - a single thickness of weld metal composed of beads (runs) laid in contact to form a pad weld or a portion of a weld made up of superimposed beads

weld metal - fused filler metal added to the fused structure metal to produce a welded joint or a weld layer

weld penetration - the depth beneath the original surface to which the structure metal has been fused in the making of a fusion weld; see PENETRATION

weld sequence - the order of succession required for making the welds of a built-up piece or the joints of a structure, to minimize distortion and residual stresses

weld toe - particularly in a filet weld, the thin end of the taper furthest from the center of the weld cross section

wheel guard - a raised curb along the outside edge of traffic lanes to safeguard constructions outside the roadway limit from collision with vehicles

wheel load - the load carried by and transmitted to the supporting structure by one wheel of a traffic vehicle, a movable bridge, or other motive equipment or device; see AXLE LOAD

weep hole - a hole in a concrete element (abutment backwall or retaining wall) used to drain water from behind the element; any small hole installed for drainage

Whipple truss - a double-intersecting through Pratt truss where the diagonals extend across two panels

wide flange - a rolled I-shaped member having flange plates of rectangular cross section, differentiated from an S-beam (American Standard) in that the flanges are not tapered

wind bracing - the bracing systems that function to resist the stresses induced by wind forces

wind lock - a lateral restraining device found on steel girder and truss bridges

wingwall - the retaining wall extension of an abutment intended to restrain and hold in place the side slope material of an approach roadway embankment

wire mesh reinforcement - a mesh made of steel wires welded together at their intersections used to reinforce concrete; welded wire fabric

wire rope - steel cable of multiple strands which are composed of steel wires twisted together

working stress - the unit stress in a member under service or design load

working stress design - a method of design using the yield stress of a material and a factor of safety that determine the maximum allowable stresses

wrought iron - cast iron that has been mechanically worked to remove slag and undissolved carbon

wythe - a single layer of brick or stone in the thickness direction

X

X-ray testing - nondestructive testing technique used for detecting internal flaws by passing X-rays through a material to film or other detector

Y

yield - permanent deformation (permanent set) which a metal piece takes when it is stressed beyond the elastic limit

yield point - see YIELD STRESS

yield stress - the stress at which noticeable, suddenly increased deformation occurs under slowly increasing load

Z

zee - steel member shaped like a modified "Z" in cross section

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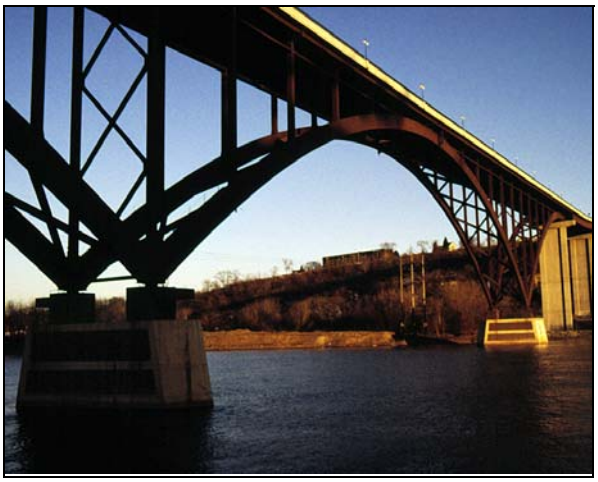
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Documentation and Evaluation

Solution Manual

Topic 5.1 – Timber Decks

Deck documentation (notes and sketch)

Total Quantity

Total deck quantity = 16' W x 26' L = 416 SF or 5 m W x 8m L = 40 m^2

Inspection notes:

Timber plank deck with top of planks weathered and underside sound. 1 plank partially failed. ≈ 1/3 of planks @ connection to fascia beams loose, warped & rotted @ ends @ southwest corner of deck.

Check List

√ Member Description

√ Defect Locations

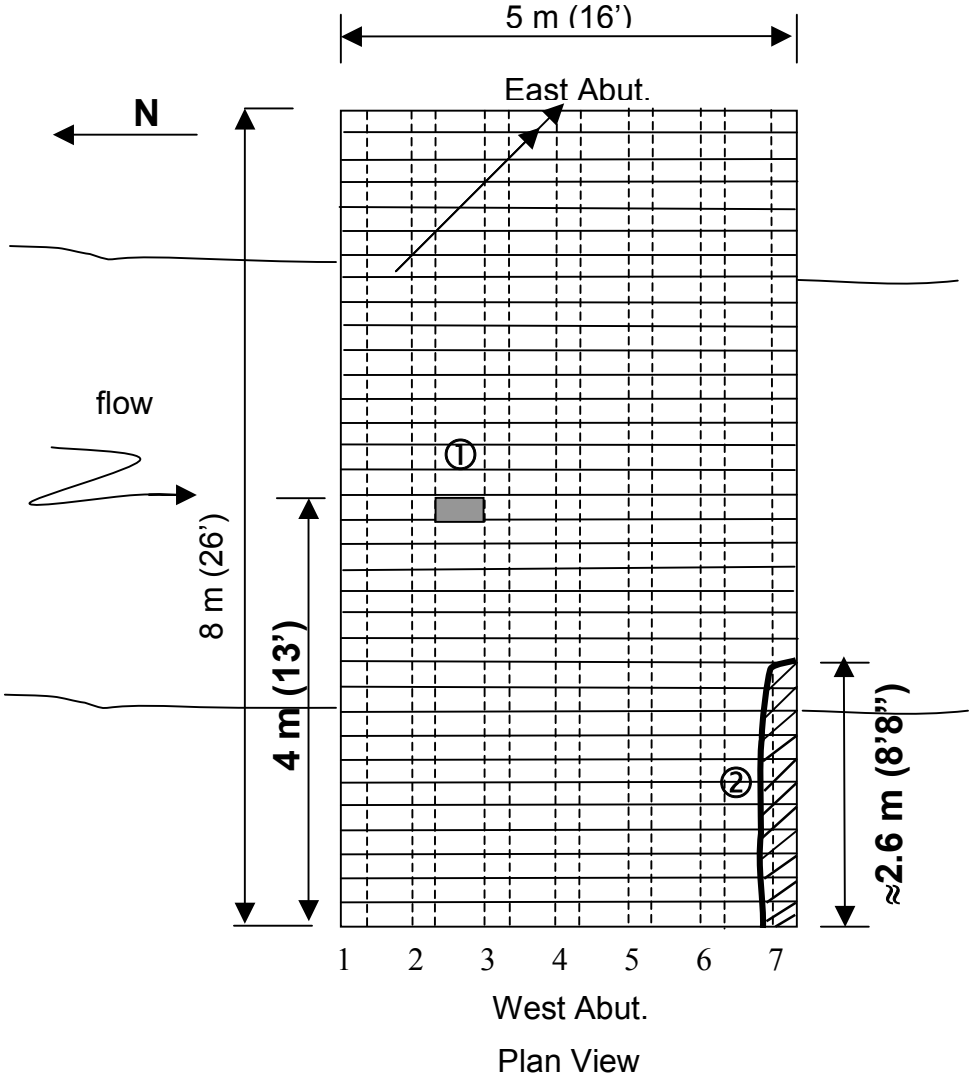
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

① Partial plank failure, 762 mm (30") long. (see Photo 3).

② Approx 11 planks loose; end 102 mm (4") rotted. (see Photo 4)



Member **TIMBER DECK**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 5.1.13 – 5.1.14. Rate the subject timber deck.

NBI Component Rating Method:
Circle the rating for this deck and explain why this rating was chosen (rating reasoning).

Deck Condition Rating = 9 8 7 **6** 5 4 3 2 1 0

Deck is in "satisfactory" condition structurally. Numerous loose planks and rotted plank ends are minor deteriorations. Partially failed plank beam isolated case.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
31 Timber Deck (Bare)	1 (40 m^2) (416 SF)	EA		1			

Topic 5.2 – Concrete Decks

Deck documentation exercise

Total Quantity:

Total deck quantity = 62' W x 30' L = 1,860 SF or 19 m W x 9 m L = 171 m^2

Inspection notes:

CIP concrete deck (bare) with HL to 3 mm (1/8") transverse and longitudinal cracks on surface in 2 SB lanes only. Underside of SB lanes have transverse & longitudinal cracks w/ efflorescence. Large spall 760 mm x 3960 mm (2'6" x 13') under median near south abutment w/ exposed rebar w/ minor section loss.

Check List

√ Member Description

√ Defect Locations

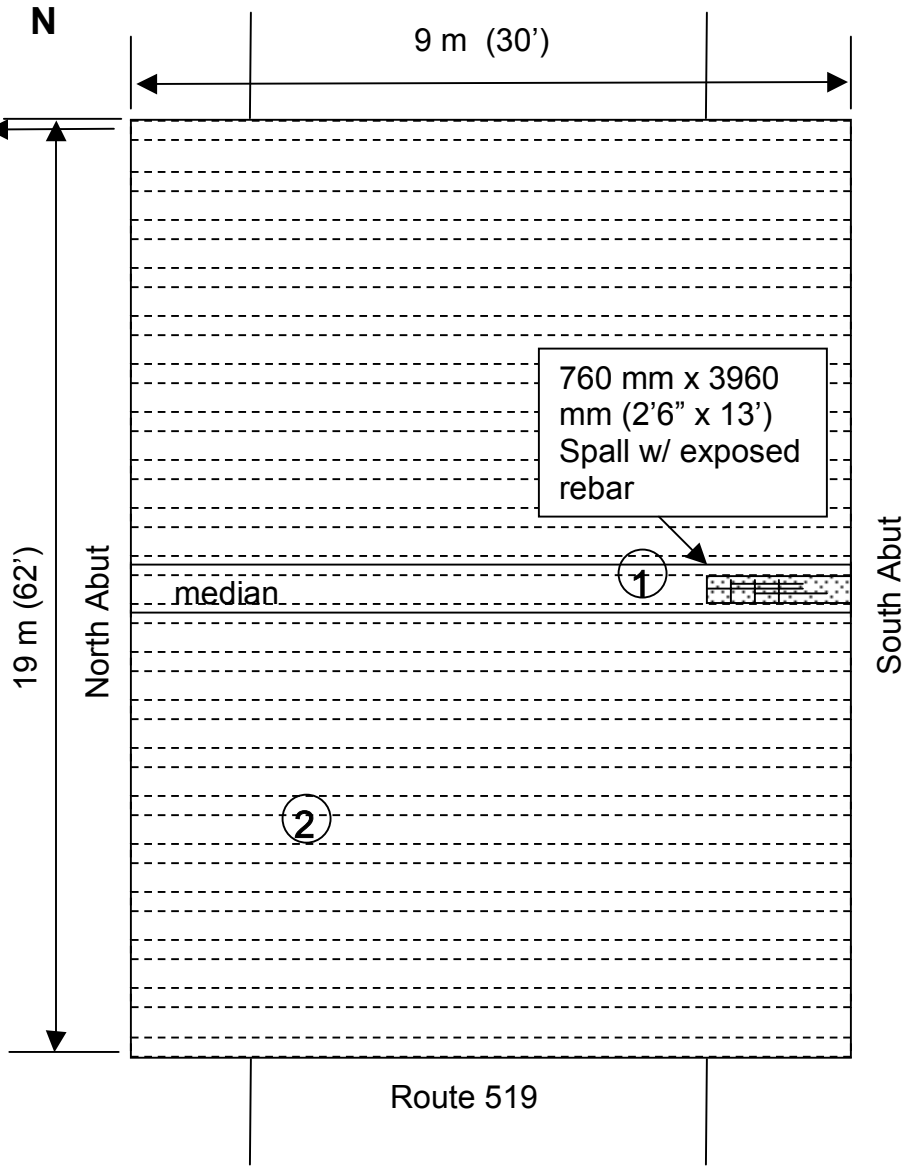
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

① Spall with exposed rebars w/ minor section loss (see Photos 6 &7)

② Hairline to medium longitudinal & transverse cracks in SB lanes top and bottom (see Photos 3, 4 &5)



Member **Concrete Deck (Bare)**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 5.2.20 – 5.2.22. Rate the subject concrete deck.

NBI Component Rating Method:
Circle the rating for this deck and explain why this rating was chosen (rating reasoning).

Deck Condition Rating = 9 8 7 6 **5** 4 3 2 1 0

Deck is in "fair" condition structurally. The transverse and longitudinal cracks are typical over one-half of the span. Efflorescence was observed. One large spall was found with minor section loss on rebar (spall was 1.7% of deck underside area).

Area provided for notes or calculations:

2.5' x 13' / 1860 SF = 1.7%

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	Core Elements						
12 Concrete (Bare)	1 (1860 SF) 172 m^2	EA	1				

	Other Elements						

	Smart Flags						
358 Deck Cracking	1	EA			1		
359 Soffit Under Surface of Deck	1	EA				1	

Topic 5.3 – Steel Decks

Deck documentation (notes and sketch)

Total Quantity
Total deck quantity = 7 m W x 9 m L = 63 m^2 or 24' W x 30' L = 720 SF

Inspection notes:

Bituminous ws w/ few scattered areas of longitudinal and transverse cracks to 1 mm (1/16") wide & no coverage @ shoulders 610 mm (2') wide x full length. Exposed portions of water tables 1 mm (1/16") surface corrosion typ. Steel pans in fascia areas heavily corroded w/ areas of 100% section loss. Similar condition between beams 2 & 3 and beams 3 & 4 @ FAB. Bearing bars in fascia bays & between beams 2 thru 4 @ FAB exhibit 3 mm (1/8") section loss.

