Publication No. FHWA NHI 03-001
October, 2002

U.S. Department
of Transportation

**Federal Highway
Administration**

Bridge Inspector's Reference Manual

BIRM

Volume 1

NATIONAL HIGHWAY INSTITUTE
Training Solutions for Transportation Excellence

Publication No. FHWA NHI 03-001
October, 2002

U.S. Department
of Transportation

**Federal Highway
Administration**

Bridge Inspector's Reference Manual

BIRM

Volume 1

NATIONAL HIGHWAY INSTITUTE
Training Solutions for Transportation Excellence

BI RM

Bridge Inspector's Reference Manual

Bridge Inspector's Reference Manual

October, 2002

FHWA NHI Publication No. 03-001

Volume 1

Bridge Inspector's Reference Manual

October, 2002

FHWA NHI Publication No. 03-002

Volume 2

NHI Course No. 130055

Safety Inspection of In-Service Bridges

Participant's Workbook

October, 2002

FHWA NHI Publication No. 03-004

Technical Report Documentation Page

1. **Report No.** FHWA NHI 03-001	2. **Government Accession No.**	3. **Recipient's Catalog No.**
4. **Title and Subtitle** Bridge Inspector's Reference Manual		5. **Report Date** October 2002
		6. **Performing Organization Code**
7. **Author (s)** Raymond A. Hartle, P.E., Thomas W. Ryan, P.E., J. Eric Mann, P.E., Leslie J. Danovich, William B. Sosko, Justin W. Bouscher		8. **Performing Organization Report No.** 23104-HRS
9. **Performing Organization Name and Address** Michael Baker, Jr., Inc. 420 Rouser Road Coraopolis, PA 15108		10. **Work Unit No. (TRAIS)**
		11. **Contract or Grant No.** DTFH61-97-D-00025
12. **Sponsoring Agency Name and Address** Federal Highway Administration (HHI-20) Turner-Fairbank Highway Research Center 6300 Georgetown Pike McLean, Virginia 22101		13. **Type of Report and Period Covered** Final Manual March 2000 - Oct. 2002
		14. **Sponsoring Agency Code**
15. **Supplementary Notes** Baker Principle Investigator: Raymond A. Hartle, P.E. Baker Project Manager: Thomas W. Ryan, P.E. FHWA Contracting Officer's Technical Representative: John M. Hooks, P.E.		

16. **Abstract**

This document, the *Bridge Inspector's Reference Manual (BIRM)*, is a comprehensive manual on programs, procedures, and techniques for inspecting and evaluating a variety of in-service highway bridges. It is intended to replace the *BITM 90* which was first published in 1991 to assist in training highway personnel for the new discipline of bridge safety inspection. *BITM 90* replaced *BITM 70* which had been in use for 20 years and has been the basis for several training programs varying in length from a few days to two weeks. Comprehensive supplements to *BITM 70* have been developed to cover inspection of fracture critical bridge members, and culverts are now covered in the *BIRM*.

The *BIRM* is a revision and upgrading of the previous manual. Improved Bridge Inspection techniques are presented, and state-of-the-art inspection equipment is included. New or expanded coverage is provided on culverts, fracture critical members, cable-stayed bridges, prestressed segmental bridges, and underwater inspection. Previous supplemental manuals on moveable bridge inspection, and nondestructive testing are excerpted and referenced. These supplemental manuals are still valid supplements to *BIRM*.

A three-week comprehensive training program on bridge inspection, based on the *BIRM*, has been developed. The program consists of a one-week course, "Engineering Concepts for Bridge Inspectors," and a two-week course, "Safety Inspection of In-Service Bridges." Together, these two courses meet the definition of a comprehensive training program in bridge inspection as defined in the National Bridge Inspection Standards. The one-week course is optional for technicians, inspectors, or engineers who have an adequate background in bridge engineering concepts.

17. **Key Words** Bridge Inspection, Bridge Evaluation, Culvert Inspection, Fracture Critical Members, Underwater Inspection	18. **Distribution Statement** This report is available to the public from the National Technical Information Service in Springfield, Virginia 22161 and from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

19. **Security Classif. (of this report)** Unclassified	20. **Security Classif. (of this page)** Unclassified	21. **No. of Pages** 1756	22. **Price**

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

This page intentionally left blank

ACKNOWLEDGEMENTS

We would like to express appreciation to the following individuals and organizations who contributed to the development and review of this training program:

Applied Science Associates, Inc.
Association of Diving Contractors, Inc. (Messrs. Castle, Harter, Hazelbaker, Hux, Maggard, McGeehan, and McGovern)
R. Richard Avent, Ph.D., P.E. (Louisiana State University)
William D. Domico, P.E. (Figg Engineering Group)
John W. Fisher, Ph.D. (Lehigh University)
Robert J. Hoyle, Jr., P.E., S.E.
Heinz P. Koretzky, P.E. (Pennsylvania DOT-Retired)
KTA-Tator, Inc.
John A. Schultz, Jr., S.E. (Hazelet and Erdal, Inc.)
Frieder Seible, Ph.D., P.E. (SEQAD Consulting Engineers, Inc.)
Daryl B. Simons, Ph.D., P.E. (Simons and Associates, Inc.)
Robert K. Simons, Ph.D., P.E. (Simons and Associates, Inc.)

Special thanks to the Technical Review Committee:

Steve Belcher (FHWA)
Barry Brecto (FHWA)
Milo Cress (FHWA)
Shelia Dawadi (FHWA)
John Hooks (FHWA)
Raymond McCormick (FHWA)
Curtis Monk (FHWA)
Jorge Pagan (FHWA)
George Romack (FHWA)
Lou Triandafilou (FHWA)
John Thiel (FHWA)
Glenn Washer (FHWA)

We would also like to thank the following individuals and organizations for providing information or visual aids for this training program:

Kathleen H. Almand (American Iron and Steel Institute)
American Traffic Safety Services Association
Bridge Grid Flooring Manufacturers Association
The D.S. Brown Company
Concrete Reinforcing Steel Institute
Cosmec, Inc.
Barry Dickson (West Virginia University)
Dynamic Isolation Systems, Inc.
Carl Edwards (Maine DOT)
Elgard Corporation
Thomas D. Everett (FHWA)
Exodermic Bridge Deck Institute
Paul S. Fisk (NDT Corporation)
Geerhard Haaijer, Ph.D. (American Institute of Steel Construction, Inc.)
John M. Hanson, Ph.D. (Wiss, Janney, Elster Associates, Inc.)
Richard P. Knight, P.E. (Dynamic Isolation Systems, Inc.)
Louisiana Department of Transportation and Development
Merriman, Inc.
William R. Mochel, P.E. (Illinois Department of Transportation)
Pennsylvania Department of Transportation
Professional Service Industries, Inc.-Pittsburgh Testing Laboratory Division
Schupack Suarez Engineers, Inc.
David Severns (Nevada DOT)
Paul W. Verrill (Maine Department of Transportation)
Watson Bowman Associates, Inc.
Stewart C. Watson (Stafford/Watson, Inc.)
Western Wood Structures, Inc.
Weston Geophysical Corporation

We would also like to acknowledge the contributions of staff members at Michael Baker Jr., Inc:

Dennis R. Baughman, P.E.
David Chang
Thomas L. Hooks
Roger W. Hove, P.E.
Maureen Kanfoush
Guy R. Lang, P.E.
Linda Montagna
Sean A. Patrick, P.E.
Timothy Pintar, P.E.
John J. Seibel
J. Keith Seibel
James Shroads
Jorge M. Suarez, P.E.
Scott D. Vannoy, P.E.
Laura E. Volle
Ruth Williams
Kenneth E. Wilson, III, P.E., S.E.

Table of Contents

TOPIC

<u>TOPIC</u>

This page intentionally left blank

List of Figures

Figure No.		Page No.

Topic P.3 Culvert Characteristics

Figure No.		Page No.
4.3.3	Sample Notebook Title Page	4.3.3
4.3.4	Sample Notebook Table of Contents	4.3.4
4.3.5	Sample Notes and Sketches Page	4.3.4
4.3.6	Framing Plan	4.3.5
4.3.7	Girder Elevation	4.3.6
4.3.8	Typical Prepared Culvert Sketches	4.3.7
4.3.9	Sample General Plan Sketch	4.3.8
4.3.10	Sample General Elevation Sketch	4.3.8
4.3.11	Sample Deck Inspection Notes	4.3.9
4.3.12	Sample Superstructure Inspection Notes	4.3.9
4.3.13	Sample Substructure Inspection Notes	4.3.10
4.3.14	Sample Photo Log	4.3.10
4.3.15	Example Inspection Form – PENNDOT Form D-450	4.3.12
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.13
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.14
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.15
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.16
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.17
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.18
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.19
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.20
4.3.15	Example Inspection Form – PENNDOT Form D-450 (Continued)	4.3.21
4.3.16	Core Element Example Inspection Form – Michigan Department of Transportation	4.3.22
4.3.17	Sample Span Numbering Scheme	4.3.24
4.3.18	Sample Typical Section Numbering Scheme	4.3.24
4.3.19	Sample Structure Orientation Sketch	4.3.25
4.3.20	Sample Truss Numbering Scheme	4.3.25
4.3.21	Steel Beam and Girder Dimensions	4.3.26
4.3.22	Truss Member and Field Splice Dimensions	4.3.26

Figure No.		Page No.

Topic 5.4
Deck Joints, Drainage Systems, Lighting and signs

Topic 5.5
Safety Features

Topic 7.6 Concrete Rigid Frames

Figure No.		Page No.

Topic 8.8
Steel Arches

Topic 10.1 Abutments and Wingwalls

Topic 10.2
Piers and Bents

Figure No.		Page No.

Topic 11.3 Underwater Inspection

Topic 12.1 Cable Supported Bridges

Topic 12.3
Concrete Pipe Culverts

Topic 12.4 Flexible Culverts

Figure No.		Page No.

This page intentionally left blank

List of Tables

This page intentionally left blank

Table of Contents

Basic Concepts Primer

Basic Equations of Bridge Mechanics

$S = \frac{F}{A}$ (Page P.1.8)

$f_a = \frac{P}{A}$ (Page P.1.14)

$\varepsilon = \frac{\Delta L}{L}$ (Page P.1.9)

$f_b = \frac{Mc}{I}$ (Page P.1.16)

$E = \frac{S}{\varepsilon}$ (Page P.1.11)

$f_v = \frac{V}{A_w}$ (Page P.1.18)

$$Bridge\ Load\ Capacity\ Rating = \frac{Allowable\ Load - Dead\ Load}{Rating\ Vehicle\ Live\ Load\ Plus\ Impact}\ x\ Vehicles\ Weight\ (Tons)$$

where:

A	=	area; cross-sectional area
A_w	=	area of web
c	=	distance from neutral axis to extreme fiber (or surface) of beam
E	=	modulus of elasticity
F	=	force; axial force
f_a	=	axial stress
f_b	=	bending stress
f_v	=	shear stress
I	=	moment of inertia
L	=	original length
M	=	applied moment
S	=	stress
V	=	vertical shear force due to external loads
ΔL	=	change in length
ε	=	strain

Basic Concepts Primer

Topic P.1 Bridge Mechanics

P.1.1 Introduction

Mechanics is the branch of physical science that deals with energy and forces and their relation to the equilibrium, deformation, or motion of bodies. The bridge inspector will primarily be concerned with statics, or the branch of mechanics dealing with solid bodies at rest and with forces in equilibrium.

The two most important reasons for a bridge inspector to study bridge mechanics are:

- To understand how bridge members function
- To recognize the impact a defect may have on the load-carrying capacity of a bridge component or element

While this section presents the basic principles of bridge mechanics, the references listed in the bibliography should be referred to for a more complete presentation of this subject.

P.1.2 Bridge Design Loadings

Bridge design loadings are loads that a bridge is designed to carry or resist and which determine the size and configuration of its members. Bridge members are designed to withstand the loads acting on them in a safe and economical manner. Loads may be concentrated or distributed depending on the way in which they are applied to the structure.

A concentrated load, or point load, is applied at a single location or over a very small area. Vehicle loads are considered concentrated loads.