Check List

√ Member Description

√ Defect Locations

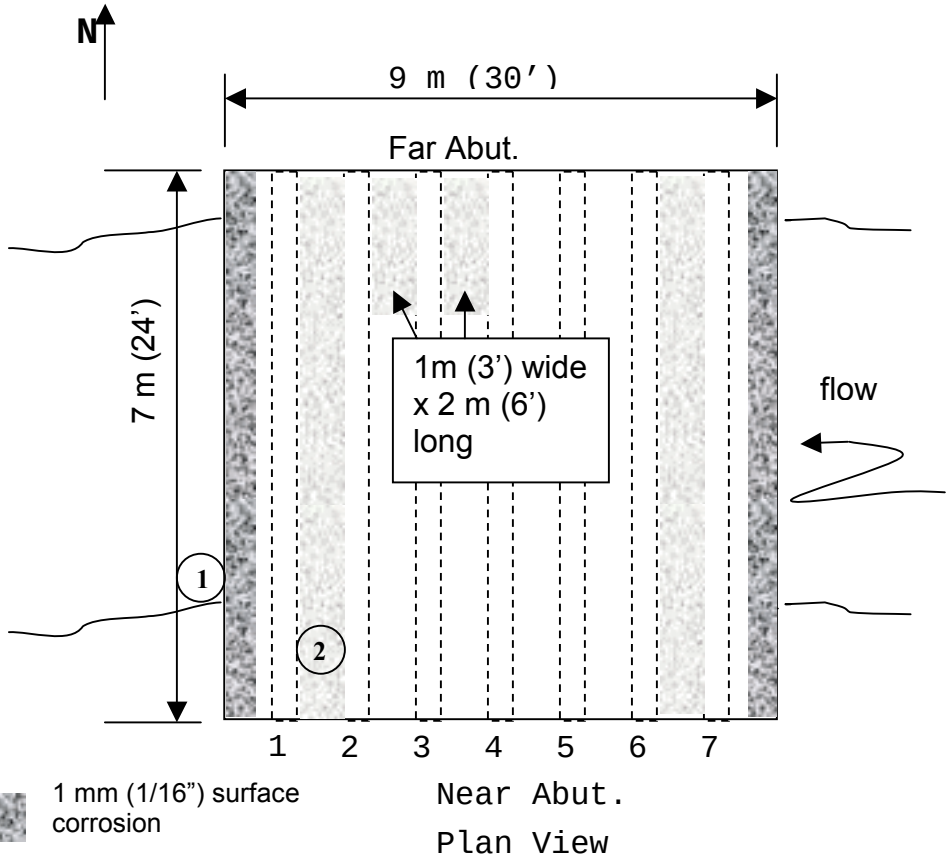
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

(1) Exposed deck along shoulders 2' wide x full length w/ surface corrosion (see Photo 1)

(2) Steel pans w/ up to 100% section loss & 3 mm (1/8") section loss to primary bearing bars (see Photos 3 & 4)



1 mm (1/16") surface corrosion

Up to 100% section loss of steel pans w/ 3 mm (1/8") section loss to primary bearing bars

Member **Steel Deck (w/ Overlay)**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 5.3.15 – 5.3.16. Rate the subject concrete deck.

NBI Component Rating Method:
Circle the rating for this deck and explain why this rating was chosen (rating reasoning).

Deck Condition Rating = 9 8 7 6 5 (4) 3 2 1 0

Deck is in "poor" condition structurally. The steel pans have areas of 100% section loss with 3 mm (1/8") section loss to primary bearing bars between beams 6 & 7 and 1 & 2 for full length of deck and beams 2-4 for 2 m (6') from FAB. The typical condition of the deck is random areas of heavy rusting including bearing bars.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	Core Elements						
29 Steel Concrete Filled	1 63 m^2 (720 SF)	EA					1

	Other Elements						

	Smart Flags						

Topic 5.4 – Deck Joints, Drainage Systems, Lighting and Signs

Element Level Method

- Determine CoRe Elements and their numbers
- Calculate quantities
- Determine correct condition states

Use the current (or Agency CoRe Element Guide if supplied) AASHTO CoRe Element Guide.



Photo 1: Central view of deck surface. There are three pourable joint seals which are each 12 m (40') long.



Photo 2: Brown staining on side of pier (not rust stains). This condition also exists on far (east) abutment.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
CoRe Elements							
301 Pourable Joint Seal	36 (120)	m (LF)	12 (40)	24 (80)			

Topic 6.1 – Solid Sawn Timber Bridges

Deck documentation exercise (notes and sketch)

Total Quantity:

Total beam quantity = 7 beams x 8 m L = 56 m or
7 beams x 26' L = 182 LF

Inspection notes:

2 m (6') long horizontal shear crack @ end of fascia beam (see Sketch). Both fascia beams with moderate sized surface checks. Interior beams w/ scattered mold & stain w/ little decay & no section loss.

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

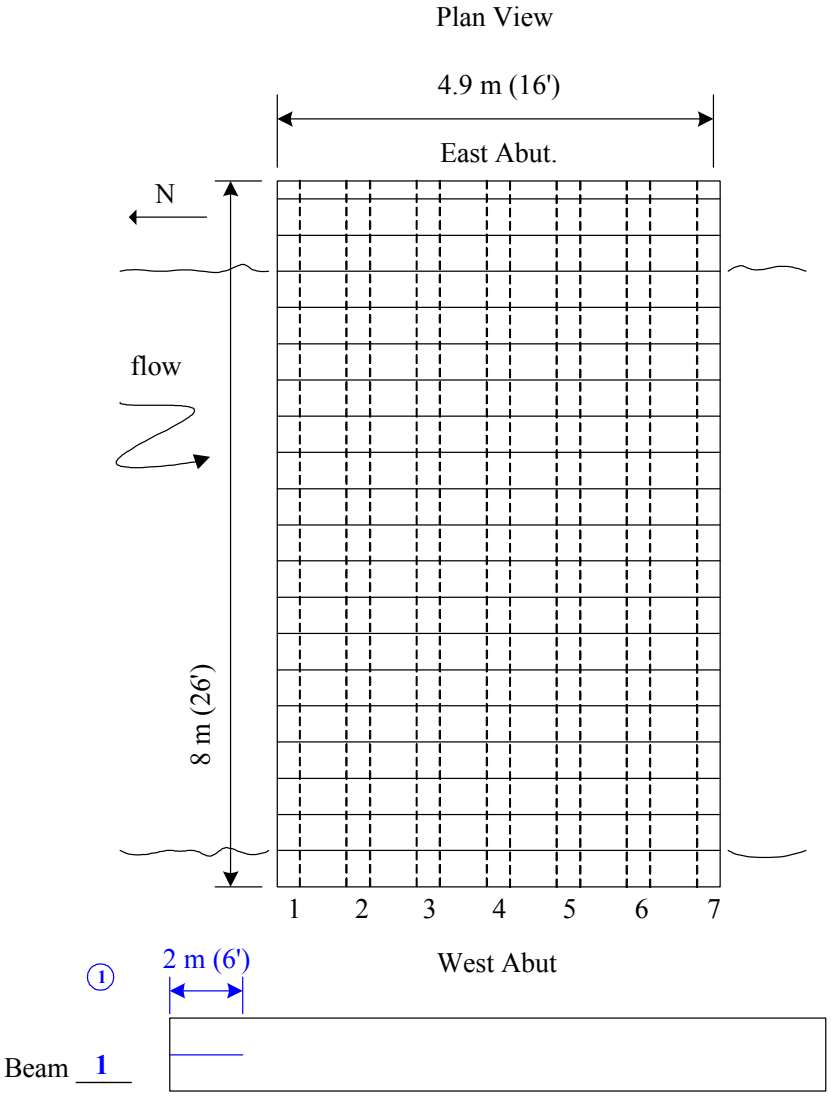
√ Defect Dimensions

√ Defect Quantity (if required)

① Horizontal shear crack in fascia beam (see Photo 2).

② Moderate surface check on both fascia beams (see Photo 3).

③ Mold & stain (see Photo 4)



Member **Timber Beam (Girder)**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 6.1.18 – 6.1.19. Rate the subject solid sawn timber multi-beam superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 **(6)** 5 4 3 2 1 0

Superstructure is in "satisfactory" condition structurally. The beams that are the primary structural members are generally sound. They do not have any section loss, but they do have molds and stains, which may indicate the beginning of decay fungi. Surface checks on fascia beams are minor defects, but do indicate extensive weathering. The horizontal shear crack on the fascia beam is localized and is not seriously affecting the capacity of the structure at this time.

Area provided for notes or calculations:

Horizontal shear crack 2 m (6 LF) Beam 1 CS 3
Fascia beams moderate size surface checks CS 2
[8 m + 8 m] – 2 m for CS 3 = 14 m
(26' + 26') – 6' for CS 3 = 46 LF

Total – CS 3 – CS 2 = CS 1
56 m – 14 m – 2 m = 40 m
182' – 46' – 6' = 130 LF

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
		CoRe Elements					
111 Timber Open Girder	56 (182)	m (LF)	40 (130)	14 (46)	2 (6)		

		Other Elements						

		Smart Flags						

Topic 6.2 – Glulam Timber Bridges

Deck documentation exercise (notes and sketch)

Total Quantity:

Total beam quantity = 7 beams x 21.3 m L = 149 m or
7 beams x 70' L = 490 LF

Inspection notes:

Max. shear zones @ beam ends near supports, no deficiencies found. Max. flexure zones near mid-span, no deficiencies except scattered water stains typical. Stained areas show no signs of decay. Diaphragms free of defects.

Check List

√ Member Description

√ Defect Locations

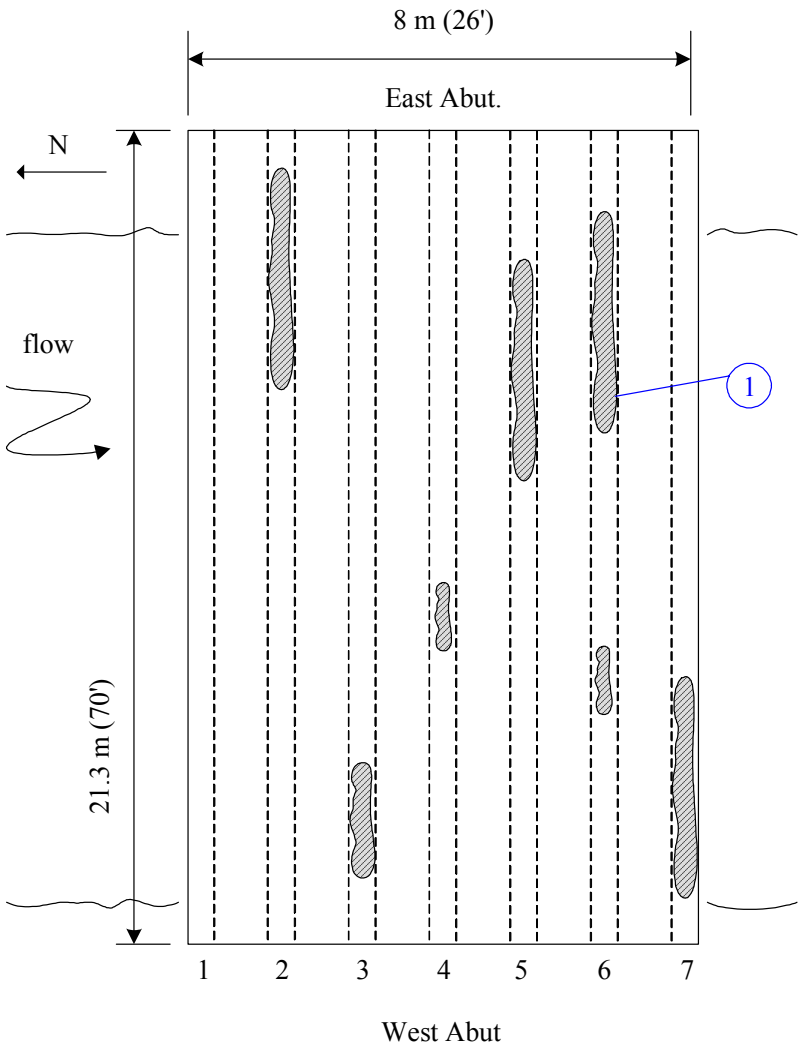
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

① Scattered water staining on beams (see photos 3 & 4).

Plan View



Member **Timber Open Girder**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 6.2.15 – 6.2.16. Rate the subject glulam timber multi-beam superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 (**8**) 7 6 5 4 3 2 1 0

Superstructure is in "very good" condition structurally. The beams which are the primary structural members, are sound with no noteworthy deficiencies or decay found. Close-up inspection of the scattered water staining on the beams indicated that no decay has begun on the beams.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
111 Timber Open Girder	149 (490)	m (LF)	149 (490)				

	Other Elements						

	Smart Flags						

Topic 7.1 – Cast in Place Slabs

Substructure documentation exercise (notes and sketch)

Total Quantity:

Slab: 9 m length x 8 m width c/c = 72m^2
(29' length x 26' width c/c = 754 ft^2)

Inspection notes:

No spalls or delaminations noted on top surface of the deck.

Corner spalls on both underside edges.

Random map cracking and dark spots on underside of slab throughout.

Check List

√ Member Description

√ Defect Locations

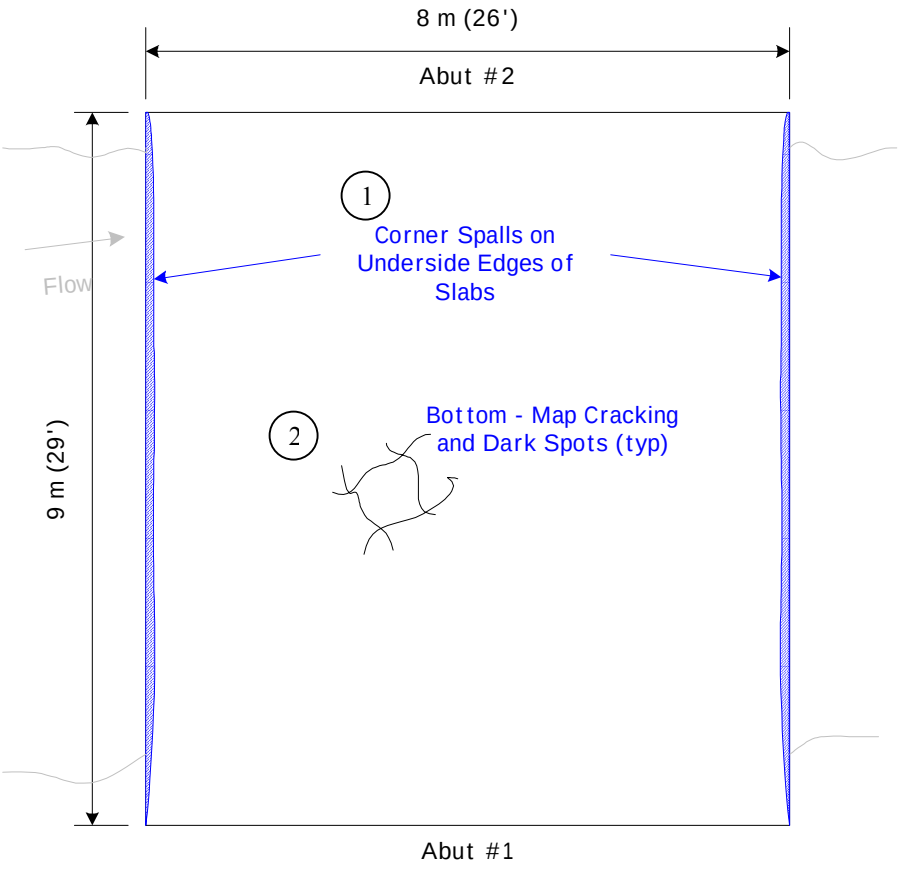
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

(1) Corner Spalls (see photo 1)

(2) Map Cracking and Dark Spots (see photo 2)



Plan View

Member **Cast-in-Place Slab - Bare**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.1.10 – 7.1.11. Rate the single span CIP concrete slab bridge.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 (6) 5 4 3 2 1 0

The superstructure is in satisfactory structural condition. The slab shows some minor deterioration. No structural cracks were found.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
38 Concrete Slab – Bare	1 72 m^2 (754 SF)	EA	1				
	Other Elements						
	Smart Flags						
358 Deck Cracking	1	EA	1				
359 Soffit of Concrete Decks and Slabs	1	EA		1			

Topic 7.2 – Tee Beams

Deck documentation exercise (notes and sketch)

Total Quantity:

Total beam quantity = 4 beams x 25' L = 100 LF or
4 beams x 7.6 m L = 30.4 m say 30.5 m
Deck = 25' c/c Span x 18' wide = 450 SF or
7.6 m c/c span x 5.5 m wide = 42 m^2

Inspection notes:

Concrete tee beams with 3 of 4 beams showing no signs of deterioration. Beam #4 has 305 mm (12") dia. X 102 mm (4") deep spall area located 3.125 m (10'3") from abut. #2. The 4 exposed #8 bars are spaced at 102 mm (4") and the measured diameters are ≈ 17 mm (11/16") remaining section. Underside of deck in good condition (photo 2)

Check List

√ Member Description

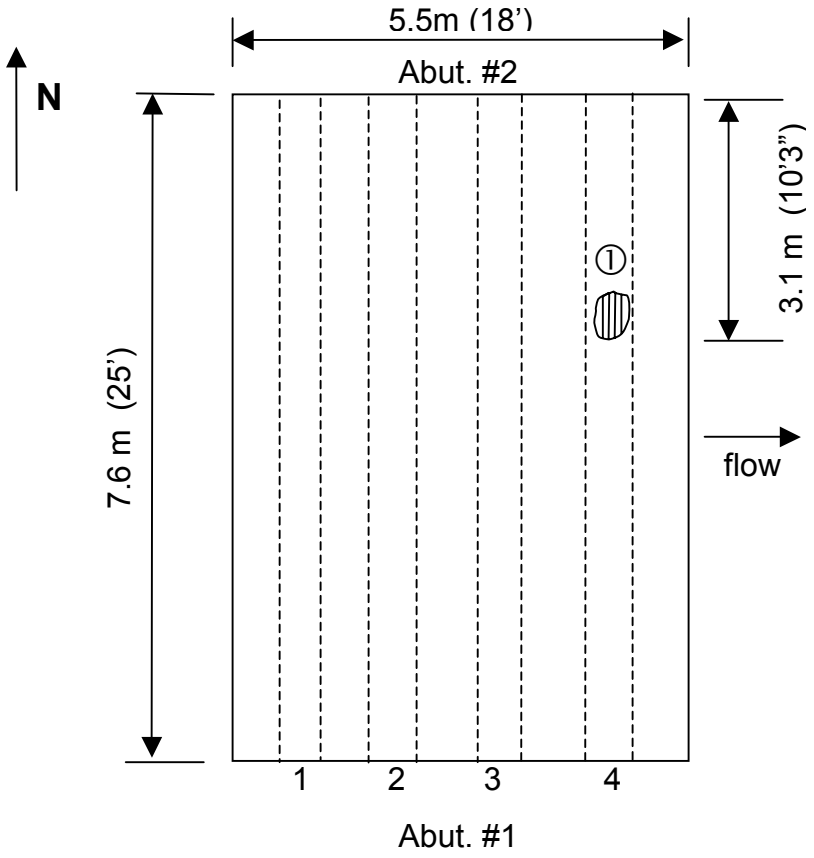
√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

① 305 mm (12") diameter x 102 mm (4") deep spall. Four exp. #8 rebar w/ 17 mm (11/16") remaining section (see Photo 3).



Plan View –Underside

Member type **_Concrete Open Beam/Girder**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.2.10 – 7.2.11. Rate the subject reinforced concrete tee beam superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Deck Condition Rating = 9 8 **7** 6 5 4 3 2 1 0

Superstructure Condition Rating = 9 8 7 6 5 **4** 3 2 1 0

Superstructure is in "poor" condition structurally. Tee beam #4, which is a primary structural member, has advanced section loss of the main tension reinforcement near mid-span. Therefore all primary structural elements are not sound. Although the guidelines call for an overall characterization of the general condition of the entire component being rated, the rating of "5" states that all primary Structural elements are sound but may have minor section loss, cracking, or spalling. In this case, "4" is the most appropriate rating because not all primary members are sound.

Underside of the deck is in good condition (photo 2). Top side of deck is not visble due To asphalt wearing surface.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
		CoRe Elements					
110 RC Open Girder	30.5 (100)	m (LF)	30 (99)			0.5 (1)	
13 Concrete Deck (unprotected with AC Overlay)	1 (42 m^2) (450 SF)	EA	1				

		Smart Flags					
359 Soffit of Concrete Decks and Slabs	1	EA	1				

Topic 7.3 – Concrete Two-Girder Systems

Deck documentation exercise (notes and sketch)

Total Quantity:

Girders: 11 m span x 2 = 22 m (36' span x 2 = 72 LF)
Deck: 11 m span x 7 m c/c = 77 m^2 (36' span x 22' c/c = 792 SF)

Inspection notes:

Hairline to 3 mm (1/8") vertical cracks on inside of girder faces. 460 mm x 250 mm (18" x 10") spall with exposed rebar at midspan of upstream girder. 760 mm x 480 mm (30" x19") spall at 4/5 span with exposed rebar and 2 mm (1/16") section loss of downstream girder. 610 mm x 150 mm (24" x 6") spall at midspan with exposed rebar and 2 mm (1/16") section loss of downstream girder. Underside of deck has extensive transverse cracking with heavy efflorescence.

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

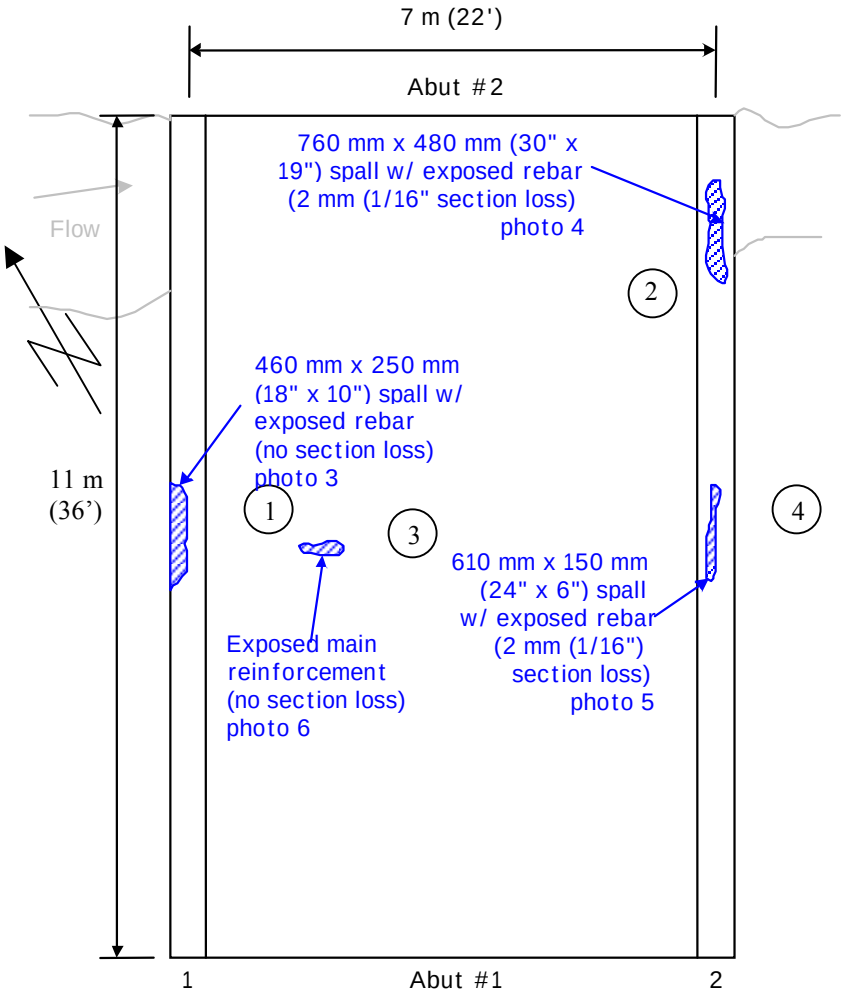
√ Defect Quantity (if required)

1 See Photo3

2 See Photo 4

3 See Photo 6

4 See Photo 5



Plan View

Member Concrete Through Girders

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.3.11 – 7.3.13. Rate the subject concrete through girder bridge.

NBI Component Rating Method:
Circle the rating for the superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 (5) 4 3 2 1 0

It is in fair condition structurally. The primary structural elements (girders) are basically sound, showing three spalls with some minor section loss, and some random vertical cracks.

Area provided for notes and calculations:

- 18" x 10" spall, exposed rebar 1.5' (Photo 3) CS. 3
- 30" x 19" spall, exposed rebar 2.5' (Photo 4) CS. 3
- 24" x 6" spall, exposed rebar w/ 1/16" section loss 2.0' (Photo 5) CS. 3

6.0'

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
110 RC Open Girder	22 (72)	m (LF)	20 (66)		2 (6)		

	Smart Flags						

Topic 7.4 – Concrete Channel Beams

Superstructure documentation exercise (notes and sketch)

Total Quantity

***Beams: 11 lines x 33 m (108') span = 363 m (1188 LF)**
Deck: 33 m (108') span x 12 m (39') c/c = 396 m^2 (4212 SF)

Inspection notes:

No problems with bearing areas and shear zones. Hairline vertical flexure cracks found on 12 of the beams. No noteworthy problems with any of the flange tie bolts and diaphragms were in good condition. Asphalt overlay exhibited full-length longitudinal hairline cracks near wheel paths.

Check List

√ Member Description

√ Defect Locations

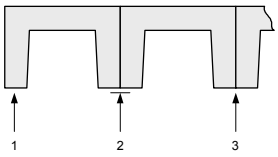
√ Defect Description /Severity

√ Defect Dimensions

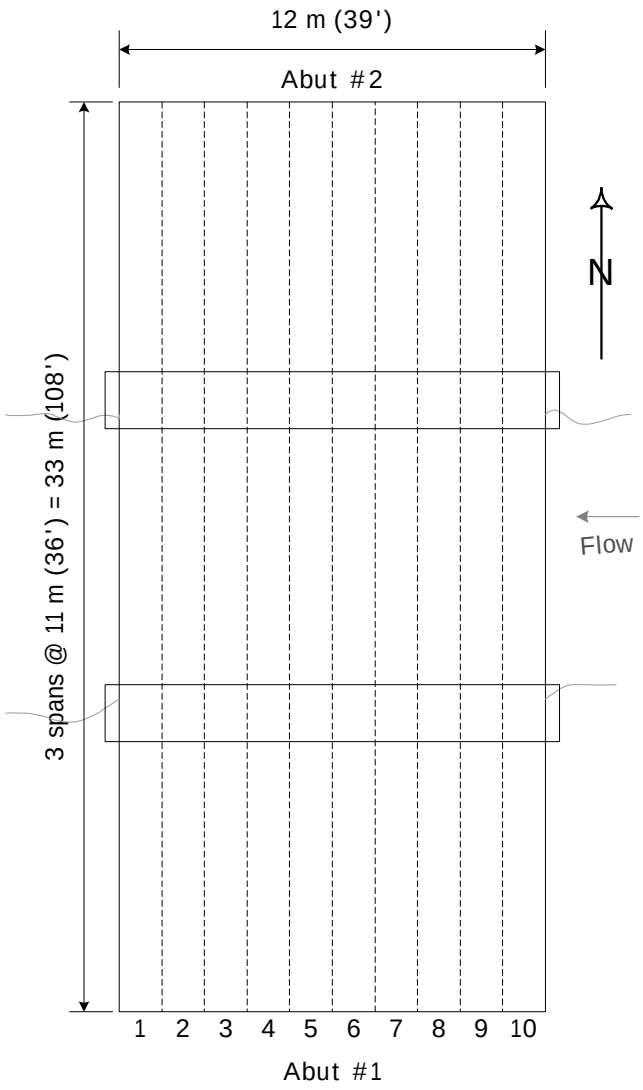
√ Defect Quantity (if required)

Note: Hairline flexure cracks found on 12 of the beams.

*Instructor Note
Beam lines (2 Faces)



Number of Exterior webs divided by 2 (22 faces / 2 = 11 beams) AASHTO



Plan View

Member **Precast Channel Beam (Mild Reinforcement)**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.4.10 – 7.4.12. Rate the subject concrete channel beam bridge.