A distributed load is applied to all or part of the member, and the amount of load per unit of length is generally constant. The weight of superstructures, bridge decks, wearing surfaces, and bridge parapets produce distributed loads. Secondary loads, such as wind, stream flow, earth cover and ice, are also usually distributed loads.

Highway bridge design loads are established by the American Association of State Highway and Transportation Officials (AASHTO). For many decades, the primary bridge design code in the United States was the AASHTO *Standard Specifications for Highway Bridges* (*Specifications*), as supplemented by agency criteria as applicable.

During the 1990's AASHTO developed and approved a new bridge design code, entitled *AASHTO LRFD Bridge Design Specifications*. It is based upon the principles of Load and Resistance Factor Design (LRFD), as described in Topic P.1.7.

Bridge design loadings can be divided into three principal categories:

- Dead loads
- Primary live loads
- Secondary loads

Dead Loads

Dead loads do not change as a function of time and are considered full-time, permanent loads acting on the structure. They consist of the weight of the materials used to build the bridge (see Figure P.1.1). Dead load includes both the self-weight of structural members and other permanent external loads. They can be broken down into two groups, initial and superimposed.

Initial dead loads are loads which are applied before the concrete deck is hardened, including the beam itself and the concrete deck. Initial deck loads must be resisted by the non-composite action of the beam alone. Superimposed dead loads are loads which are applied after the concrete deck has hardened (on a composite bridge), including parapets and any anticipated future deck pavement. Superimposed dead loads are resisted by the beam and the concrete deck acting compositely. Non-composite and composite action are described in Topic P.1.10.

Dead load includes both the self-weight of the structural members and other permanent external loads.

Example of self-weight: A 6.1 m (20-foot) long beam weighs 0.73 kN per m (50 pounds per linear foot). The total weight of the beam is 4.45 kN (1000 pounds). This weight is called the self-weight of the beam.

Example of an external dead load: If a utility such as a water line is permanently attached to the beam in the previous example, then the weight of the water line is an external dead load. The weight of the water line plus the self weight of the beam comprises the total dead load.

Total dead load on a structure may change during the life of the bridge due to additions such as deck overlays, parapets, utility lines, and inspection catwalks.

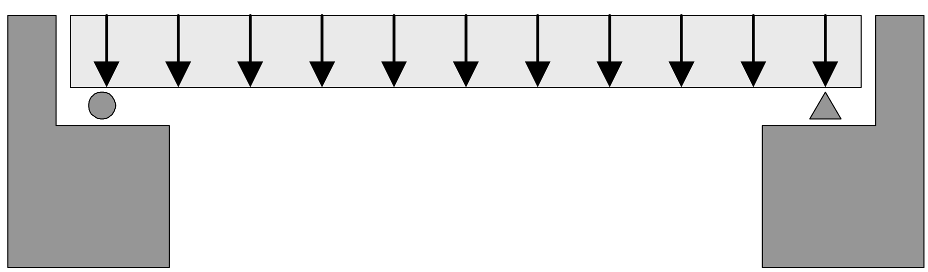


Figure P.1.1 Dead Load on a Bridge

Primary Live Loads

Live loads are considered part-time or temporary loads, mostly of short-term duration, acting on the structure. In bridge applications, the primary live loads are moving vehicular loads (see Figure P.1.2).

To account for the affects of speed, vibration, and momentum, highway live loads are typically increased for impact. Impact is expressed as a fraction of the live load, and its value is a function of the span length.

Standard vehicle live loads have been established by AASHTO for use in bridge design and rating. It is important to note that these standard vehicles do not represent actual vehicles. Rather, they were developed to allow a relatively simple method of analysis based on an approximation of the actual live load.

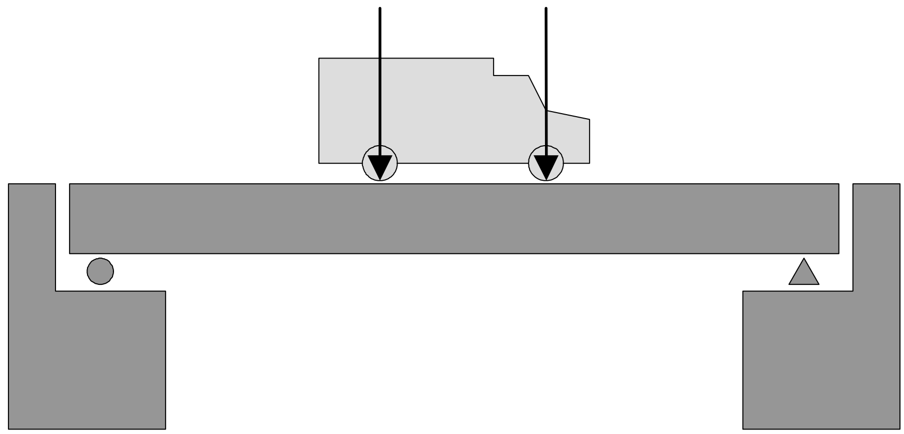


Figure P.1.2 Vehicle Live Load on a Bridge

AASHTO Truck Loadings

There are two basic types of standard truck loadings described in the current AASHTO *Specifications*. The first type is a single unit vehicle with two axles spaced at 14 feet (4.3 m) and designated as a highway truck or "H" truck (see Figure P.1.3). The weight of the front axle is 20% of the gross vehicle weight, while the weight of the rear axle is 80% of the gross vehicle weight. The "H" designation is followed by the gross tonnage of the particular design vehicle.

Example of an H truck loading: H20-35 indicates a 20 ton vehicle with a front axle weighing 4 tons, a rear axle weighing 16 tons, and the two axles spaced 14 feet apart. This standard truck loading was first published in 1935.

The second type of standard truck loading is a two unit, three axle vehicle comprised of a highway tractor with a semi-trailer. It is designated as a highway semi-trailer truck or "HS" truck (see Figure P.1.4).

The tractor weight and wheel spacing is identical to the H truck loading. The semi-trailer axle weight is equal to the weight of the rear tractor axle, and its spacing from the rear tractor axle can vary from 4.3 to 9.1 m (14 to 30 feet). The "HS" designation is followed by a number indicating the gross weight in tons of the tractor only.

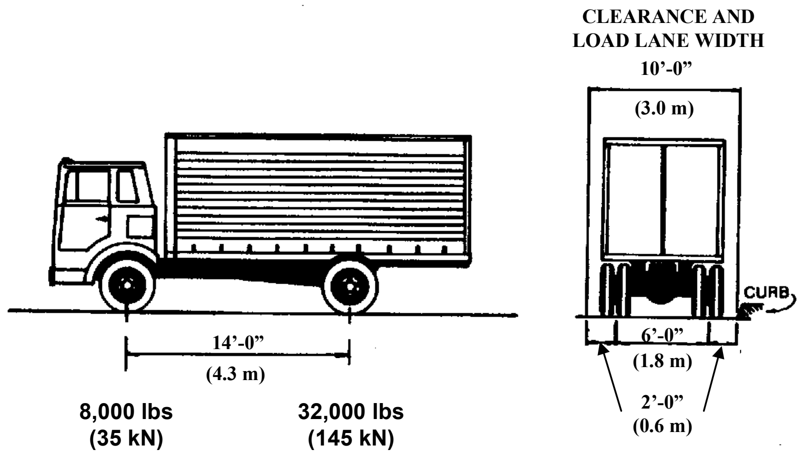


Figure P.1.3 AASHTO H20 Truck

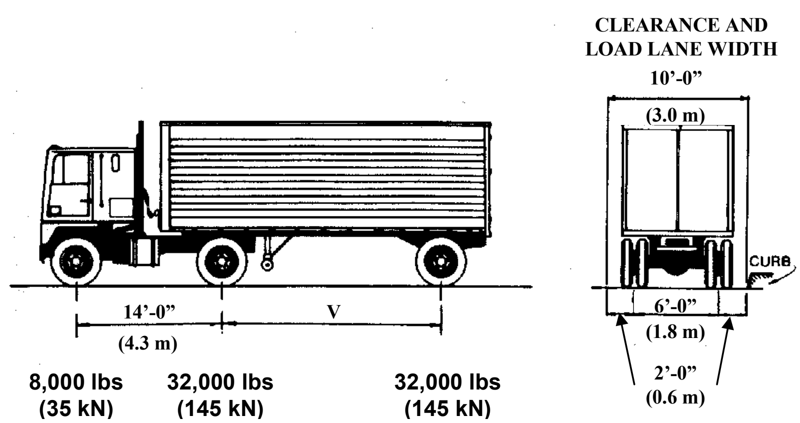


Figure P.1.4 AASHTO HS20 Truck

Example of an HS truck loading: HS20-44 indicates a vehicle with a front tractor axle weighing 4 tons, a rear tractor axle weighing 16 tons, and a semi-trailer axle weighing 16 tons. The tractor portion alone weighs 20 tons, but the gross vehicle weight is 36 tons. This standard truck loading was first published in 1944.

In specifications prior to 1944, a standard loading of H15 was used. In 1944, the policy of affixing the publication year of design loadings was adopted. In specifications prior to 1965, the HS20-44 loading was designated as H20-S16-44, with the S16 identifying the gross axle weight of the semi-trailer in tons.

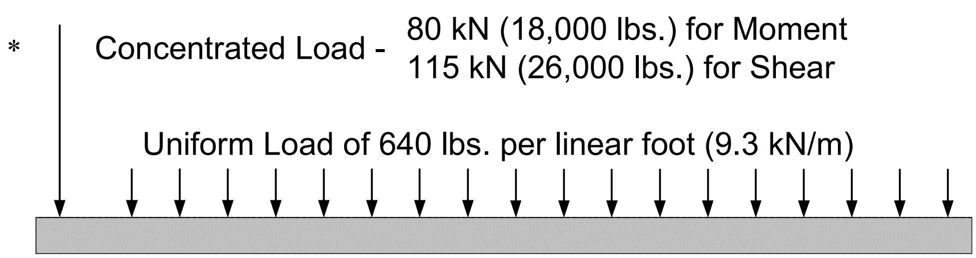
The H and HS vehicles do not represent actual vehicles, but can be considered as "umbrella" loads. The wheel spacings, weight distributions, and clearance of the Standard Design Vehicles were developed to give a simpler method of analysis, based on a good approximation of actual live loads.

The H and HS vehicle loads are the most common loadings for design, analysis, and rating, but other loading types are used in special cases.

AASHTO Lane Loadings

In addition to the standard truck loadings, a system of equivalent lane loadings was developed in order to provide a simple method of calculating bridge response to a series, or "train", of trucks. Lane loading consists of a uniform load per linear foot of traffic lane combined with a concentrated load located on the span to produce the most critical situation (see Figure P.1.5).

For design and load capacity rating analysis, an investigation of both a truck loading and a lane loading must be made to determine which produces the greatest stress for each particular member. Lane loading will generally govern over truck loading for longer spans. Both the H and HS loadings have corresponding lane loads.



H20-44 Loading
HS20-44 Loading

* Use two concentrated loads for negative moment in continuous spans (Refer to AASHTO Page 23)

Figure P.1.5 AASHTO Lane Loadings.

Alternate Military Loading

The Alternate Military Loading is a single unit vehicle with two axles spaced at 1.2 m (4 feet) and weighing 110 kN (12 tons) each. It has been part of the AASHTO *Specifications* since 1977. Bridges on interstate highways or other highways which are potential defense routes are designed for either an HS20 loading or an Alternate Military Loading (see Figure P.1.6).

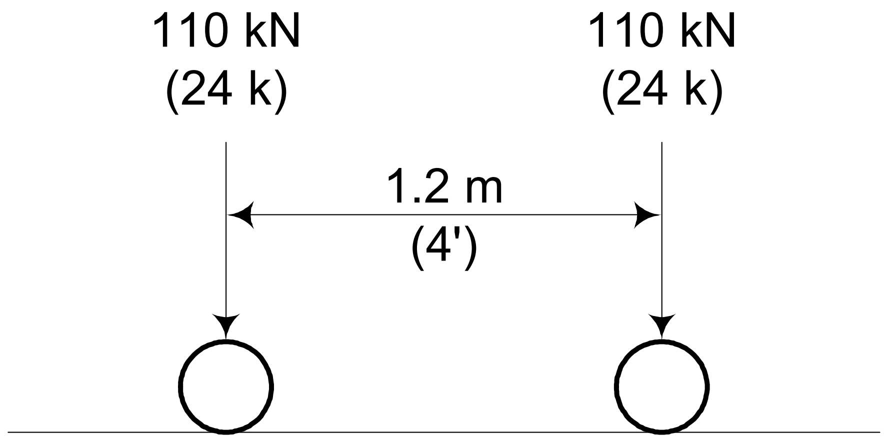


Figure P.1.6 Alternate Military Loading

LRFD Live Loads

The AASHTO LRFD design vehicular live load, designated HL-93, is a modified version of the HS-20 highway loadings from the AASHTO Standard Specifications. Under HS-20 loading as described earlier, the truck **or** lane load is applied to each loaded lane. Under HL-93 loading, the design truck or tandem, in combination with the lane load, is applied to each loaded lane.

The LRFD design truck is exactly the same as the AASHTO HS-20 design truck. The LRFD design tandem, on the other hand, consists of a pair of 110 kN axials spread at 1.2 m (25 kip axles spaced 4 feet) apart. The transverse wheel spacing of all of the trucks is 6 feet.

The magnitude of the HL-93 lane load is equal to that of the HS-20 lane load. The lane load is 9 kN per meter (0.64 kips per linear foot) longitudinally and it is distributed uniformly over a 3 m (10 foot) width in the transverse direction. The difference between the HL-93 lane load and the HS-20 lane load is that the HL-93 lane load does not include a point load.

Finally, for LRFD live loading, the dynamic load allowance, or impact, is applied to the design truck or tandem but is not applied to the design lane load. It is typically 33 percent of the design vehicle.

Permit Vehicles

Permit vehicles are overweight vehicles which, in order to travel a state's highways, must apply for a permit from that state. They are usually heavy trucks (e.g., combination trucks, construction vehicles, or cranes) that have varying axle spacings depending upon the design of the individual truck. To ensure that these vehicles can safely operate on existing highways and bridges, most states require that bridges be designed for a permit vehicle or that the bridge be checked to determine if it can carry a specific type of vehicle. For safe and legal operation, agencies issue permits upon request that identify the required gross weight, number of axles, axle spacing, and maximum axle weights for a designated route (see Figure P.1.7).

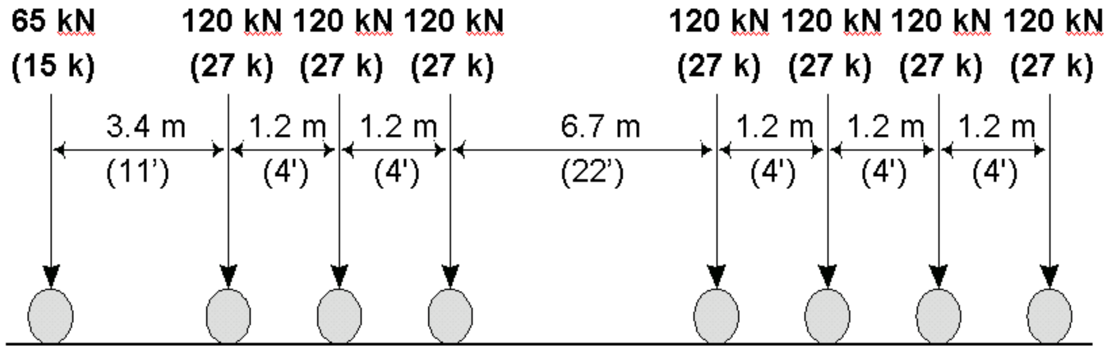


Figure P.1.7 910 kN (204 kip) Permit Vehicle (for Pennsylvania)

Secondary Loads

In addition to dead loads and primary live loads, bridge components are designed to resist secondary loads, which include the following:

- **Earth pressure** - a horizontal force acting on earth-retaining substructure units, such as abutments and retaining walls
- **Buoyancy** - the force created due to the tendency of an object to rise when submerged in water
- **Wind load on structure** - wind pressure on the exposed area of a bridge
- **Wind load on live load** - wind effects transferred through the live load vehicles crossing the bridge
- **Longitudinal force** - a force in the direction of the bridge caused by braking and accelerating of live load vehicles
- **Centrifugal force -** an outward force that a live load vehicle exerts on a curved bridge
- **Rib shortening -** a force in arches and frames created by a change in the geometrical configuration due to dead load
- **Shrinkage -** applied primarily to concrete structures, this is a multi-directional force due to dimensional changes resulting from the curing process
- **Temperature -** since materials expand as temperature increases and contract as temperature decreases, the force caused by these dimensional changes must be considered
- **Earthquake -** bridge structures must be built so that motion during an earthquake will not cause a collapse
- **Stream flow pressure -** a horizontal force acting on bridge components constructed in flowing water
- **Ice pressure** - a horizontal force created by static or floating ice jammed against bridge components
- **Impact loading -** the dynamic effect of suddenly receiving a live load; this additional force can be up to 30% of the applied primary live load force
- **Sidewalk loading -** sidewalk floors and their immediate supports are designed for a pedestrian live load not exceeding 4.1 kN per square meter (85 pounds per square foot)
- **Curb loading -** curbs are designed to resist a lateral force of not less than 7.3 kN per linear meter (500 pounds per linear foot)
- **Railing loading -** railings are provided along the edges of structures for protection of traffic and pedestrians; the maximum transverse load applied to any one element need not exceed 44.5 kN (10 kips)

A bridge may be subjected to several of these loads simultaneously. The AASHTO *Specifications* have established a table of loading groups. For each group, a set of loads is considered with a coefficient to be applied for each particular load. The coefficients used were developed based on the probability of various loads acting simultaneously.

P.1.3 Material Response to Loadings

Each member of a bridge has a unique purpose and function, which directly affects the selection of material, shape, and size for that member. Certain terms are used to describe the response of a bridge material to loads. A working knowledge of these terms is essential for the bridge inspector.

Force

A force is the action that one body exerts on another body. Force has two components: magnitude and direction (see Figure P.1.8). The basic English unit of force is called pound (abbreviated as lb.). The basic metric unit of force is called Newton (N). A common unit of force used among engineers is a kip (K), which is 1000 pounds. In the metric system, the kilonewton (kN), which is 1000 Newtons, is used. Note: 1 kip = 4.4 kilonewton.

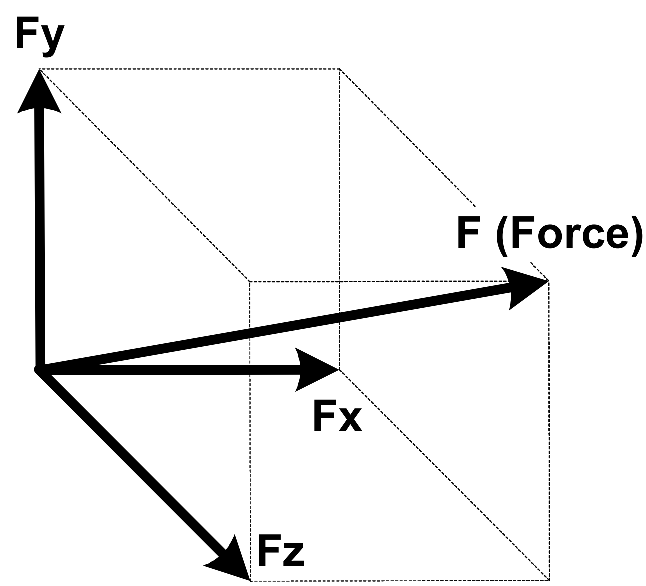


Figure P.1.8 Basic Force Components

Stress

Stress is a basic unit of measure used to denote the intensity of an internal force. When a force is applied to a material, an internal stress is developed. Stress is defined as a force per unit of cross-sectional area.

$$\text{Stress (S)} = \frac{\text{Force (F)}}{\text{Area (A)}}$$

The basic English unit of stress is pounds per square inch (abbreviated as psi). However, stress can also be expressed in kips per square inch (ksi) or in any other units of force per unit area. The basic metric unit of stress is Newton per square meter, or Pascal (Pa). An allowable unit stress is generally established for a given material. Note: 1 ksi = 6.9 Pa.

Example of a stress: If a 30,000 lb. force acts uniformly over an area of 10 square inches, then the stress caused by this force is 3000 psi (or 3 ksi).

Similarly, if a 40,000 Newton force acts uniformly over an area of 20 square meters, then the stress caused by this force is 2000 Pa.

Deformation

Deformation is the local distortion or change in shape of a material due to stress.

Strain

Strain is a basic unit of measure used to describe an amount of deformation. It denotes the ratio of a material's deformed dimension to a material's original dimensions. For example, strain in a longitudinal direction is computed by dividing the change in length by the original length.

$$\text{Strain}\ (\varepsilon) = \frac{\text{Change in Length}\ (\Delta L)}{\text{Original Length}\ (L)}$$

Strain is a dimensionless quantity. However, it can also be expressed as a percentage or in units of length per length (e.g., inch/inch).

Example of strain: If a weight acting on a 20 foot long column causes an axial deformation of 0.002 feet, then the resulting axial strain is 0.002 feet divided by 20 feet, or 0.0001 foot/foot. This strain can also be expressed simply as 0.0001 (with no units) or as 0.0l%.

Similarly, if a weight acting on a 50 m long column causes an axial deformation of 0.05 m, then the resulting axial strain is 0.001 m/m. This strain can also be compressed simply as 0.001 (with no units) or as 0.1%.

Elastic Deformation

Elastic deformation is the reversible distortion of a material. A member is elastically deformed if it returns to its original shape upon removal of a force. Elastic strain is sometimes termed reversible strain because it disappears after the stress is removed. Bridges are designed to deform elastically and return to their original shape after the live loads are removed.

Example of elastic deformation: A stretched rubber band will return to its original shape after being released from a taut position. Generally, if the strain is elastic, there is a direct proportion between the amount of strain and the applied stress.

Plastic Deformation

Plastic deformation is the irreversible or permanent distortion of a material. A material is plastically deformed if it retains a deformed shape upon removal of a force. Plastic strain is sometimes termed irreversible or permanent strain because it remains after the stress is removed. Plastic strain is not directly proportional to the given applied stress as is the case with the elastic strain.

Example of plastic deformation: If a car crashed into a brick wall, the fenders and bumpers would deform. This deformation would remain even after the car backed away from the wall. Therefore, the fenders and bumpers have undergone plastic deformation.

Creep

Creep is a form of plastic deformation that occurs gradually at stress levels normally associated with elastic deformation. Creep is defined as the gradual, continuing irreversible change in the dimensions of a member due to the sustained application of load. It is caused by the molecular readjustments in a material under constant load. The creep rate is the change in strain (plastic deformation) over a certain period of time.

Example of creep: If a lump of putty is left untouched on a table for several days, it will gradually settle and change in shape. This deformation is due to the sustained application of its own weight and illustrates the effects of creep.

Thermal Effects

In bridges, thermal effects are most commonly experienced in the longitudinal expansion and contraction of the superstructure. It is possible to disregard deformations caused by thermal effects when members are free to expand and contract. However, there may be members for which expansion and contraction is inhibited or prevented in certain directions. Thermal changes in these members can cause significant frictional stresses and must be considered by the inspector.

Materials expand as temperature increases and contract as temperature decreases. The amount of thermal deformation in a member depends on:

- A coefficient of thermal expansion, unique for each material
- The temperature change
- The member length

Example of thermal effects: Most thermometers operate on the principle that the material within the glass bulb expands as the temperature increases and contracts as the temperature decreases.

Stress-Strain Relationship

For most structural materials, values of stress and strain are directly proportional (see Figure P.1.9). However, this proportionality exists only up to a particular value of stress called the elastic limit. Two other frequently used terms, which closely correspond with the elastic limit, are the proportional limit and the yield point.

When applying stress up to the elastic limit, a material deforms elastically. Beyond the elastic limit, deformation is plastic and strain is not directly proportional to a given applied stress. The material property, which defines its stress-strain relationship, is called the modulus of elasticity, or Young's modulus.