NBI Component Rating Method:
Indicate the item number, circle the rating for the channel beam superstructure, and provide reasoning for the rating.

Deck Condition Rating = 9 8 (7) 6 5 4 3 2 1 0
Superstructure Condition Rating = 9 8 (7) 6 5 4 3 2 1 0

It is in good condition structurally. The primary elements (channel beams) are sound with only hairline flexure cracks noted.

Area provided for notes and calculations:

- 12 Beams with hairline cracks **(Condition State 2)**

  12 x 11 m = 132 m
  12 x 36' = 432'

  CS 1
  1188 – 432' = 756 LF
  363 – 132 m = 231 m

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
110 RC Open Girder	329 (1188)	m (LF)	198 (756)	132 (432)			
13 Concrete Deck Unprotected with AC Overlay	1 (391 m^2) (4212 SF)	EA	1				

	Smart Flags						
359 Soffit of Concrete Decks and Slabs	1	EA	1				

Topic 7.5 – Concrete Arches and Arch Culverts

Superstructure documentation

Total Quantity:
Arch culvert: Use the barrel length of 7 m (22 LF)

Inspection notes:
Cracking, efflorescence, scaling, and spalling in an area 1.2 m x 0.46 m (48” x 18”) on the south underside face of the arch (Photo 2). A large corner spall 0.46 m x 0.3 m (18” x 12”) at the southeast corner of the arch (Photo 4). Cracking, efflorescence, scaling, and spalling in and area 0.3 m x 0.3 m (12” x 12”) of the north underside face of the arch (Photo 5). A large corner spall 0.9 m x 0.6 m (36” x 24”) at the northeast corner of the arch (Photo 8).

Check List

√ Member Description

√ Defect Locations

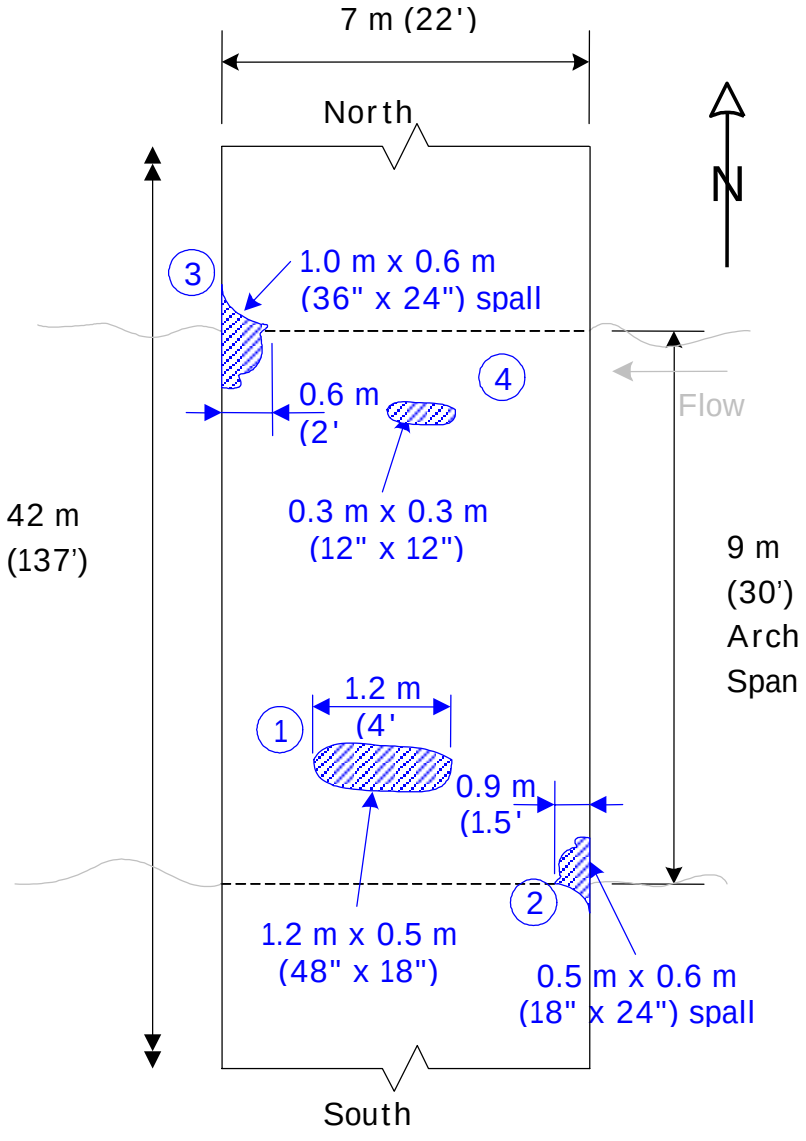
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

① Cracking, spalling (see Photo 2)

② Large corner spall (see Photo 4)



Member **Reinforced Concrete Arch Culvert**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.5.17 – 7.5.19. Rate the subject closed spandrel concrete arch culvert.

NBI Component Rating Method:
Circle the rating for the superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 (5) 4 3 2 1 0

It is in fair structural condition. The concrete arch is generally sound but has some cracking, scaling and spalling. The structural capacity of the structure appears unaffected by the deterioration.

Area provided for notes and calculations:
Spalls

(1) 4' – CS – 3

(2) 1.5' – CS – 3

(3) 2' – CS – 3

(4) -* - CS – 3
7.5' (say 8') or 2.4 m say 3 m

* 1 cancels it out. If (4) condition state would have been worse, it would have governed.

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
241 Reinforced Concrete Arch Culvert	7 (22)	m (LF)	4 (14)		3 (8)		

	Smart Flags						

Topic 7.6 – Concrete Rigid Frames

Superstructure documentation exercise (notes and sketch)

Total Quantity:

Slab: 21 m x 24 m c/c = 504 m^2
(slab-supported by substructure)
68' x 78' c/c = 5304 SF
Abutment: 24 m + 1 m x 2 abutments = 50 m
78' + 3' x 2 abutments = 162 LF

Inspection notes:
Fascia areas of slab were generally in satisfactory condition. Vertical corners were generally in good condition. No cracks were found in faces of legs. Underside of slab at north end has several spalls, exposed rebar, and rust stains at construction joint. Only minor section loss. The construction joint at frame legs has areas of spalling and exposed rebar with minor section loss. The asphalt-wearing surface is in good condition.
No problems noted in the vertical walls.

Check List

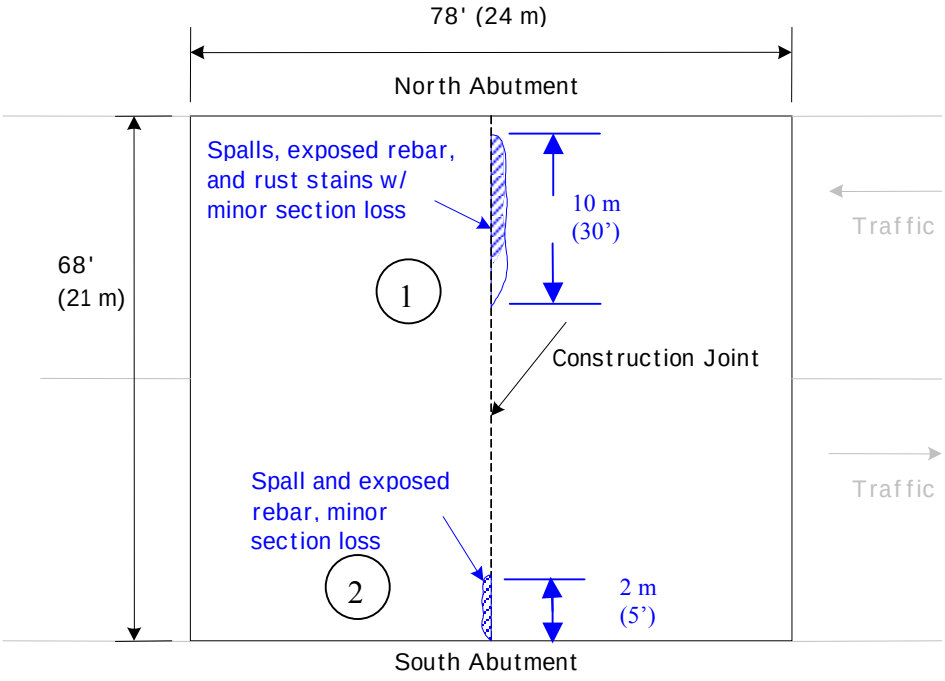
√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)



(1) Spalls, exposed rebar and rust stains at joint (see photo 3)

(2) Spall and exposed rebar with minor section loss (see photo 5)

Member **Concrete Slab with Overlay Concrete Abutment**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.6.11 – 7.6.13. Rate the subject concrete rigid frame bridge.

NBI Component Rating Method:
Circle the rating for the superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 (6) 5 4 3 2 1 0

Substructure Condition Rating = 9 8 (7) 6 5 4 3 2 1 0

It is in satisfactory structural condition. The frame slab (superstructure) has a minor amount of rebar section loss and spalling along the construction joint.

Area for notes and calculations:

359 – (10' x 5') 1' wide / (68' x 78')=0.3%
215 – No problems noted (photo 2)
39 – AC Overlay – No problems noted
Affected area (30' x 2' max + 5' x 2') / 5304 = 1.3%

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
		CoRe Elements					
39 Concrete Slab – Unprotected w/ AC Overlay	1 504 m^2 (5304 SF)	EA	1				
215 Abutment	50 (162)	m (LF)	50 (162)				

		Other Elements					

		Smart Flags					
359 Soffit of Concrete Decks and Slabs	1	EA	1				

Topic 7.7 – Precast and Prestressed Slabs

Superstructure documentation exercise (notes and sketch)

Total Quantity

Slab Beams: *1 beam x 10 m = 10 m
1 beam x 32.5’ = 32.5’
6 x 10 m = 60 m
(6 x 32.5’ = 195 LF)
Slab: 10 m x 1.2 m/beam x 6 beams = 72 m^2
32.5’ x 4’/beam x 6 beams = 780 SF

Inspection notes:

Underside of slab units showed minor scaling and spalling of the concrete along the joints with signs of leakage. Lateral posttension ties showed no defects. Asphalt wearing surface did not have any reflective cracks.

Check List

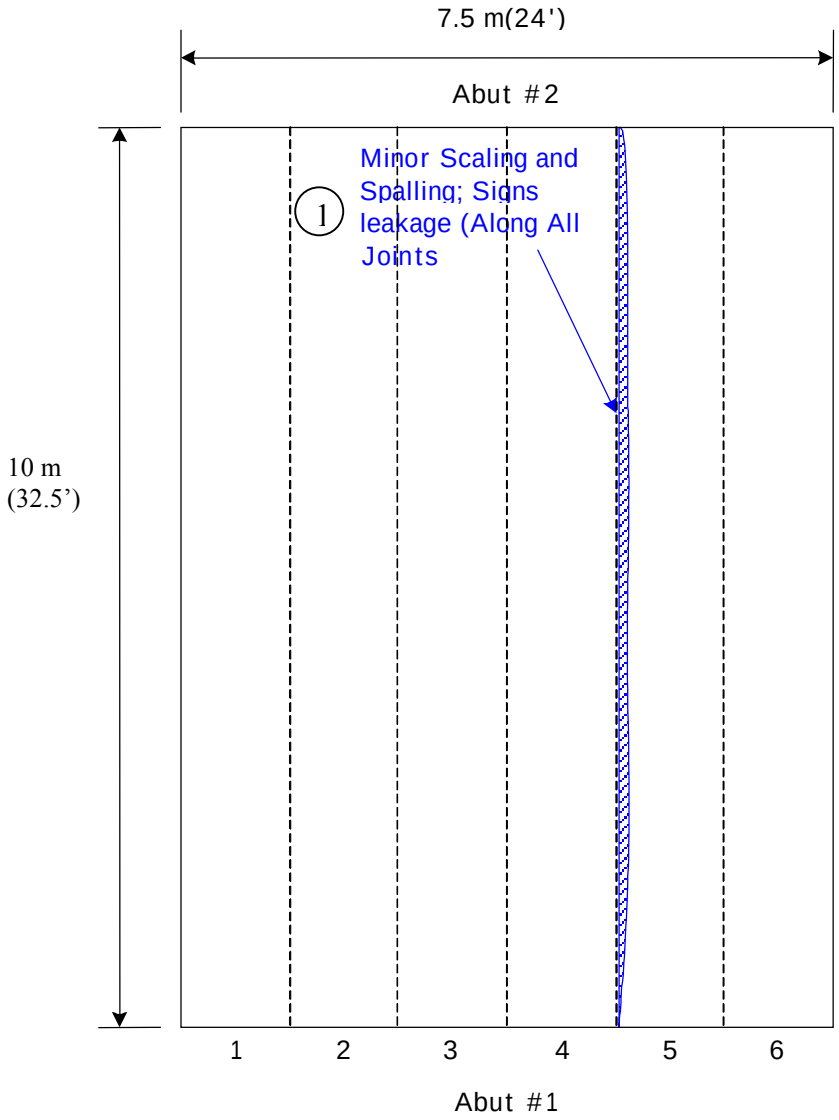
√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)



Plan View

Member **Closed Web/Box Girder**

① Signs of leakage at all joints (see photos 5 – 7)

*AASHTO Method of counting beams (similar to channel beams Topic 7.4) Number of exterior webs / 2

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.7.10 – 7.7.12. Rate the subject prestressed concrete slab bridge.

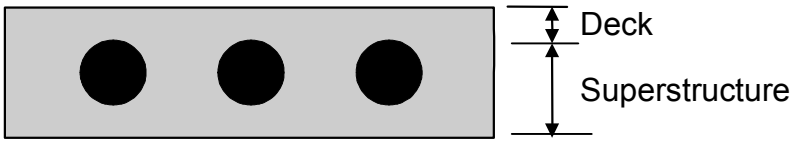
NBI Component Rating Method:
Circle the rating for the superstructure and deck and explain why these ratings were chosen (rating reasoning).