Modulus of Elasticity

Each material has a unique modulus of elasticity, which defines the ratio of a given stress to its corresponding strain. It is the slope of the elastic portion of the stress-strain curve.

$$\text{Modulus of Elasticity (E)} = \frac{\text{Stress (S)}}{\text{Strain }(\varepsilon)}$$

The modulus of elasticity applies only as long as the elastic limit of the material has not been reached. The units for modulus of elasticity are the same as those for stress (i.e., psi or ksi for English, and Pa or kPa for metric).

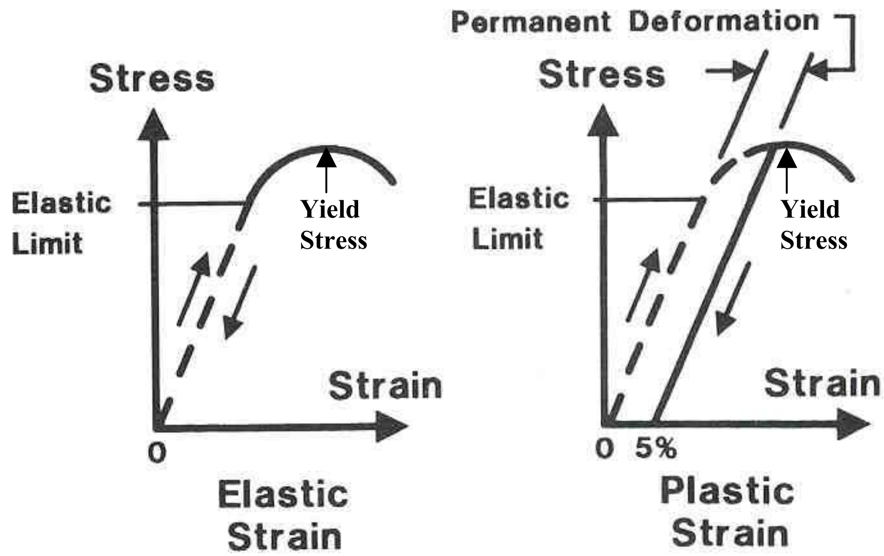


Figure P.1.9 Stress-Strain Diagram

Example of modulus of elasticity: If a stress of 20 Mpa (2900 psi) is below the elastic limit and causes a strain of 0.0001, then the modulus of elasticity can be computed based on these values of stress and strain.

$$E = \frac{2{,}900\,\text{psi}}{0.0001} = 29{,}000{,}000\,\text{psi} = 29{,}000\,\text{ksi}\,(200{,}000\,\text{MPa})$$

This is approximately equal to the modulus of elasticity for steel. The modulus of elasticity for concrete is approximately 3000 to 4500 ksi, and for commonly used grades of timber it is approximately 11,000 Mpa (1600 ksi).

Ductility and Brittleness

Ductility is the measure of plastic (permanent) strain that a material can endure. A ductile material will undergo a large amount of plastic deformation before breaking. It will also have a greatly reduced cross-sectional area before breaking.

Example of ductility: A baker working with pizza dough will find that the dough can be stretched a great deal before it will break into two sections. Therefore, pizza dough is a ductile material. When the dough finally does break, it will have a greatly reduced cross-sectional area.

Structural materials for bridges that are generally ductile include:

- Steel
- Aluminum
- Copper
- Wood

Brittle, or non-ductile, materials will not undergo significant plastic deformation before breaking. Failure of a brittle material occurs suddenly, with little or no warning.

Example of brittleness: A glass table may be able to support several magazines and books. However, if more and more weight is piled onto the table, the glass will eventually break with little or no warning. Therefore, glass is a brittle material.

Structural materials for bridges that are generally brittle include:

- Concrete
- Cast iron
- Stone
- Fiber Reinforced Polymer

Fatigue

Fatigue is a material response that describes the tendency of a material to break when subjected to repeated loading. Fatigue failure occurs within the elastic range of a material after a certain number and magnitude of stress cycles have been applied.

Each material has a hypothetical maximum stress value to which it can be loaded and unloaded an infinite number of times. This stress value is referred to as the fatigue limit and is usually lower than the breaking strength for infrequently applied loads.

Ductile materials such as steel and aluminum have high fatigue limits, while brittle materials such as concrete have low fatigue limits. Wood has a high fatigue limit even though it is more like a brittle material than a ductile one.

Example of fatigue: If a steel paper clip is bent and then allowed to return to its original position (elastic deformation), it is unlikely that the paper clip will break into two pieces. However, if this action is repeated many times, the paper clip will eventually break. The paper clip failure is analogous to a fatigue failure.

For a description of fatigue categories for various steel details, refer to Topic 8.1.

Isotropy

A material that has the same mechanical properties regardless of which direction it is loaded is said to be isotropic.

Example of isotropy: Plain, unreinforced concrete, and steel.

For a description of isotropic materials, refer to Topics 2.2 and 2.3.

P.1.4 Mechanics of Materials

Materials respond to loadings in a manner dependent on their mechanical properties. In characterizing materials, certain mechanical properties must be defined.

Yield Strength

The ability of a material to resist plastic (permanent) deformation is called the yield strength. Yield strength corresponds to stress level defined by a material's yield point.

Tensile Strength

The tensile strength of a material is the stress level defined by the maximum tensile load that it can resist without failure. Tensile strength corresponds to the highest ordinate on the stress-strain curve and is sometimes referred to as the ultimate strength.

Toughness

Toughness is a measure of the energy required to break a material. It is related to ductility. Toughness is not necessarily related to strength. A material might have high strength but little toughness. A ductile material with the same strength as a non-ductile material will require more energy to break and thus exhibit more toughness. For highway bridges, the CVN (Charpy V-notch) toughness is the toughness value usually used. It is an indicator of the ability of the steel to resist crack propagation in the presence of a notch or flaw. The unit for toughness N-m @ degrees C (ft-lbs @ degrees F).

P.1.5 Bridge Response to Loadings

Each member of a bridge is intended to respond to loads in a particular way. The bridge inspector must understand the manner in which loads are applied to each member in order to evaluate if it functions as intended. Once the inspector understands a bridge member's response to loadings, he will be able to determine if a member defect has an adverse effect on the load-carrying capacity of that member.

Bridge members respond to various loadings by resisting four basic types of forces. These are:

- Axial forces (compression and tension)
- Bending forces (flexure)
- Shear forces
- Torsional forces

Equilibrium

In calculating these forces, the analysis is governed by equations of equilibrium. Equilibrium equations represent a balanced force system and may be expressed as:

$$\sum V = 0$$
$$\sum H = 0$$
$$\sum M = 0$$

where:
$\sum$ = summation of
V = vertical forces
H = horizontal forces
M = moments (bending forces)

Axial Forces

An axial force is a push or pull type of force which acts in the long direction of a member. An axial force causes compression if it is pushing and tension if it is pulling (see Figure P.1.10). Axial forces are generally expressed in English units of pounds or kips, and metric units of Newtons or kilonewtons.

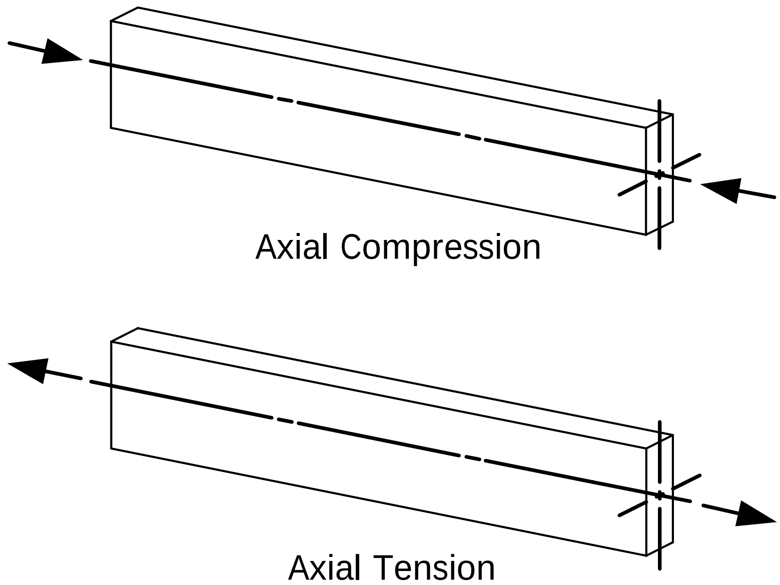


Figure P.1.10 Axial Forces

Example of an axial force: A man sitting on top of a fence post is exerting an axial force that causes compression in the fence post. A group of people playing tug-of-war exerts an axial force that causes tension in the rope.

Truss members are common bridge elements which carry axial loads. They are designed for both compression and tension forces. Cables are designed for axial forces in tension. Columns are vertical bridge elements designed for compressive axial forces.

True axial forces act uniformly over a cross-sectional area. Therefore, axial stress can be calculated by dividing the force by the area on which it acts.

$$f_a = \frac{P}{A}$$

where: f_a = axial stress
P = axial force
A = cross-sectional area

When bridge members are designed to resist axial forces, the cross-sectional area will vary depending on the magnitude of the force, whether the force is tensile or compressive, and the type of material used.

For tension and compression members, the cross-sectional area must satisfy the previous equation for an acceptable axial stress. However, the acceptable axial compressive stress is generally lower than that for tension because of a phenomenon called buckling.

Bending Forces

Bending forces in bridge members are caused by moment. A moment is commonly developed by a transverse loading which causes a member to bend. The greatest bending moment that a beam can resist is generally the governing factor which determines the size and material of the member. Bending moments produce both compression and tension forces at different locations in the member and can be positive or negative (see Figure P.1.11). Moments are generally expressed in English units of pound-feet or kip-feet, and metric units of Newton-meters or kilonewton-meters.

Example of bending moment: When a rectangular rubber eraser is bent, a moment is produced in the eraser. If the ends are bent upwards, the top half of the eraser can be seen to shorten, while the bottom half can be seen to lengthen. Therefore, the moment produces compression forces in the top layers of the eraser and tension forces in the bottom layers.

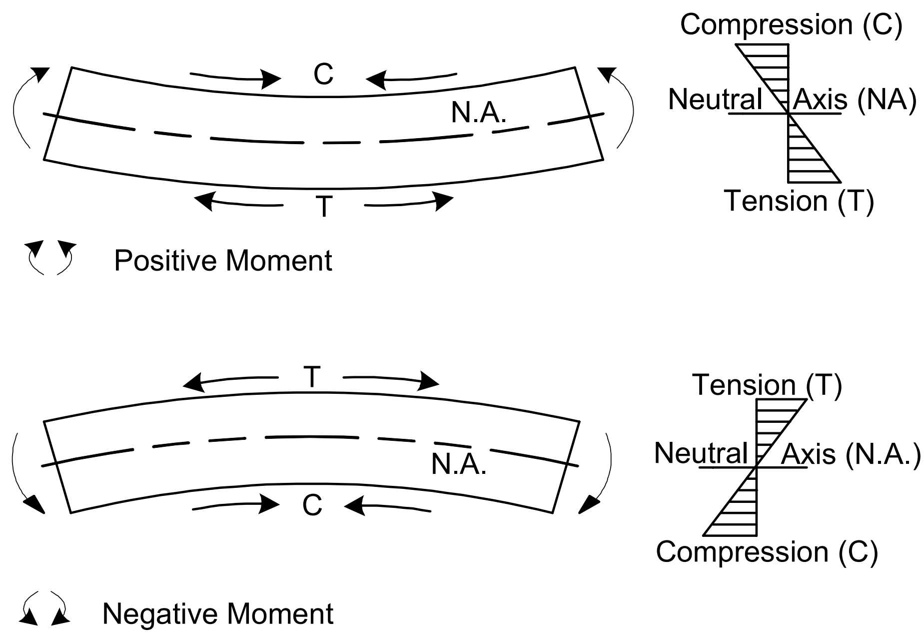


Figure P.1.11 Positive and Negative Moment

Beams and girders are the most common bridge elements used to resist bending moments. The flanges are most critical because they provide the greatest resistance to the compressive and tensile forces developed by the moment (see Figure P.1.12).

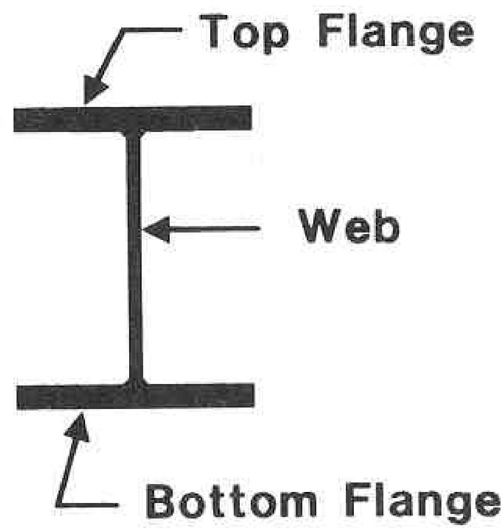


Figure P.1.12 Girder Cross Section

Bending members have a neutral axis at which there are no bending stresses. On a cross section of a member, bending stresses vary linearly with respect to the distance from the neutral axis (see Figures P.1.11 and P.1.13).

The formula for maximum bending stress is (see Figure P.1.13):

$$f_b = \frac{Mc}{I}$$

where:
- f_b = bending stress on extreme fiber (or surface) of beam
- M = applied moment
- c = distance from neutral axis to extreme fiber (or surface) of beam
- I = moment of inertia (a property of the beam cross-sectional area and shape)

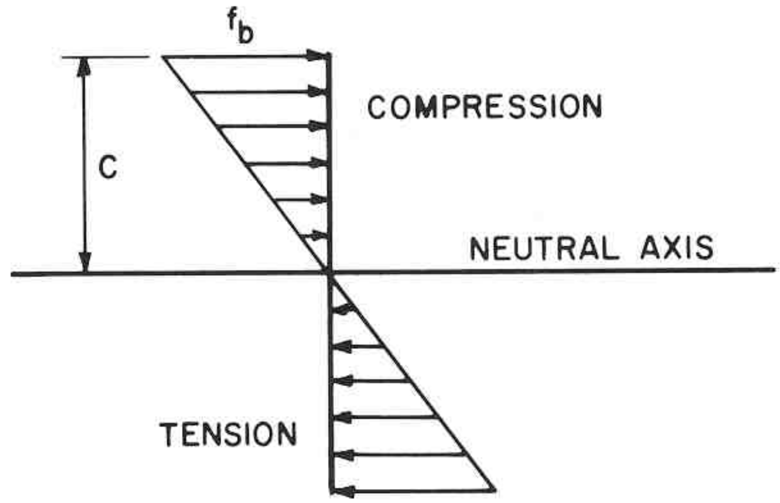


Figure P.1.13 Bending Stresses

Shear Forces

Shear is a force, which results from equal but opposite transverse forces, which tend to slide one section of a member past an adjacent section (see Figure P.1.14). Shear forces are generally expressed in English units of pounds or kips, and metric units of Newtons or kilonewtons.

Example of shear: When scissors are used to cut a piece of paper, a shear force has caused one side of the paper to separate from the other. Scissors are often referred to as shears since they exert a shear force.

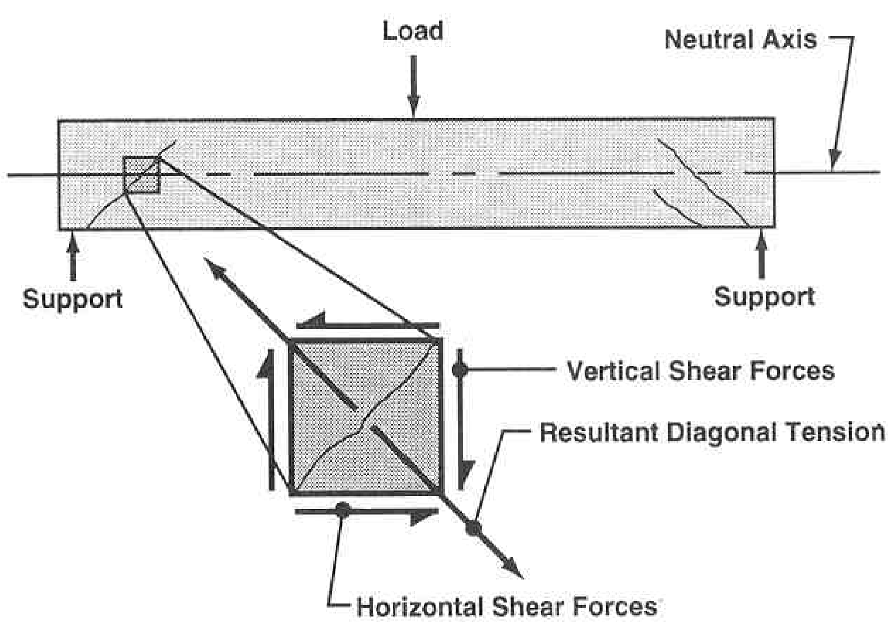


Figure P.1.14 Shear Forces in a Member Element

Beams and girders are common shear resisting members. In an I- or T-beam, most of the shear is resisted by the web (see Figure P.1.12). The shear stress produced by the transverse forces is manifested in a horizontal shear stress which is accompanied by a vertical shear stress of equal magnitude. Vertical shear strength is generally considered in most design criteria. The formula for vertical shear stress in I- or T-beams is:

$$f_v = \frac{V}{A_W}$$

where:	f_v	=	shear stress
	V	=	vertical shear due to external loads
	A_w	=	area of web

In a solid rectangular beam, shear is resisted by the entire cross section, and the formula for vertical shear stress is:

$$f_v = \frac{3V}{2A}$$

where: A = cross-sectional area

Torsional Forces

Torsion is a force resulting from externally applied moments which tend to rotate or twist a member about its longitudinal axis. Torsional force is commonly referred to as torque and is generally expressed in English units of pound-feet or kip-feet, and metric units of Newton-meters or Kilonewtons-meters.

Example of torsion: One end of a long rectangular steel bar is clamped horizontally in a vise so that the long side is up and down. Using a large wrench, a moment is applied to the other end, which causes it to rotate so that the long side is now left to right. The steel bar is resisting a torsional force or torque which has twisted it 90˚ with respect to its original orientation (see Figure P.1.15).

Torsional forces develop in bridge members, which are interconnected and experience unbalanced loadings. Bridge elements are generally not designed as torsional members. However, in some bridge superstructures where elements are framed together, torsional forces can occur in longitudinal members. When these members experience differential deflection, adjoining transverse members apply twisting moments resulting in torsion. In addition, curved bridges are generally subject to torsion (see Figure P.1.16).

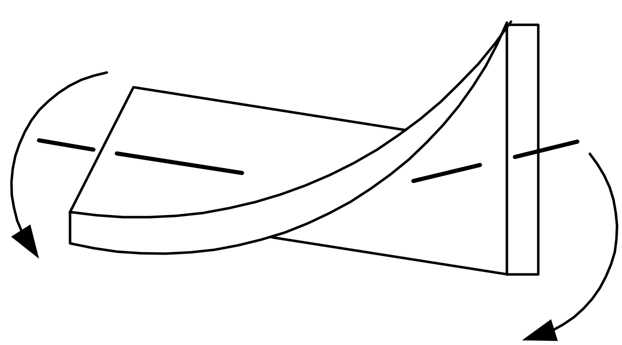


Figure P.1.15 Torsion

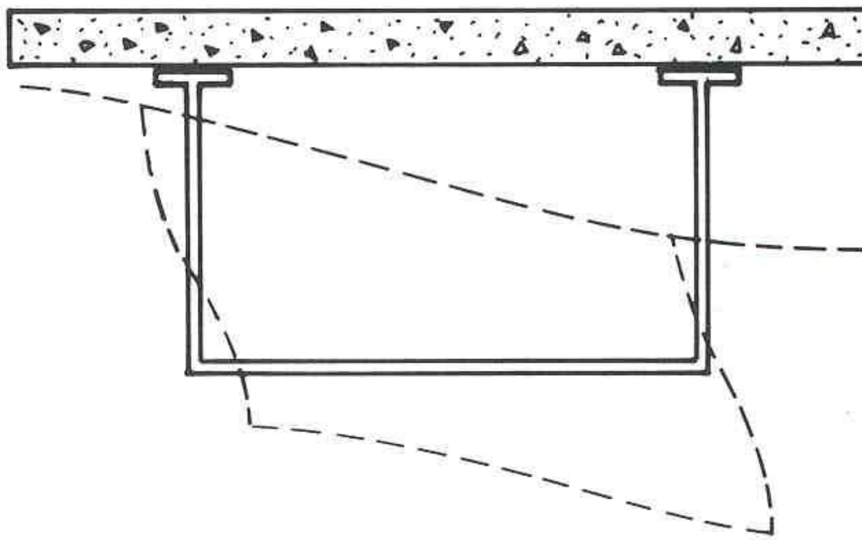


Figure P.1.16 Torsional Distortion

Reactions

A reaction is a force provided by a support that is equal but opposite to the force transmitted from a member to its support (see Figure P.1.17). Reactions are most commonly vertical forces, but a reaction can also be a horizontal force. The reaction at a support is the measure of force that it must transmit to the ground. A vertical reaction increases as the loads on the member are increased or as the loads are moved closer to that particular support. Reactions are generally expressed in English units of pounds or kips, and metric units of Newtons or kilonewtons.

Example of reactions: Consider a bookshelf consisting of a piece of wood supported at its two ends by bricks. The bricks serve as supports, and the reaction is based on the weight of the shelf and the weight of the books on the shelf. As more books are added, the reaction provided by the bricks will increase. As the books are shifted to one side, the reaction provided by the bricks at that side will increase, while the reaction at the other side will decrease.

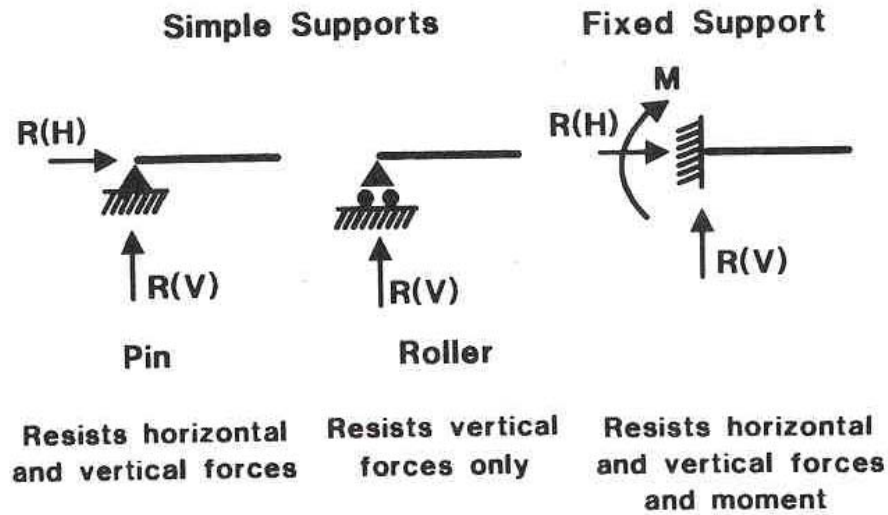


Figure P.1.17 Types of Supports

The loads of the entire bridge always equal the reactions provided by the abutments and the piers. However, on a smaller scale, each individual beam and girder also exerts forces, which create reactions provided by its supporting members.

Overloads

Overload damage or serious cracking may occur when members are overstressed. Overload occurs when the stresses applied are greater than the elastic limit for the material.

Buckling

Buckling is the tendency of a member to deform or bend out of plane when subjected to a compressive force. As the length and slenderness of a compression member increases, the likelihood of buckling also increases.

Compression members require additional cross-sectional area or bracing to resist buckling.

Example of buckling: A paper or plastic straw compressed axially at both ends with an increasing force will eventually buckle.

Elongation

Elongation is the tendency of a member to extend or stretch when subjected to a tensile force. Elongation can be either elastic or plastic.

Example of elongation: A piece of taffy pulled will stretch in a plastic manner.