Superstructure Condition Rating = 9 8 (7) 6 5 4 3 2 1 0

Deck Condition Rating = 9 8 (7) 6 5 4 3 2 1 0

It is in good structural condition. The primary structural elements (slab units) show only minor deterioration and damage. Two rust stains, a little joint leakage, occasional edge deterioration at some joints are too minor to warrant a "6"

Area provided for notes or calculations:



Instructor Note: No smart flags for deck/slab since neither top or bottom surfaces visible.

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
		CoRe Elements					
104 Closed Web/Box Girder	10 (33) 60 (195)	m (LF) m (LF)		10 (33) 60 (195)			
*39 Concrete Slab – Unprotected w/ AC Overlay	1 72 m^2 (780 SF)	EA m^2 SF	1				

		Other Elements					

		Smart Flags					
* *							

* Top of slab beam serves as a deck – we have to have a deck (No reflective cracking on asphalt W.S.)
* * No soffit since under side of superstructure (not) deck or slab visible

Topic 7.9 – Prestressed I-Beams

Superstructure documentation

Total Quantity for Element Level Inspection

Beams: 12 m x 10 beams x 2 spans = 240
+ 21 m x 14 beams x 2 spans = 588 = 828 m
(40' x 10 beams x 2 spans = 800
+ 69' x 14 beams x 2 spans = 1932 = 2732 LF)

Inspection notes:

Collision damage to bottom flange of Beam #1 – Span 2 with severed prestressing strand. There is evidence of additional collision damage to same spot. No cracks discovered.

Check List

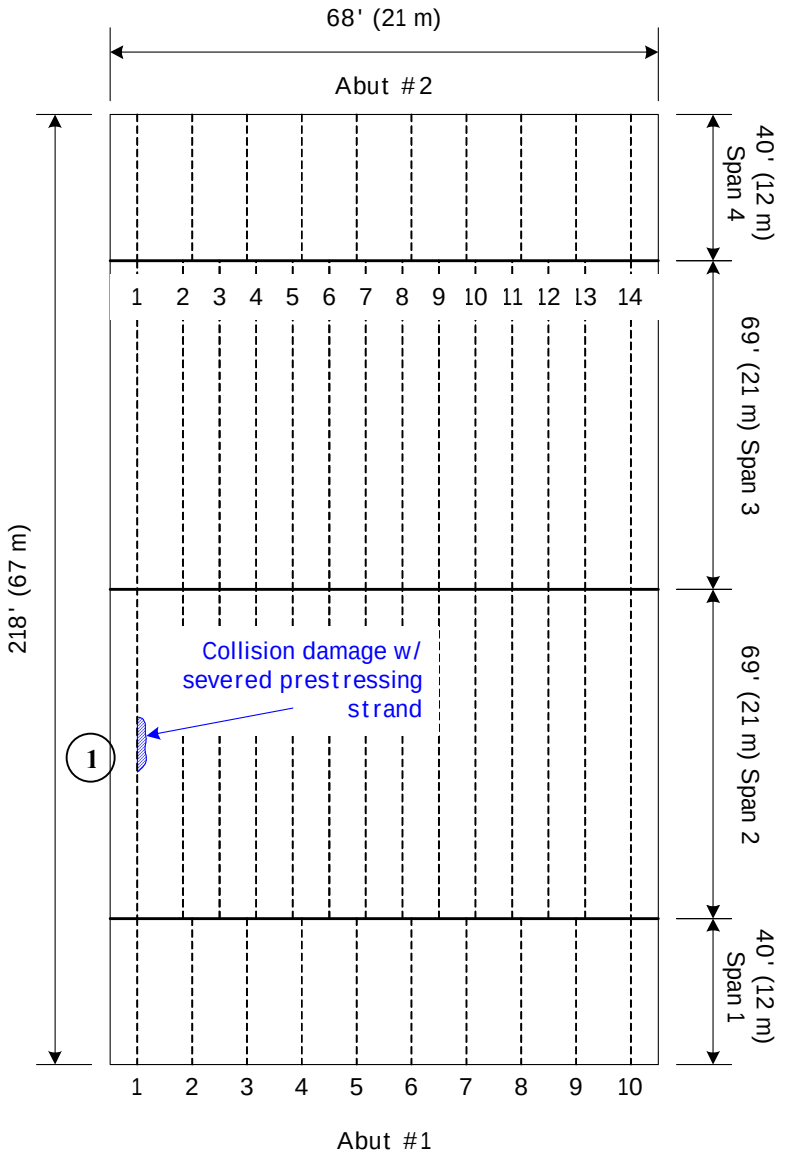
√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)



Plan View

(1) Collision damage with severed prestressing strand (see photos 3, 4 and 5)

Member **Prestressed Concrete Open Girder/Beam**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.9.12 – 7.9.14. Rate the subject prestressed I-beam bridge.

NBI Component Rating Method:
Circle the rating for the superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 (5) 4 3 2 1 0

It is in fair structural condition. The rating reason includes the following:

- **The severed strand is example of local failure.**
- **Noncritical collision damage to structural element.**
- **Only one strand broken.**
- **No camber loss noted.**
- **No evidence of structural distress.**
- **Large number of beams.**
- **No other cracks noted.**

Area provided for notes and calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
109 Prestressed Concrete Open Girder	833 (2732)	m (LF)	832.5 (2731)			0.5 (1)	
	Other Elements						
	Smart Flags						
362 Traffic Impact	1	EA			1		

Topic 7.10 – Prestressed Box-Beams

Superstructure documentation

Total Quantity

Beams: 12 m x 1 Bm = 12 m **12 m span x 7 = 84 m**
38’ x 1 Bm = 38’ **38’ span x 7 = 266 LF**
Deck: 8.5 m x 12 m = 102 m^2
28’ x 38’ = 1064 SF

Inspection notes:

Transverse cracks found in exposed top flange of Beam #6 for its full length, Beam #3 for 1 m (4’) from Abut 2, Beam #2 for 2 m (6’) form Abut 2, and Beam #4 for 1 m (3’) from Abut 1. Joints b/n box beams typically leak, but no sign of rust stains or damage of the reinforcement. Transverse flexure cracks also found in the center 2 m (6’) of the bottom flange of four box beams. Longitudinal crack and spall on the underside of Beam #6 extending from Abut 1 for 5 m (15’) towards midspan.

Check List

√ Member Description

√ Defect Locations

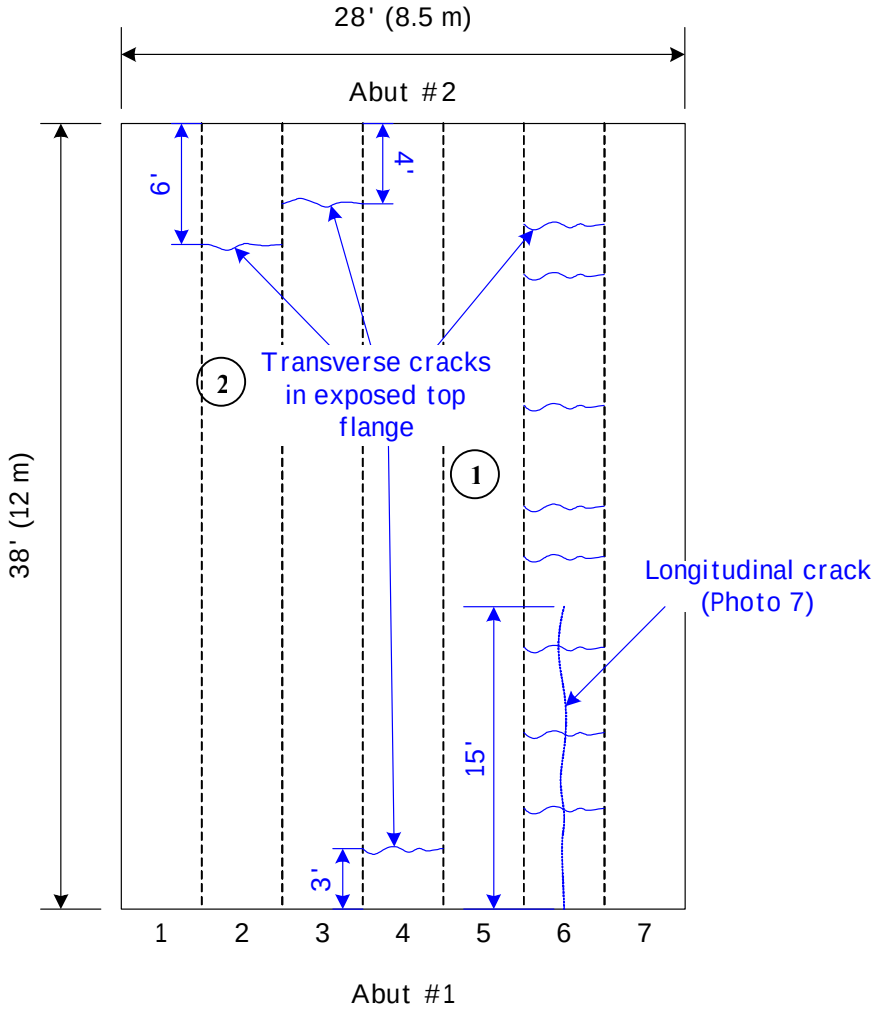
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

(1) Transverse cracks were found throughout beam #6. (see photos 2 & 3)

(2) Transverse flexure cracks found in center 6’ of the bottom flange of four box beams. (see photos 4 – 6)



Plan View

Member **Prestressed Concrete Closed Web/Box Girder**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.10.13 – 7.10.15. Rate the subject prestressed box beam bridge.

NBI Component Rating Method:
Circle the rating for the superstructure and deck and explain why these ratings were chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 5 (4) 3 2 1 0
Deck Condition Rating = 9 8 7 6 5 (4) 3 2 1 0

It is in poor structural condition. The primary structural members, the box beams, have structural cracks and advanced deterioration and spalling. The top flange transverse cracks are not due to flexure. They are a result of insufficient dead load and likely have been present since the beams were erected. Likewise, the longitudinal crack on the bottom of one beam is again not likely due to flexure, but more probably due to premature release of one or two strands at that corner. The longitudinal crack, however, is worsening. The transverse cracks in the bottom flange of the four beams are flexure cracks and it's relevant that they are "working" cracks, with loss of camber and differential deflection noted.

Area provided for notes or calculations:

Multiple Girders	Single Girder
CS4 Photos 4 – 6: 4 (2 m) = 8 m (4 beams) 4 (6') = 24'	**CS4** 2 m (6')
CS3 Photo 7: 5 m or 15'	**CS3** 5 m (15')
CS2 84 m – 5 m – 8 m = 71 m 266' – 15' – 24' = 277'	**CS2** 12 m – 5 m – 2 m = 5 m 38' – 6' – 15' = 17'

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
CoRe Elements							
104 Prestressed Concrete Closed Web/Box Girder	12 (38) 84 (266)	m (LF) m (LF)		5 (17) 71 (227)	5 (15) 5 (15)	2 (6) 8 (24)	
12 Concrete Bare Unprotected	1	EA	1				
Other Elements							
Smart Flags							
358 Deck Cracking	1	EA			1		

Topic 7.11 – Concrete Box Culverts

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.11.24 – 7.11.26. Rate the subject segmental concrete box girder bridge.

NBI Component Rating Method:
Circle the rating for the superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 (7) 6 5 4 3 2 1 0

The superstructure is in good condition. Light spall (minor) occurs only on several random segments. Spall is determined to be light and it occurs within the 13 mm (½") wearing surface only.

Element Level Inspection Method:

- Determine the CoRe Elements name and number.
- Calculate total quantities.
- Determine correct condition states.

Use the current (or Agency CoRe Element Guide if supplied) AASHTO CoRe Element Guide.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
CoRe Elements							
104 Prestressed Closed Web/Box Girder	305 (1000)	m (LF)	182 (600)	123 (400)			
22 Concrete Deck Protected with Rigid Overlay	1 (3,438 m^2) (37,000 SF)	EA (m^2) (SF)		1 (3,438 m^2) (37,000 SF)			

Other Elements							

Smart Flags							

Topic 7.12 – Concrete Box Culverts

Culvert documentation exercise (notes and sketch)

Total Quantity
Culvert Length: 13.5 m x 1 barrel = 13.5 m
45’ x 1 barrel = 45’

Inspection notes:
Cracking with efflorescence on left portion of wingwall. Roadway above with cracking and patched areas. Top of culvert with heavy efflorescence, significant cracking and large spall. 2 mm (1/16”) wide transverse cracks on walls on top slab and walls of culvert typical throughout length.

Check List

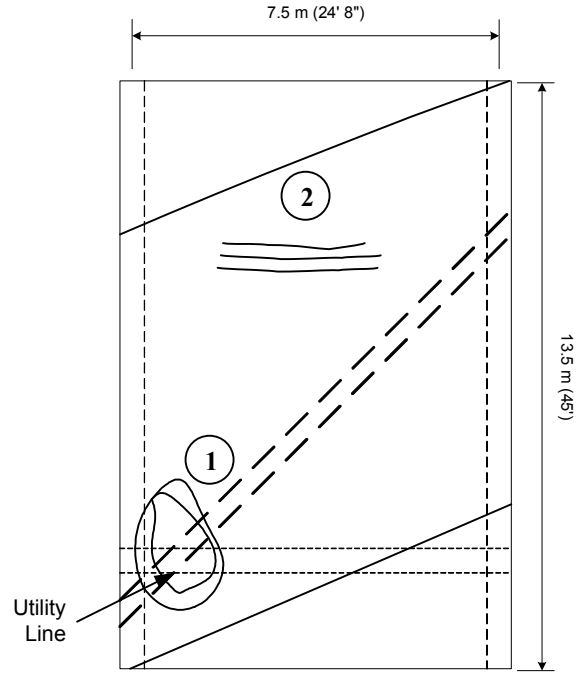
√ Member Description

√ Defect Locations

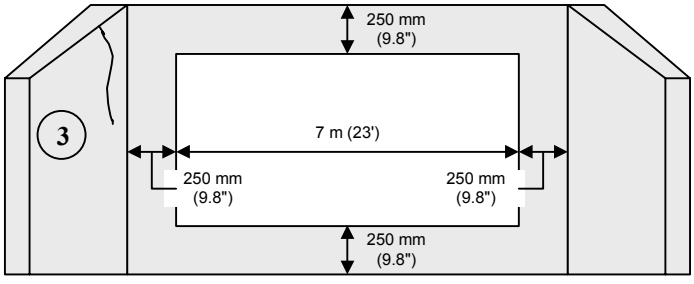
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)



Plan View



Elevation View

(1) Large spall, 1 m (3’) in diameter, above utility line (see photo 3)

(2) Cracking, efflorescence typical throughout (see photo 4)

(3) Cracking with efflorescence on left portion of wingwall (see photo 1)

Member **Concrete Culvert**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 7.12.9 – 7.12.10. Rate the subject concrete box culvert.