P.1.6 Bridge Movements

Bridges move because of many factors; some are anticipated, but others are not. Unanticipated movements generally result from settlement, sliding, and rotation of foundations. Anticipated movements include live load deflections, thermal expansions and contractions, shrinkage and creep, earthquakes, rotations, wind drifting, and vibrations. Of these movements, the three major anticipated movements are live load deflections, thermal movements. and rotational movements.

Live Load Deflections

Deflection produced by live loading should not be excessive because of aesthetics, user discomfort, and possible damage to the whole structure.

Limitations are generally expressed as a deflection-to-span ratio. AASHTO generally limits live load bridge deflection for steel and concrete bridges to 1/800 (i.e., 25-mm (1inch) vertical movement per 20.3 m (67 feet) of span length). For bridges that have sidewalks, AASHTO limits live load bridge deflection to 1/1000 (i.e., 25-mm (1-inch) vertical movement per 25 m (83 feet) of span length).

Thermal Movements

The longitudinal expansion and contraction of a bridge is dependent on the range of temperature change, length of bridge, and most importantly, materials used in construction. Thermal movements are frequently accommodated using expansion joints and movable bearings. To accommodate thermal movements, AASHTO recommends the designer allow 32-mm (1-1/4 inches) of movement for each 30.5 m (100 feet) of span length for steel bridges and 30-mm (1-3/16 inches) of movement for each 30.5 m (100 feet) of span length for concrete bridges.

Rotational Movements

Rotational movement in bridges is a direct result of live load deflection and occurs with the greatest magnitude at the bridge supports. This movement can be accommodated using bearing devices that permit rotation.

P.1.7 Design Methods

Bridge engineers use various design methods that incorporate safety factors to account for uncertainties and random deviations in material strength, fabrication, construction, durability, and loadings.

Allowable Stress Design

The Allowable Stress Design (ASD) or Working Stress Design (WSD) is a method in which the maximum stress a particular member may carry is limited to an allowable or working stress. The allowable or working stress is determined by applying an appropriate factor of safety to the limiting stress of the material. For example, the allowable tensile stress for a steel tension member is 0.55 times the steel yield stress. This results in a safety factor of 1.8. The capacity of the member is based on either the inventory rating level or the operating rating level. AASHTO currently has ten possible WSD group loadings.

Load Factor Design

Load Factor Design (LFD) is a method in which the ultimate strength of a material is limited to the combined effect of the factored loads. The factored loads are determined from the applied loadings, which are increased by selected multipliers that provide a factor of safety. The load factors for AASHTO Group I are 1.3(DL+1.67(LL+I)). AASHTO currently has ten possible LFD group loadings.

Load and Resistance Factor Design

Load and Resistance Factor Design (LRFD) is a design procedure based on the actual strength, rather than on an arbitrary calculated stress. It is an ultimate strength concept where both working loads and resistance are multiplied by factors, and the design performed by assuming the strength exceeds the load. (The load multipliers used in LRFD are not the same multipliers that are used in LFD.)

These design methods are conservative due to safety factors and limit the stress in bridge members to a level well within the material's elastic range, provided that the structural members are in good condition. That is why it is important for inspectors to accurately report any deficiency found in the members.

P.1.8 Bridge Ratings

One of the primary functions of a bridge inspection is to collect information necessary for a bridge load capacity rating. Therefore, the bridge inspector should understand the principles of bridge load ratings. Bridge load rating methods and guidelines are provided by AASHTO in the *Manual for Condition Evaluation of Bridges*.

A bridge load rating is used to determine the usable live load capacity of a bridge. Each member of a bridge has a unique load rating, and the bridge load rating represents the most critical one. Bridge load rating is generally expressed in units of tons, and it is computed based on the following basic formula:

$$\text{Bridge Rating Factor (RF)} = \frac{C - A_1 D}{A_2 L(1 + I)}$$

where:

- RF= the rating factor for the live-load carrying capacity; the rating factor multiplied by the rating vehicle in tons gives the rating of the structure
- C = the capacity of the member
- D = the dead load effect on the member
- L = the live load effect on the member
- I = the impact factor to be used with the live load effect
- A_1 = factor for dead loads
- A_2 = factor for live loads

Inventory Rating

The inventory rating level generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load, which can safely utilize an existing structure for an indefinite period of time. For the allowable stress method, the inventory rating for steel used to be based on 55% of the yield stress. Inventory ratings have been refined to reflect the various material and load types. See the *Manual for Condition Evaluation of Bridges* (Section 6.6.2 for Allowable Stress Inventory Ratings and Section 6.6.3 for Load Factor Inventory Ratings).

Operating Rating

Load ratings based on the operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge. For steel, the allowable stress for operating rating used to be 75% of the yield stress. Operating ratings have been refined to reflect the various material and load types. See the *Manual for Condition Evaluation of Bridges* (Section 6.6.2 for Allowable Stress Operating Ratings and Section 6.6.3 for Load Factor Operating Ratings).

Special permits for heavier than normal vehicles may occasionally be issued by a governing agency. The load produced by the permit vehicle must not exceed the structural capacity determined by the operating rating.

Rating Vehicles

Rating vehicles are truck loads applied to the bridge to establish the inventory and operating ratings. These rating vehicles (see Figure P.1.18) include:

- H loading
- HS loading
- Alternate Interstate Loading (Military Loading)
- Type 3 unit
- Type 3-S2 unit
- Type 3-3 unit
- The maximum legal load vehicles of the state

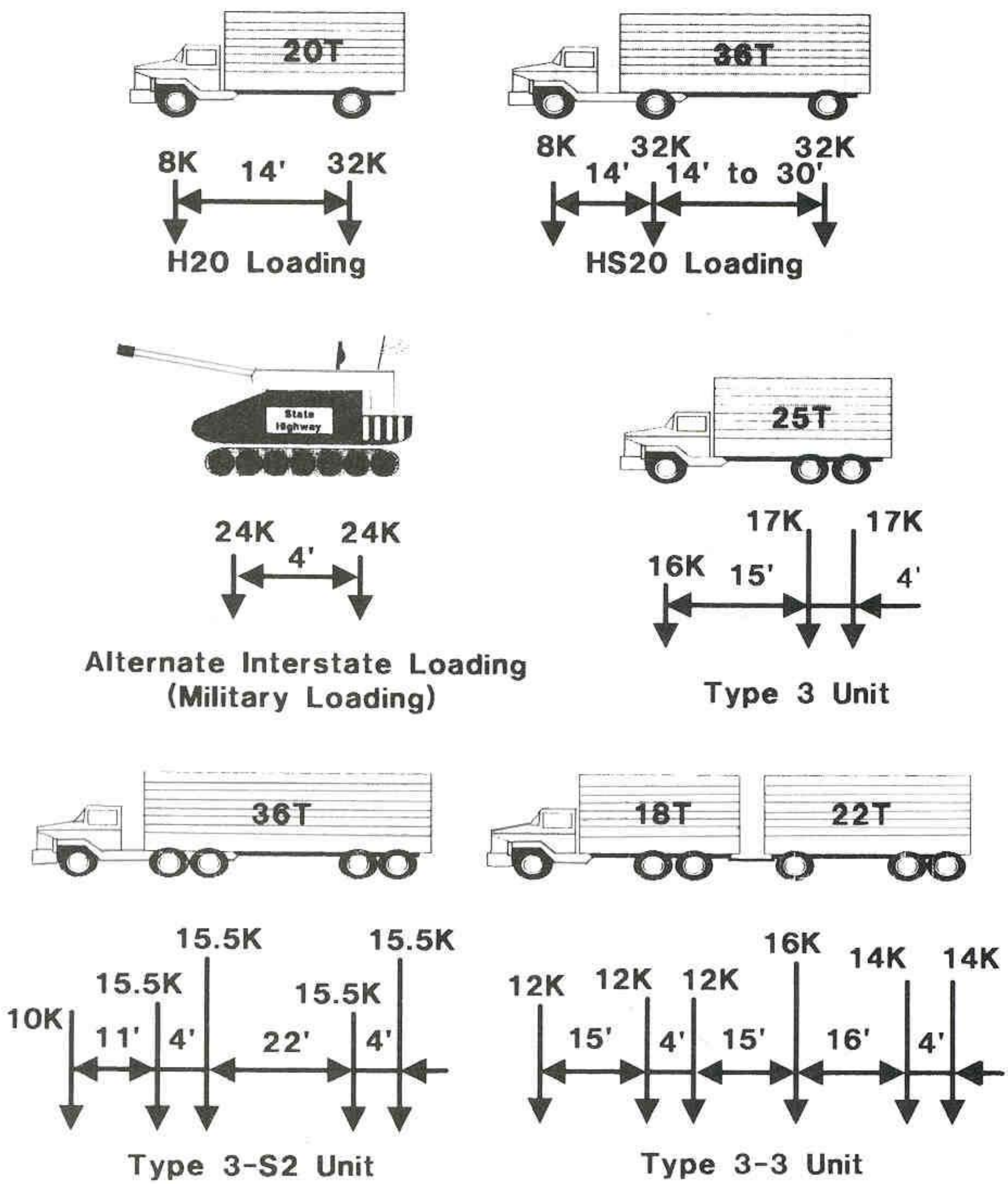


Figure P.1.18 Rating Vehicles
The axle spacing and weights of the Type 3 unit, Type 3-S2 unit, and Type 3-3 unit are based on actual vehicles. However, as mentioned previously, the H and HS loadings do not represent actual vehicles.

These standard rating vehicles were chosen based on load regulations of most states and governing agencies. However, individual states and agencies may also establish their own unique rating vehicles.

Bridge Posting

Bridge loads are posted to warn the public of the load capacity of a bridge, to avoid safety hazards, and to adhere to federal law. Federal law requires bridges to be inspected every two years for lengths greater than 6.1 m (20 feet), and for bridge postings that can't carry standard truck loads. Federal law requires bridges to be posted when the State's legal loads exceed the operating rating for the bridge. It is the inspector's responsibility to gather and provide information that the structural engineer can use to analyze and rate the bridge.

The federal regulations for safe load-carrying capacity are determined by the FHWA and AASHTO under the following criteria:

- Physical condition
- Potential for fatigue damage
- Type of structure/configuration
- Truck traffic data

Bridge postings show the maximum allowable load by law for single vehicles and combinations while still maintaining an adequate safety margin (see Figure P.1.19).

Figure P.1.19 Bridge Weight Limit Posting

Failure to comply with bridge posting may result in fines, tort suits/financial liabilities, accidents, or even death. In addition, bridges may be damaged when postings are ignored (see Figure P.1.20).

Figure P.1.20 Damaged Bridge due to Failure to Comply with Bridge Posting

P.1.9 Span Classifications

Beams and bridges are classified into three span classifications that are based on the nature of the supports and the interrelationship between spans. These classifications are:

- Simple
- Continuous
- Cantilever

Simple

A simple span is a span with only two supports, each of which is at or near the end of the span (see Figure P.1.21).

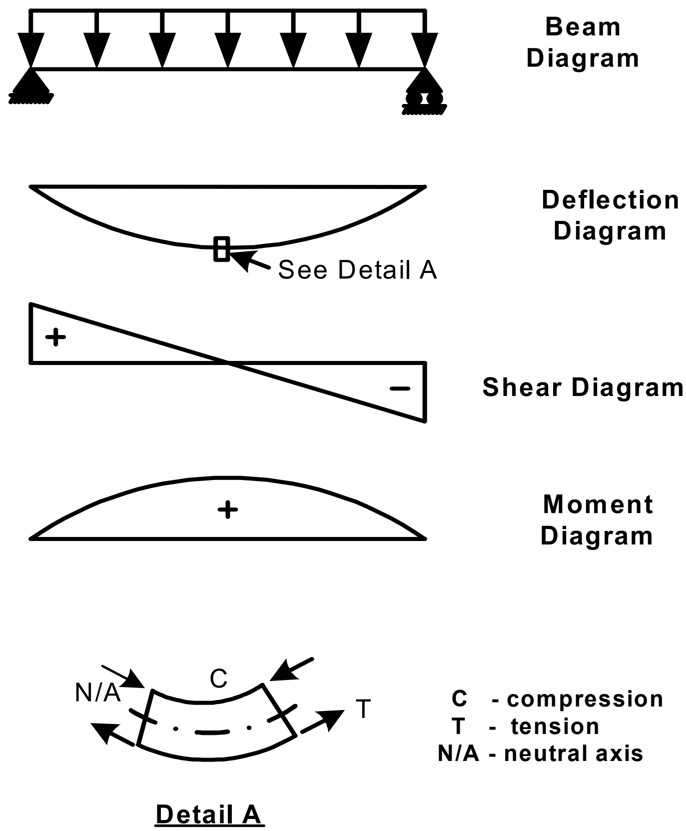


Figure P.1.21 Simple Span

A simple span bridge can have a single span supported at the ends by two abutments or multiple spans with each span behaving independently of the others. Some characteristics of simple span bridges are:

- When loaded, the span deflects downward and rotates at the supports (i.e., the abutments)
- The sum of the reactions provided by the two supports equals the entire load
- Shear forces are maximum at the supports and zero at or near the middle of the spans
- Bending moment throughout the span is positive and maximum at or near the middle of the span (the same location at which shear is zero); bending moment is zero at the supports
- The part of the superstructure below the neutral axis is in tension while the portion above the neutral axis is in compression

A simple span bridge is easily analyzed using equilibrium equations. However, it does not always provide the most economical design solution.

Continuous

A continuous span is a configuration in which a beam has one or more intermediate supports and the behavior of each individual span is dependent on its adjacent spans (see Figure P.1.22).

A continuous span bridge is one which is supported at the ends by two abutments and which spans uninterrupted over one or more piers. Some characteristics of continuous span bridges are:

- When loaded, the spans deflect downward and rotate at the supports (i.e., the abutments and the piers)
- The reactions provided by the supports depend on the span configuration and the distribution of the loads
- Shear forces are maximum at the supports and zero at or near the middle of the spans
- Positive bending moment is greatest at or near the middle of each span
- Negative bending moment is greatest at the intermediate supports (i.e., the piers); the bending moment is zero at the end supports (i.e., the abutments); there are also two locations per intermediate support at which bending moment is zero, known as inflection points
- For positive bending moments, compression occurs on the top portion of the beam and tension occurs on the bottom portion of the beam
- For negative bending moments, tension occurs on the top portion of the beam and compression occurs on the bottom portion of the beam

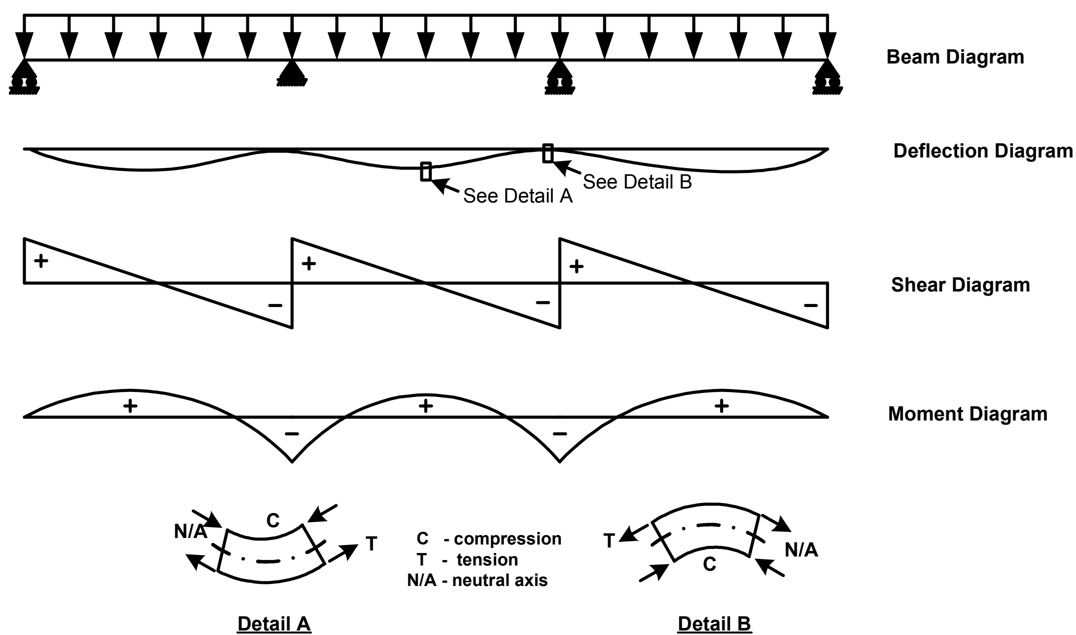


Figure P.1.22 Continuous Span

A continuous span bridge allows longer spans and is more economical than a bridge consisting of many simple spans. This is due to its efficient design with members that are more shallow. However, a continuous bridge is more difficult to analyze than a simple span bridge and is more susceptible to overstress conditions if the abutments or piers settle. Simple span bridges and continuous span bridges are both commonly used.

Cantilever

A cantilever span is a span with one end restrained against rotation and deflection and the other end completely free (see Figure P.1.23). The restrained end is also known as a fixed support (see Figure P.1.17).

While a cantilever generally does not form an entire bridge, portions of a bridge can behave as a cantilever (e.g., cantilever bridges and bascule bridges). Some characteristics of cantilevers are:

- When loaded, the span deflects downward, but there is no rotation or deflection at the support
- The fixed support reaction consists of a vertical force and a resisting moment
- The shear is maximum at the fixed support and is zero at the free end
- The bending moment throughout the span is negative and maximum at the fixed support; bending moment is zero at the free end

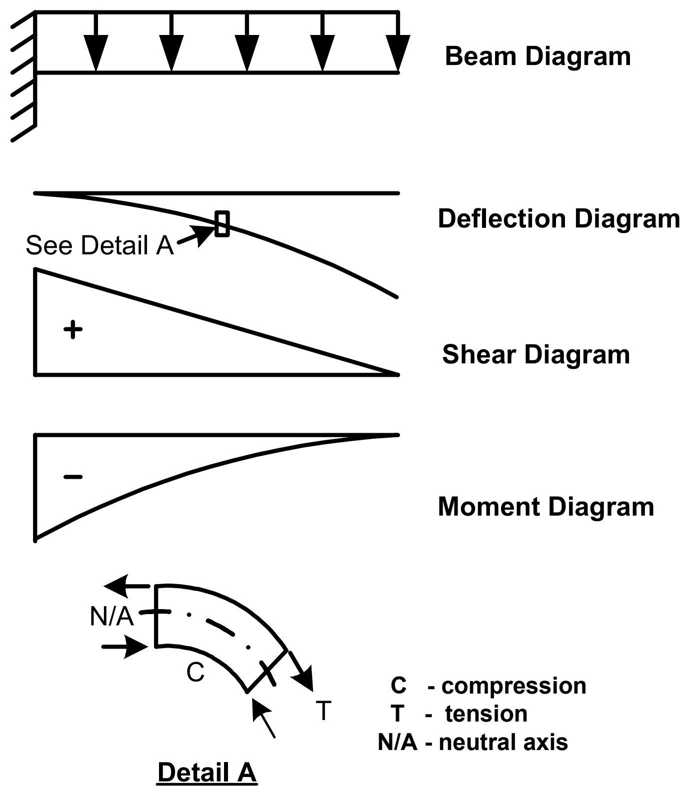


Figure P.1.23 Cantilever Span

When cantilever spans are incorporated into a bridge, they are generally extensions of a continuous span. Therefore, moment and rotation at the cantilever support will be dependent on the adjacent span (see Figure P.1.24).

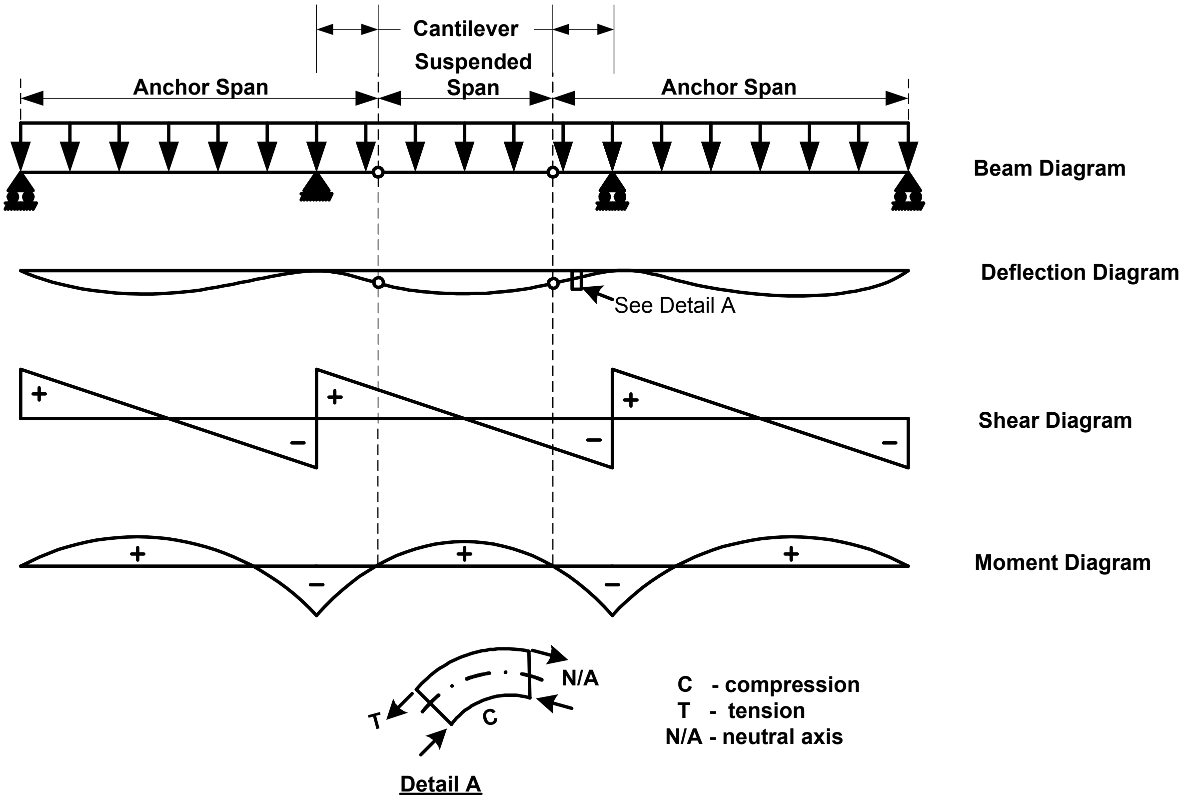


Figure P.1.24 Cantilever Bridge

P.1.10 Bridge Roadway Interaction

Bridges also have two classifications that are based on the relationship between the deck and the beams. These classifications are:

- Non-composite
- Composite

Non-composite

A non-composite structure is one in which the beams act independently of the deck. Therefore, the beams alone must resist all of the loads applied to them, including the dead load of the beams, deck, and railing, and all of the live loads.

Composite

A composite structure is one in which the deck acts together with the beams to resist the loads (see Figure P.1.25). The deck material must be strong enough to contribute significantly to the overall strength of the section. The deck material is different than the superstructure material. The most common combinations are concrete on steel and concrete on prestressed concrete. Shear connectors such as studs, spirals, channels, or stirrups that are attached to the beams and are embedded in a concrete deck provide composite action. This ensures that the beams and the deck will act as a unit by preventing slippage between the two when a load is applied.