NBI Component Rating Method:
Circle the rating for the culvert and explain why this rating was chosen (rating reasoning).

Culvert Condition Rating = 9 8 7 6 (5) 4 3 2 1 0

There are no alignment problems or settlement to speak of. The rating then will be determined by the condition of the concrete. The concrete has efflorescence, spalls, and cracks. The spalls are isolated but the efflorescence and cracks are significant and are widespread.

Since the cracking and efflorescence is extensive, the best rating for this culvert is a "5".

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
241 Concrete Culvert	13.5 (45)	m (LF)			13.5 (45)		

	Other Elements						

	Smart Flags						

Topic 8.2 – Steel Multi-Beams/Girders

Superstructure documentation exercise (notes and sketch)

Total Quantity

Total beam quantity =

7 beams x 9.45m L = 66.15 m say 66 m

7 beams x 31' L = 217 LF

Inspection notes:

Rolled steel beams with typical random pitting throughout the beams. The max. depth of pitting is 1/16" (2 mm). Typical surface rust along top flange of beams 2 mm. (1/16") section loss to top flange of beam #1 for 0.6 m (2') from south abutment. Welded connection @ pile support – no cracks

Check List

√ Member Description

√ Defect Locations

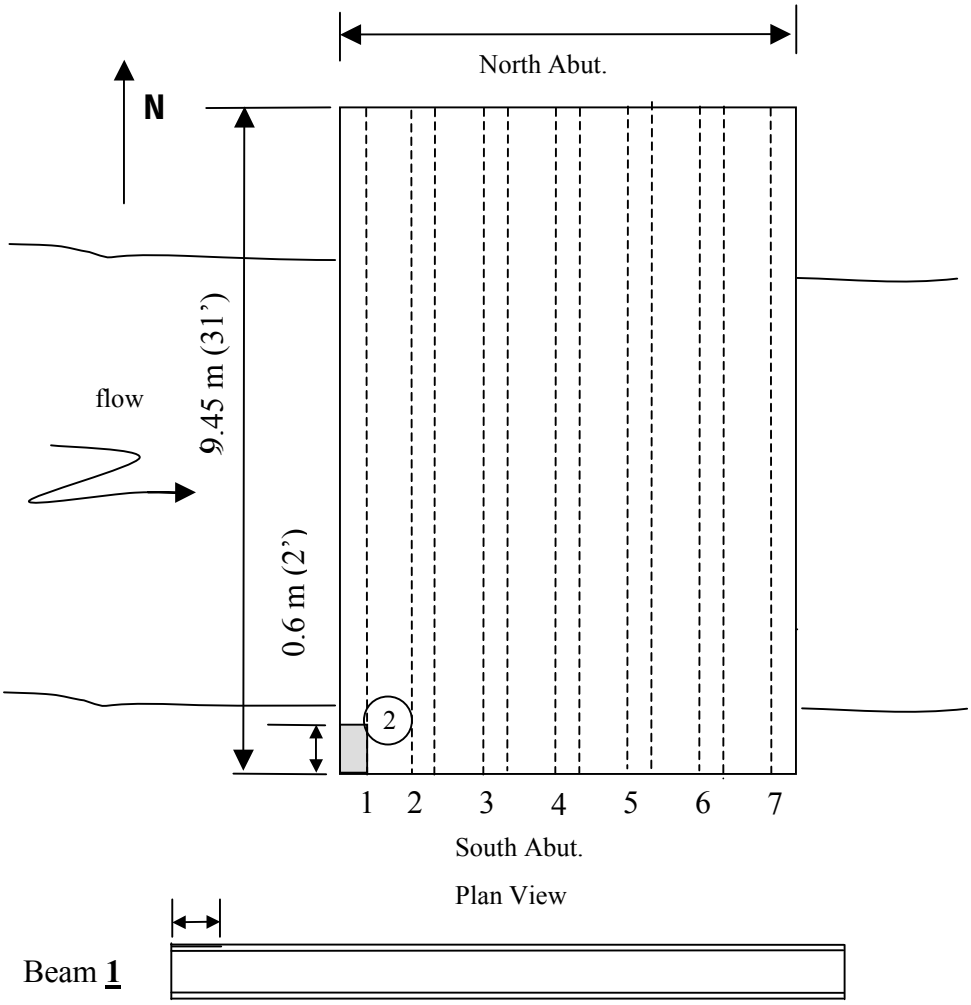
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

(1) Typical random pitting up to 2 mm (1/16") typical throughout beams. (see Photo 4).

(2) 2 mm (1/16") section loss to top flange of beam for 0.6 m (2') length. (see Photo 3).



Member **STEEL GIRDER**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 8.2.15 – 8.2.16. Rate the subject steel beam superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 **(5)** 4 3 2 1 0

Superstructure is in "fair" condition structurally. The beams which are the primary structural members, are generally sound with minor section loss. The pitting is random.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
CoRe Elements							
107 Painted Steel Open Girder	66 (217)	M (LF)				66 (217)	

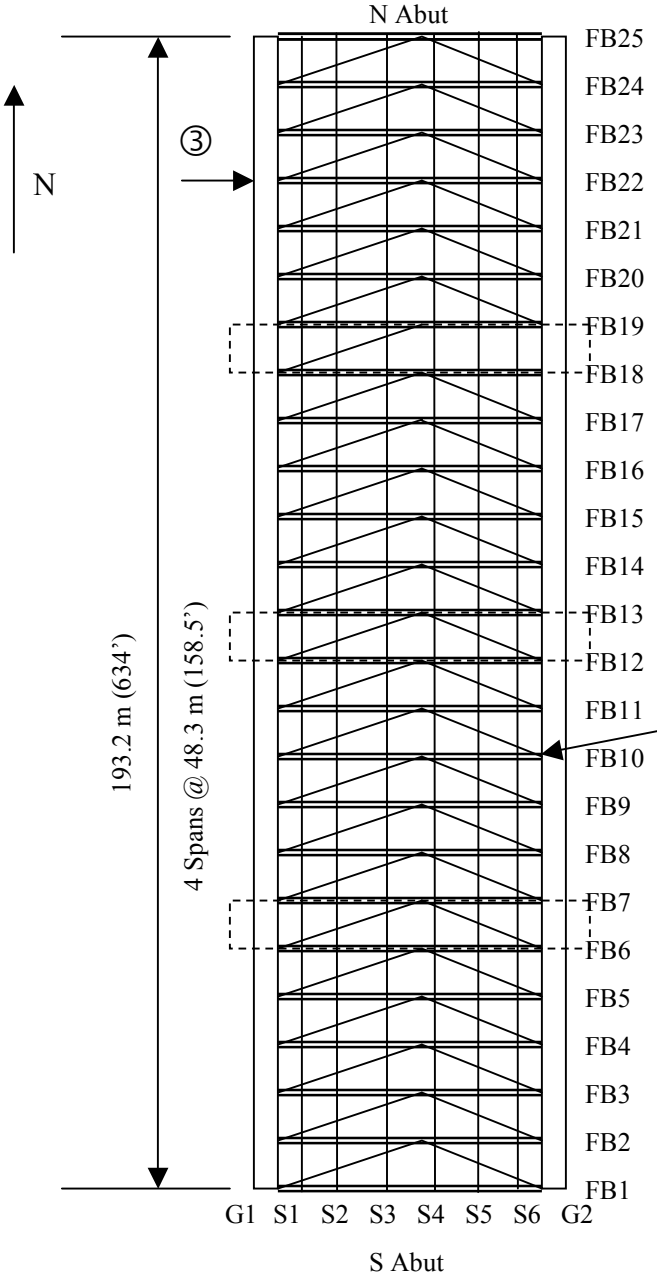
Other Elements							

Smart Flags							
363 Section Loss	1	EA		1			

Topic 8.3 – Steel Two-Girder System

Superstructure Documentation Exercise (sketch)

Framing plan



② Floorbeam Web Crack Table

FB#	G#	Crack Size	Year
3	1	1/2	2002
3	2	1/2	
6	1	3/4	
7	1	1/4	
7	2	1	
9	1	1	
9	2	1/2	
11	1	1/2	
11	2	11/16	
12	1	5/8	
12	2	1/2	
13	1	1/2	
13	2	3/8	
14	2	3/4	
16	1	11/16	
17	1	13/16	
18	1	3/4	
18	2	3/8	
20	2	1 3/4	
21	2	13/16	
23	1	1	

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

①19 mm (¾") vertical crack in web stiffener weld of girder #2 @ north elevation of FB#10
(see Photo 3)

② Floorbeam web cracks
(see Photos 4-7)

③ 4 missing welds @ longitudinal web stiffener on west elevation of girder #1, span 4
(see Photo 8)

Member **Open Steel Girder**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 8.3.16 – 8.3.19. Rate the subject steel two-girder superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 5 **(4)** 3 2 1 0

Superstructure is in "poor" condition structurally. The floorbeams are primary structural members and are typically found to have fatigue cracks due to poor detail at critical locations.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
CoRe Elements							
107 Painted Steel Open Girder	386 (1268)	m (LF)	386 (1268)				
113 Painted Stringers	1159 (3804)	m (LF)	1159 (3804)				
152 Painted Floor Beams	259 (850)	m (LF)	259 (850)				

Smart Flags							
356 Steel Fatigue	1	EA			1		

Topic 8.4 – Pin and Hanger Assemblies

Rating Exercise

- Determine the CoRe elements name and number
- Calculate total quantities
- Determine correct condition states

Element Level Method:

Use the current AASHTO CoRe Element Guide. (Use agency core element guide if supplied)

Use the photo, and field conditions given below to determine the condition state rating of the pin and hanger assembly.



Photo 1: Close up of bottom pin and pin cap in a painted pin and hanger assembly. The top pin is in similar condition. There is corrosion with minor section loss around the pin cap and on the hanger in various locations. The superstructure consists of two deck girders. This condition is typical for both pin & hanger assemblies.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
161 Pin and Hanger Assembly Painted (Pg. 25)	*CoRe Elements*						
	2	EA				2	

	Smart Flags						
363 Section Loss (Pg. 49)	1	EA		1			

Topic 8.5 – Steel Box Girders

Superstructure documentation

Total Quantity:

Closed Box Girder = 2 EA x 222.5 m L = 445 m or
2 EA x 730' L = 1460 LF

Open Girder (Outriggers) = 74 EA x 6.1 m L x 2 faces= 902 m
74 EA x 20' L x 2 faces = 2960 LF

Exp. Bearings = 4 locations X 2 EA = 8 EA

Fixed Bearings = 1 location x 2 EA = 2 EA

Steel Fatigue = 1 crack location = 1 EA

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

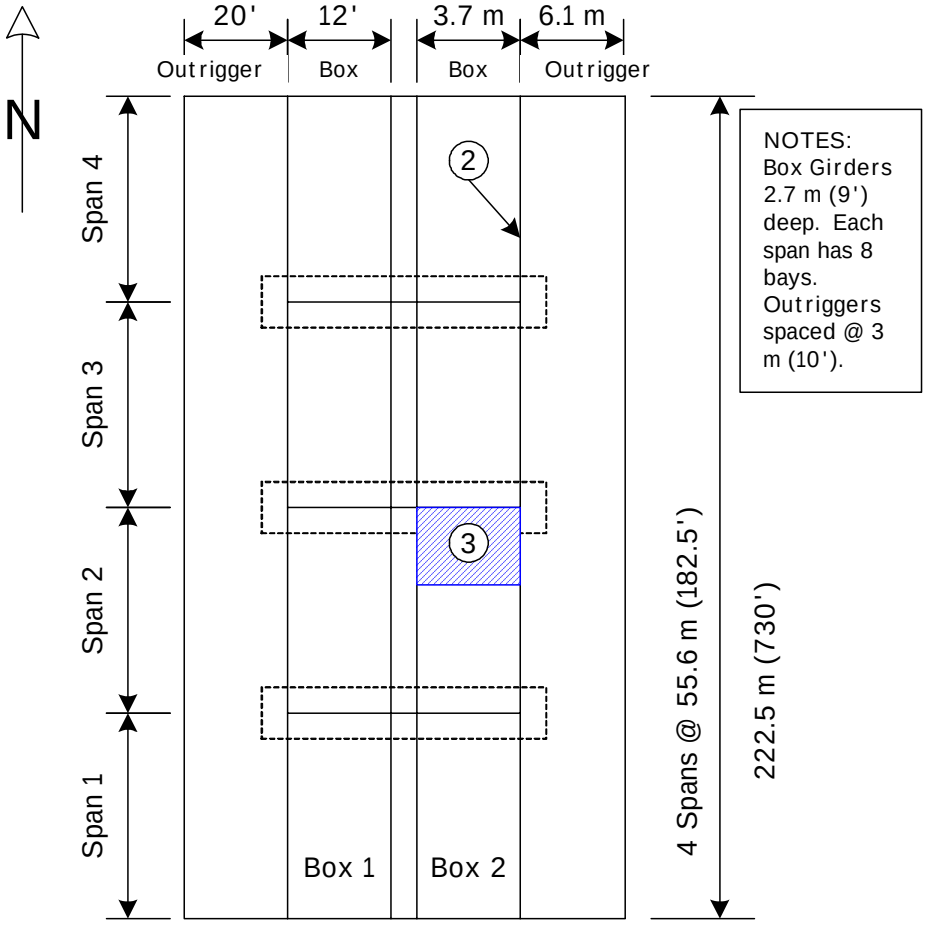
√ Defect Dimensions

√ Defect Quantity (if required)

Inspection notes:

Failure of paint system on insides of boxes w/ surface rust along top of bottom flange plate. A 184 mm (7 ¼") horizontal fatigue crack was found in longitudinal web stiffener weld located in span 4, Box 2, inside east web between diaphragms 1 & 2. Fire damage w/ heavy debris in Span 2, Box 2, Bays 6, 7 & 8. Cantilevered outriggers in good condition. Bearings in good condition.

① Typical failure of paint system on inside of boxes with surface rust on top of bottom flange.
(see Photos 3 &4)
② 184 mm (7 ¼") crack Span 4, Box 2, east web.
(see Photo 5)
③ Fire damage Span 2, Box 2, Bays 6, 7, and 8.
(see Photos 6 & 7)



Member Steel Closed Box Girder

Practice Unit

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 8.5.13 – 8.5.15. Rate the subject steel beam superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 (6) 5 4 3 2 1 0

Superstructure is in "fair" condition structurally. All structural elements are sound but have minor deterioration. There is a 184 mm (7 ¼") crack in longitudinal stiffener weld in compression area. Fire damage did not appear to cause any structural damage to the bottom flange or webs (deck damage is rated separately). There is paint peeling with surface rust. 10% of remaining steel exhibits pitting.

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
102 Painted Steel Closed Web Box Girder	445 (1460)	m (LF)			445 (1460)		
107 Painted Steel Open Girder	902 (2960)	m (LF)	902 (2960)				

	Smart Flags						
356 Steel Fatigue	1	EA		1			

Topic 8.6 – Steel Trusses

Superstructure documentation exercise (notes and sketch)

Total Quantity

Truss = 2 EA x 34.5 m L = 69 m or
(2 EA x 113' L = 226 LF)
Stringers = 5 EA x 34.5 m L = 172.5 m say 172 m
(5 EA x 113' L = 565 LF)
Floorbeams = 9 EA x 8.2 m L = 73.8 m say 74 m or
(9 EA x 27' L = 243 LF)

Inspection notes:

Both trusses in generally satisfactory condition. All FB's have 2 mm (1/16") to 6 mm (¼") section loss along top & bottom flanges typical. All stringers have 2 mm (1/16") to 5 mm (3/16") section loss on top & bottom flanges & web for 2 LF from FB connections see sketch.

Check List

√ Member Description

√ Defect Locations

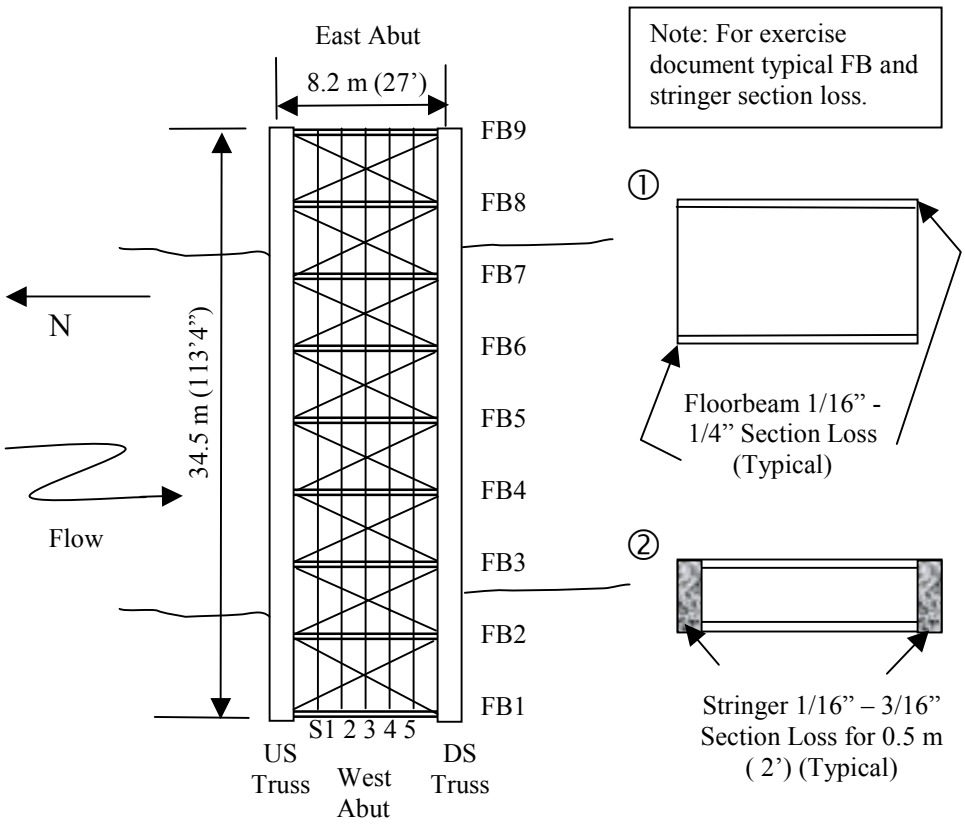
√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

① Floorbeams have 1/16" – ¼" section loss on top & bottom flanges typical. (see Photos 6B – 12)

② Stringers have 1/16" – 3/16" section loss on web & flanges for 2' from connection to FB typical. (see Photos 6B – 12)



Member **Painted Steel Truss**

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 8.6.25 – 8.6.28. Rate the subject steel truss superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 5 **(4)** 3 2 1 0

SUPERSTRUCTURE IS IN "POOR" CONDITION STRUCTURALLY. THE PRIMARY STRUCTURAL MEMBERS (FLOOR SYSTEM) HAVE SIGNIFICANT AND WIDE SPREAD SECTION LOSS. STRUCTURAL CAPACITY OF THE FLOOR SYSTEM IS ASSUMED TO BE ADVERSELY AFFECTED.

Area provided for notes or calculations:

(Metric) Stringers (8 sets) (5 stringers/set) (0.5 m /end) (2ends) = 40 m

(English) Stringers (8 sets) (5 stringers/set) (2' /end) (2ends) = 160'

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
CoRe Elements							
126 Through Truss Excluding Bottom Chord	69 (226)	m (LF)	69 (226)				
121 Through Truss Bottom Chord	69 (226)	m (LF)	69 (226)				
113 Painted Stringers	172 (565)	m (LF)	132 (405)				40 (160)
152 Painted Floor Beams	74 (243)	m (LF)					74 (243)
Smart Flags							
363 Section Loss	1	EA			1		

Topic 8.8 – Steel Arches

Superstructure documentation exercise (notes and sketch)

The superstructure of the arch span consists of solid ribbed box members. The floorbeams are fabricated girders, which are approximately 2438 mm (8') deep and spaced at 9449 mm (31'). The stringers are 533 mm (21") deep rolled wide flange beams. Top lateral bracing is fabricated I-girders.

Check List

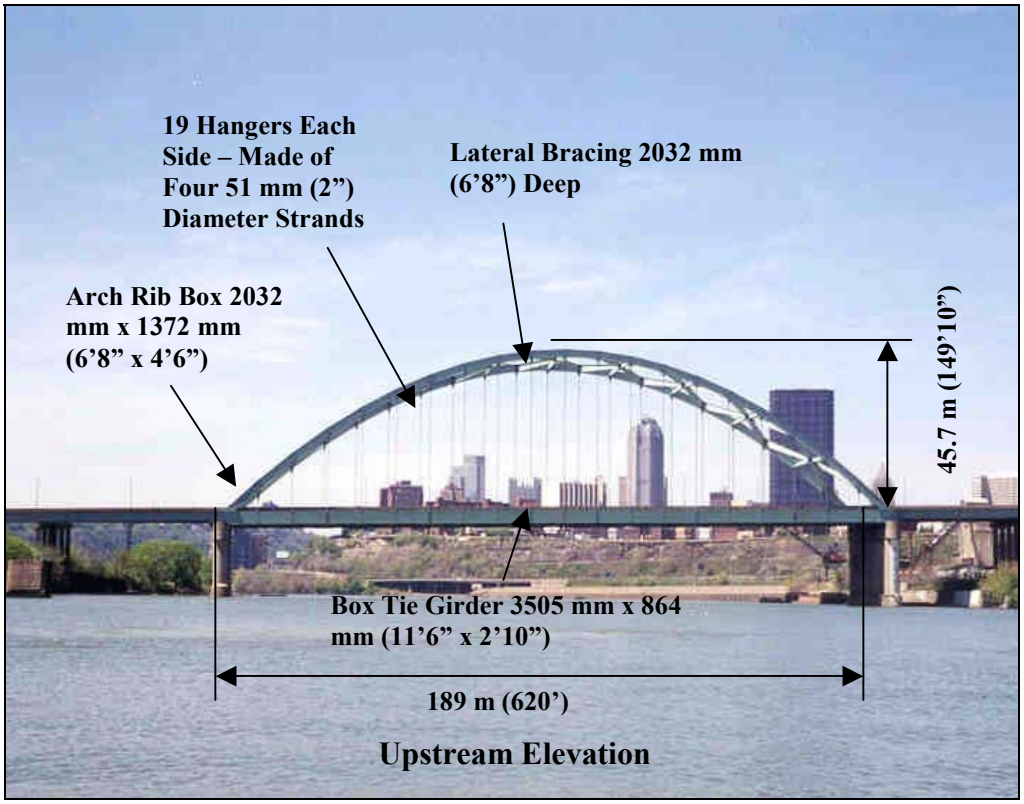
√ Member Description

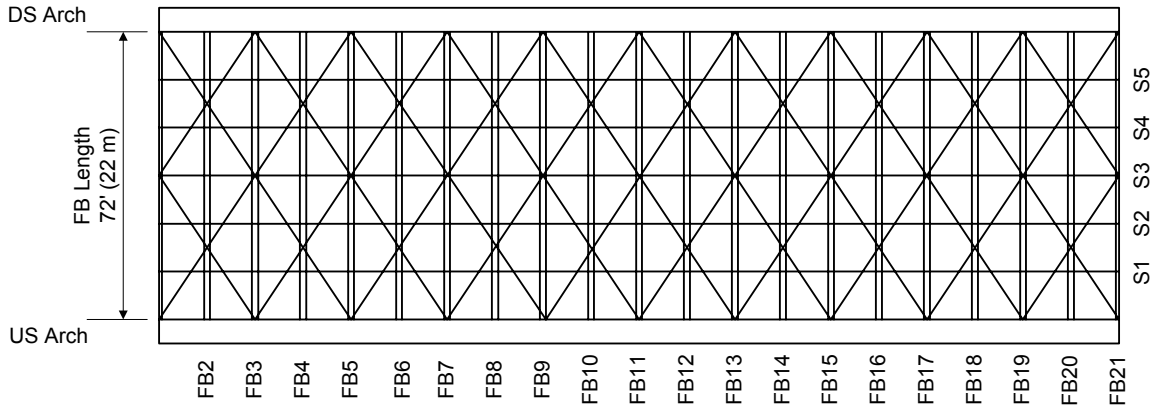
√ Defect Locations

√ Defect Description /Severity

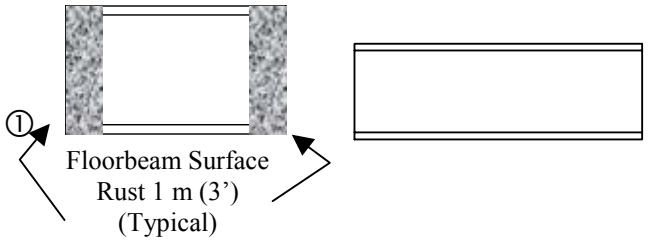
√ Defect Dimensions

√ Defect Quantity (if required)





① Minor surface rust at FB connection to tie box girder typical. (see Photo 2)



Note: For exercise document typical FB and stringer section loss.

Superstructure documentation exercise (notes and sketch)

Total Quantity

Arch = 2 EA x 189 m L = 378 m or
2 EA x 620' L = 1240 LF
Closed Box Girder = 2 EA x 189 m = 378 m or
2 EA x 620' L = 1240 LF
Cable = 19 Hangers x 2 sides = 38 x 4 Cables
= 152 EA
Stringers = 5 EA x 189 m L = 945 m or
5 EA x 620' L = 3100 LF
Floorbeams = 21 EA x 22 m L = 462 m or
21 EA x 72' L = 1512 LF

Inspection notes:

No defects were found in both arch rib members. Box tie girders typically in good condition with minor surface rusting throughout. Stringers generally in good condition. The upper and lower hanger connections appear to be in good condition, as do the cables themselves. The floorbeams typically show minor surface rusting at the box tie girder connections. Numerous cracks were found in FB web-to-top flange welds near box tie girder connection of both arches. 77 cracks total; 5 cracks propagated into FB base metal. Crack lengths vary from 13 – 146 mm (½" – 5 ¾").

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

② Minor surface rusting typical inside box girders. (see Photo 3)

③ 77 cracks at FB top flange-to-web weld near the box tie girder connection of both arches. (see Photo 4)

Topic 8.8 – Steel Arches (continued)

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 8.8.20 – 8.8.22. Rate the subject steel tied arch superstructure.

NBI Component Rating Method:
Circle the rating for this superstructure and explain why this rating was chosen (rating reasoning).

Superstructure Condition Rating = 9 8 7 6 (5) 4 3 2 1 0

Superstructure is in "fair" condition structurally. The primary structural elements are sound, but have out-of-plane distortion cracks caused by the longitudinal movement of the floor system. There are 77 such crack locations in FB top flange-to-web welds.

Area provided for notes or calculations:

Floorbeams – Surface Rust CoRe 152
1 m x (2 ends/FB x 21 FB) = 42 m CS 3
3' x (2 ends/FB x 21 FB) = 126' CS 3

462 – 42 = 420 m CS 1
1512 – 126 = 1386' CS 1

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
Painted Steel	*CoRe Elements*						
102 Closed Web/Box Girder	378 (1240)	m (LF)			378 (1240)		
113 Stringer	945 (3100)	m (LF)	945 (3100)				
141 Arch	378 (1240)	m (LF)	378 (1240)				
152 Floorbeam	462 (1512)	m (LF)	420 (1386)		42 (126)		
146 Uncoated Cable (not in concrete)	152	EA	152				
	Smart Flags						
356 Steel Fatigue	1	EA			1		

Topic 9.1 – Bridge Bearings

Area provided for notes or calculations:

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	Core Elements						
311 Moveable Bearing	2	EA		2			
313 Fixed Bearing	2	EA	2				

	Other Elements						

	Smart Flags						

Topic 10.1 – Abutments

Substructure documentation exercise (notes and sketch)

Total Quantity

Total abutment quantity = 2 abuts x 18 m L =36 m
2 abuts x 60' L = 120 LF

Inspection notes:

North abutment has full ht vert crack 2 – 3 mm (1/16" – 1/8") wide w/ water leakage, delam & efflo. 1st exp jt has opened 89 mm (3 ½") @ top this is 12 mm (1/2" more than last insp. South abut has cracked, delam bearing seat w/ water leakage & rust stains. S. abut has full ht vert crack 2 mm (1/16") wide w/ delam, efflo & rust stains.

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

√ Defect Quantity (if required)

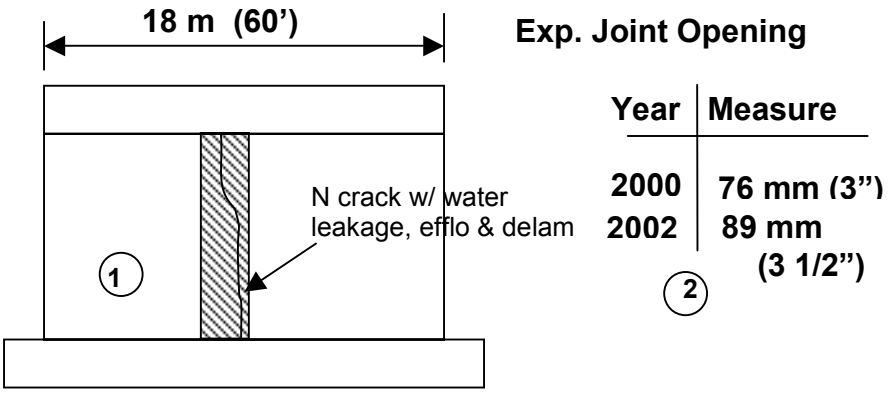
(1) Full height vertical crack 2 –3 mm (1/16" – 1/8"). (see Photo 3)

(2) First exp. Joint from west end of north abut. (see Photo 2)

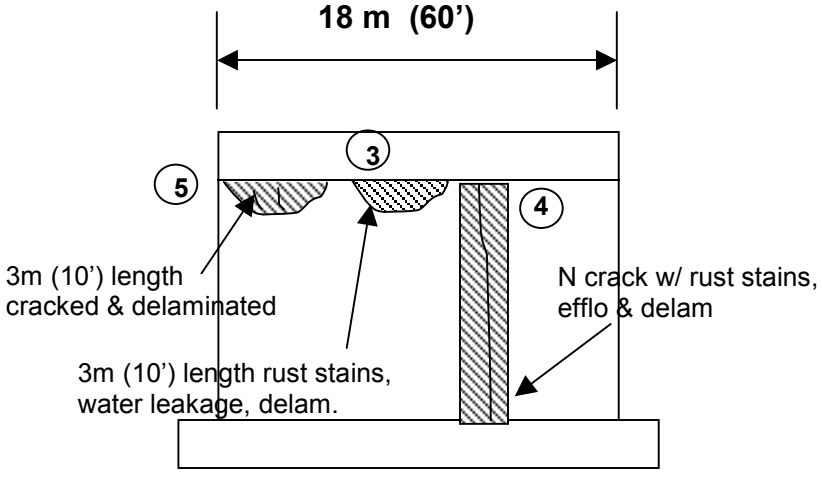
(3) Bearing seat of south abut (see Photo 6)

(4) South abut. 2 mm (1/16") crack. (see Photo 7)

(5) Bearing seat of south abut east corner (see Photo 8)



South Abutment



Rating Exercise

Use the photos, field conditions, and dimensions given on pages 10.3.2 – 10.3.4. Rate the subject reinforced concrete substructure.

NBI Component Rating Method:
Circle the rating for this substructure and explain why this rating was chosen (rating reasoning).

Substructure Condition Rating = 9 8 7 6 **(5)** 4 3 2 1 0

Substructure is in "fair" condition structurally. There is minor deterioration, cracking, and spalling of the concrete that requires documentation. Bearing areas are somewhat affected by the deterioration, but the primary elements (abutments) are still sound.

Area provided for notes or calculations:

Cracked & delaminated bearing areas of south abutment:
2 areas X 3 m (10') each = 6 m (20') in CS 3

Full ht vertical cracks 2 – 3 mm (1/16" – 1/8") with efflorescence & delamination, one (1) at each abutment:
Assume a 0.5 m (1') wide area x 2 cracks = 1 m (2') CS 2

Remainder of abutment length goes into CS 1:
36 m (120') – 6 m (20') – 1 m (2') = 29 m (98')

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	CoRe Elements						
215 Abutment	36 (120)	m (LF)	29 (98)	1 (2)	6 (20)		

	Other Elements						

	Smart Flags						
360 Settlement	1	EA			1		

Topic 10.2 – Piers and Bents

Substructure documentation exercise (notes and sketch)

Total Quantities:

Pier caps = 3 caps x 14 m L= 42 m
3 caps x 45' L = 135'
Column = 3 piers x 2 EA = 6 EA

Inspection notes:

Abuts: N. - good condition
F. - 2 mm (1/16") cracking typ.
Pier #1: Cap - delam, spalling, efflo & exp rebar
N. column - severe spalling & rebar w/ section loss
Pier #2: N. column - cracking, delam & 6 mm (¼") full ht crack
S. column - efflo, delam & severe spallinq w/ missing conc & rebar w/ loss
Pier #3: N. column - hairline cracks & repairs

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

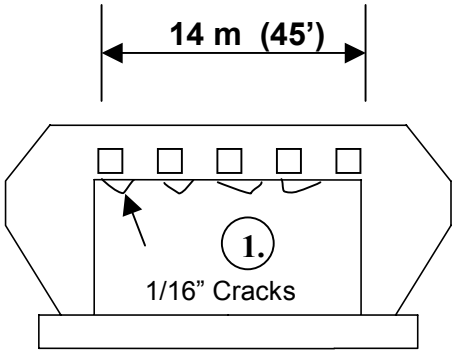
√ Defect Dimensions

√ Defect Quantity (if required)

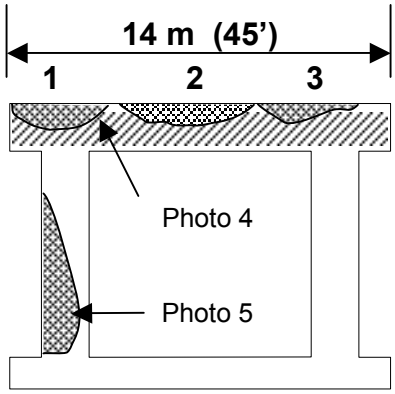
1. Typical cracking and separation of concrete (see photos 2A & B).

Delaminated

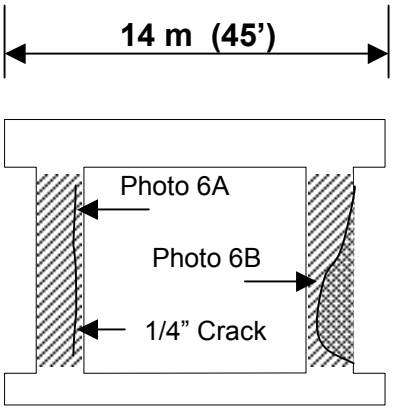
Spalled



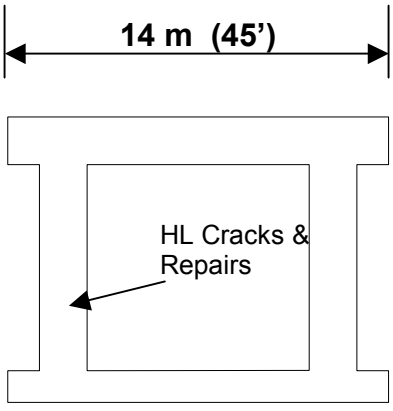
Far (east) Abutment



West Face Pier No. 1



West Face Pier No. 2



West Face Pier No. 3

N

Rating Exercise

Use the photos, field conditions, and dimensions given on pages 10.3.6 – 10.3.8. Rate the subject reinforced concrete substructure.

NBI Component Rating Method:
Circle the rating for this substructure and explain why this rating was chosen (rating reasoning).

Substructure Condition Rating = 9 8 7 6 5 **4** 3 2 1 0

Substructure is in "poor" condition structurally. The piers show excessive disintegration of the concrete and advanced corrosion of the reinforcing bars.

Area provided for notes and calculations:

Pier	Column	CS
1	North	4
	South	1
2	North	2
	South	4
3	North	2
	South	1

Element	Total Quantity	Unit	Quantities in Condition States				
			1	2	3	4	5
	Core Elements						
234 RC Cap	42 (135)	m (LF)	28 (90)		14 (45)		
205 RC Columns	6	EA	2	2		2	

	Other Elements						

	Smart Flags						

Topic 10.3 – NBI Case Studies

Substructure documentation exercise (notes and sketch)

Inspection notes:

Both north & south abutments exhibit minor HL cracks & spalling throughout. S abut blackened br seat from fire w/ no damage noted. N abut ftg completely exposed w/ no undermining & exhibits heavy abrasion. N abut has moved 3" (76 mm) longitudinally since last insp w/ no rotation noted.

Check List

√ Member Description

√ Defect Locations

√ Defect Description /Severity

√ Defect Dimensions

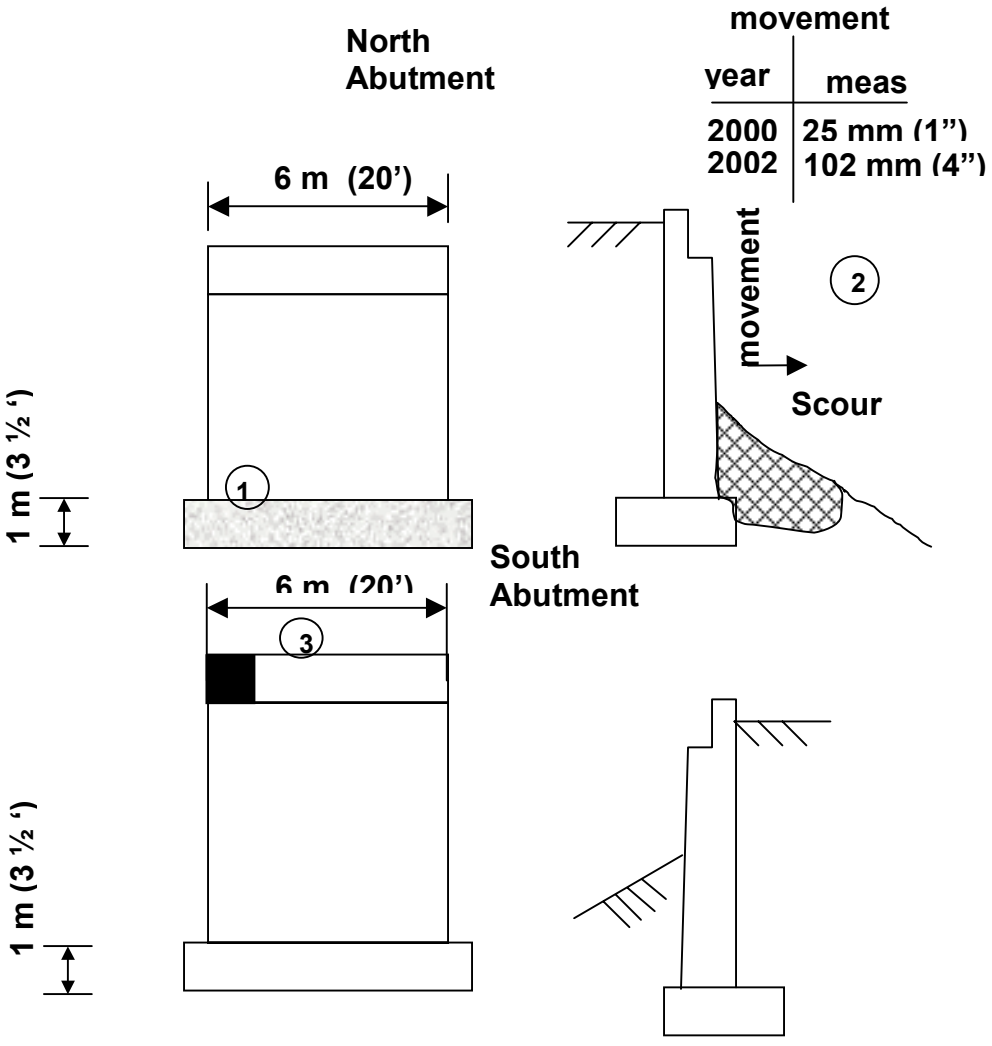
√ Defect Quantity (if required)

① Heavy abrasion of footing. (see Photo 2)

② Exposed ftg w/ no undermine; 3" (76 mm) movement since last insp. (see Photo 3A)

③ Fire blackening (see Photo 3B)

④ Minor spalls & random HL cracks typical (see Photo 4)



Rating Exercise

Use the photos, field conditions, and dimensions given on pages 10.3.10 – 10.3.11. Rate the subject reinforced concrete substructure.

NBI Component Rating Method:
Circle the rating for this substructure and explain why this rating was chosen (rating reasoning).

Substructure Condition Rating = 9 8 7 6 5 4 **3** 2 1 0

Substructure is in "serious" condition structurally. The shifting channel and local scour have caused saturation of the clay foundation material under north abutment. This has resulted in active sliding failure evident by the abnormal tilt of the rocker bearings.

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