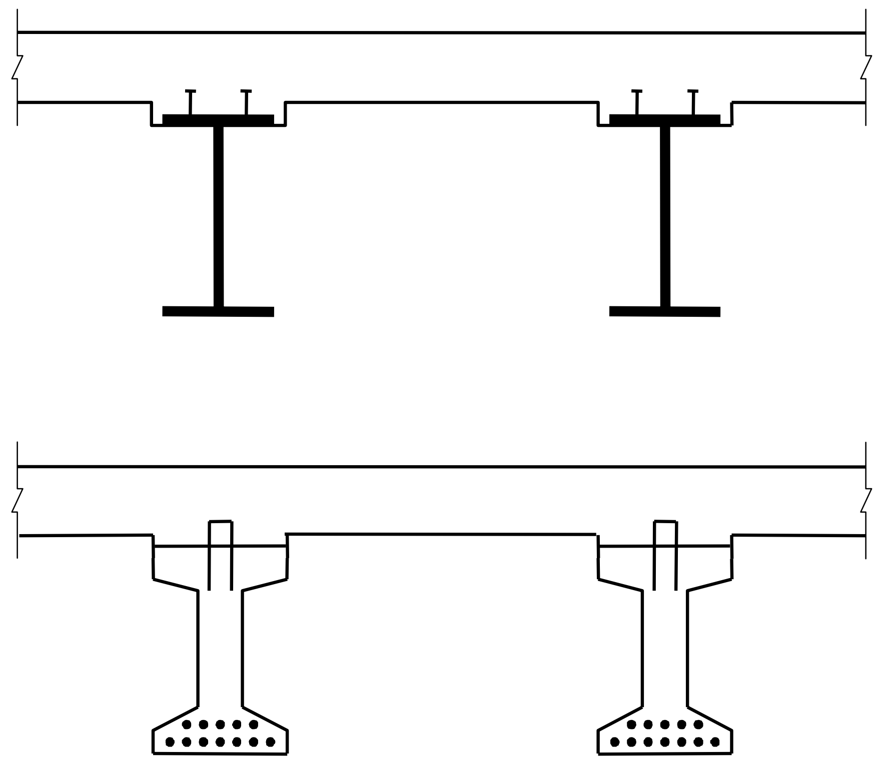


Figure P.1.25 Composite Concrete Deck on Steel Beams and Pretressed Concrete Beams

Composite action is achieved only after the concrete deck has hardened. Therefore, some of the dead load (Dead Load 1) must be resisted by the non-composite action of the beam alone. These dead loads include the weight of:

- The beam itself
- Any diaphragms and cross-bracing
- The concrete deck
- Any concrete haunch between the beam and the deck
- Any other loads which are applied before the concrete deck has hardened

Other dead loads, known as superimposed dead loads (Dead Load 2), are resisted by the beam and the concrete deck acting compositely. Superimposed dead loads include the weight of:

- Any anticipated future deck pavement
- Parapets
- Railings
- Any other loads which are applied after the concrete deck has hardened

Since live loads are applied to the bridge only after the deck has hardened, they are also resisted by the composite section.

The bridge inspector can identify a simple span, a continuous span, and a cantilever span based on their configuration. However, the bridge inspector can not identify the relationship between the deck and the beams while at the bridge site. Therefore, bridge plans must be reviewed to determine whether a structure is non-composite or composite.

Integral

On an integral bridge deck, the deck portion of the beam is constructed to act integrally with the stem, providing greater stiffness and allowing increased span lengths (see Figure P.1.26).

Figure P.1.26 Integral Bridge Deck

Orthotropic

An orthotropic deck consists of a flat, thin steel plate stiffened by a series of closely spaced longitudinal ribs at right angles to the floor beams. The deck acts integrally with the steel superstructure. An orthotropic deck becomes the top flange of the entire floor system. Orthotropic decks are occasionally used on large bridges (see Figure P.1.27).

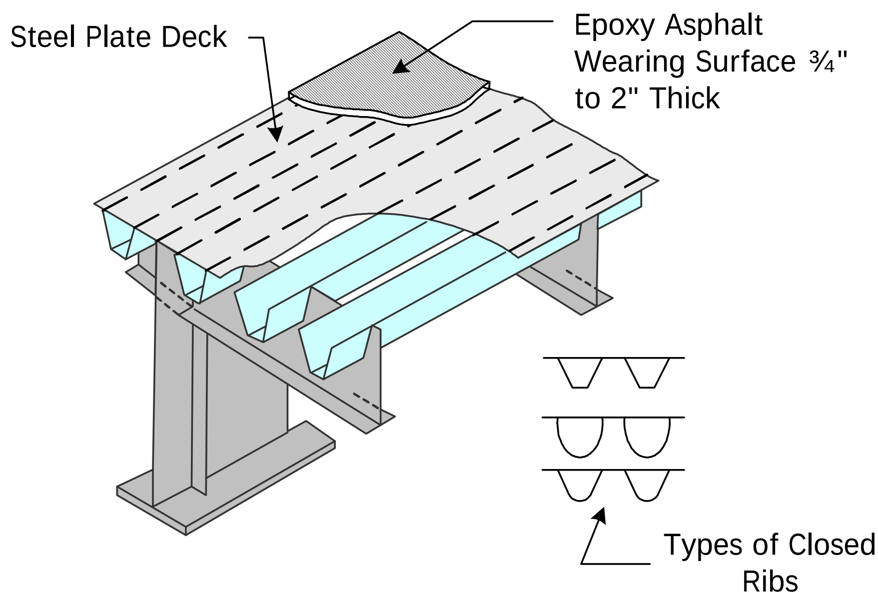


Figure P.1.27 Orthotropic Bridge Deck

P.1.11 Redundancy

Redundancy in bridge design is a configuration in which a bridge or bridge member has three or more independent load paths so that failure of one member or member element would not result in total failure.

There are three types of redundancy in bridge design.

Load Path Redundancy

Bridge designs that are load path redundant have three or more main load-carrying members or load paths. If one member were to fail, load would be redistributed to the other members and bridge failure would not occur. Bridge designs that are non-redundant have two or fewer main load carrying members or load paths.

Structural Redundancy

Most 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. In the event of a member failure, loading from that span can be redistributed to the adjacent spans and total bridge failure would not occur.

Internal Redundancy

Internal redundancy is when a bridge member contains several elements which are mechanically fastened together so that multiple load paths are formed. Failure of one member element would not cause total failure of the member.

Redundancy is discussed in greater detail in Topic 8.1.

P.1.12 Foundations

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

Spread Footings

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 slab made of reinforced concrete. This type of foundation "spreads out" the loads from the bridge to the underlying rock or well-compacted 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 just below the recognized maximum frost line depth for that area (see Figure P.1.28).

Pile Foundations

A pile foundation is used when the soil is not suited for supporting the bridge or when the bedrock is not close to the ground surface. A pile is a long, slender support that is typically driven into the ground but can be partially exposed. It is made from steel, concrete, 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 caisson," "drilled shaft" and "bored pile" are frequently used by engineers to denote drilled pile construction, sometimes referred to as pier foundations (see Figure P.1.29).

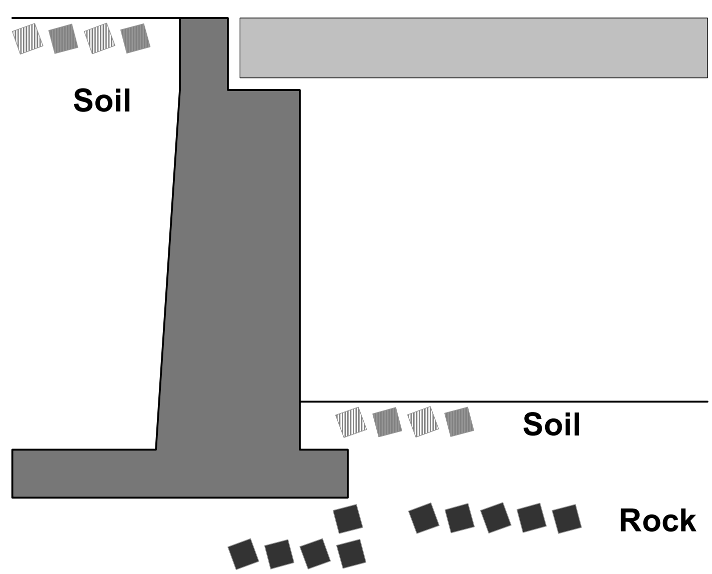


Figure P.1.28 Spread Footing

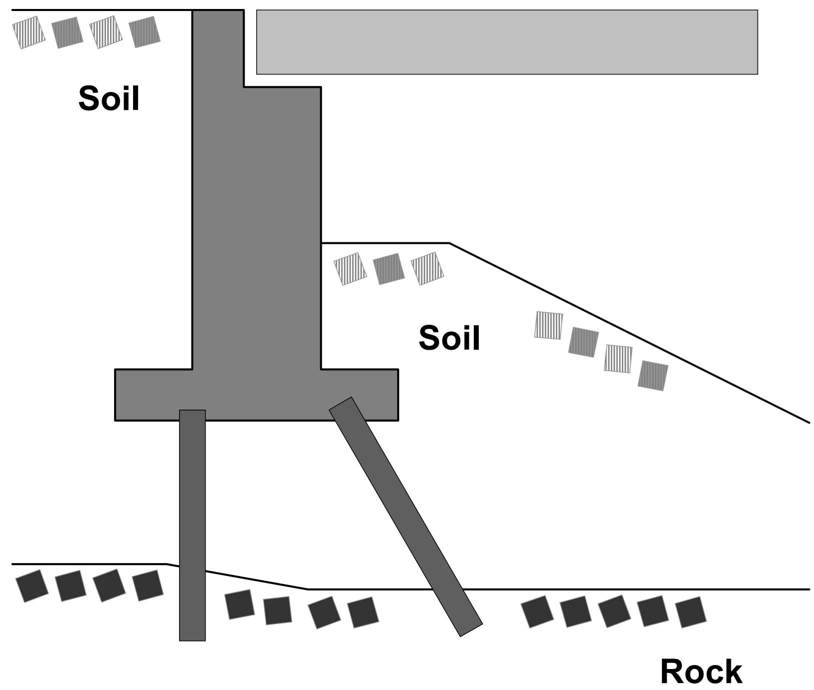


Figure P.1.29 Pile Foundation

Table of Contents

Topic P.2 Bridge Components and Elements

P.2.1 Introduction

The bridge inspector should be familiar with the terminology and elementary theory of bridge mechanics and materials. This topic presents the terminology needed by inspectors to properly identify and describe the individual elements that comprise a bridge. First the major components of a bridge are introduced. Then the basic member shapes and connections of the bridge are presented. Finally, the purpose and function of the major bridge components are described in detail.

P.2.2 NBIS Structure Length

According to the Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges the minimum length for a structure carrying traffic loads is 6.1meters (20 feet). The structure length is measured as shown on Figure P.2.1

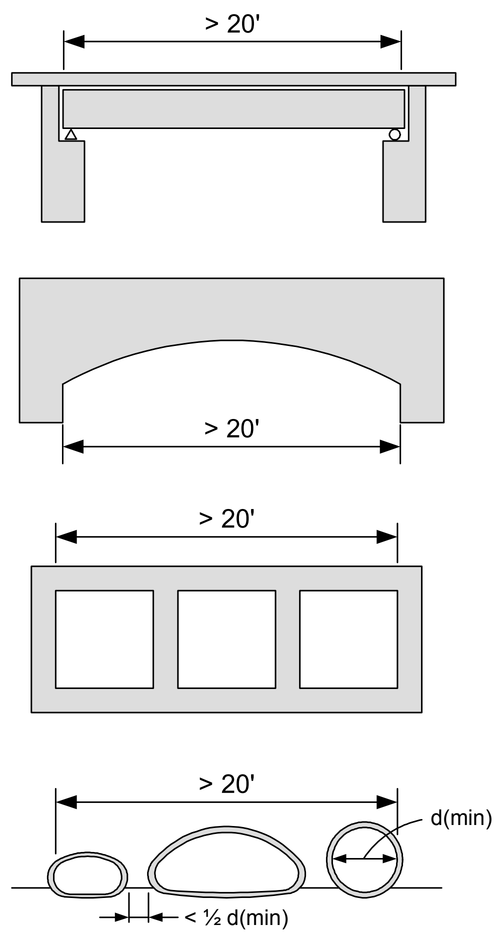


Figure P.2.1 NBIS Structure Length

P.2.3 Major Bridge Components

A thorough and complete bridge inspection is dependent upon the bridge inspector's ability to identify and understand the function of the major bridge components and their elements. Most bridges can be divided into three basic parts or components (see Figure P.2.1A):

- Deck
- Superstructure
- Substructure

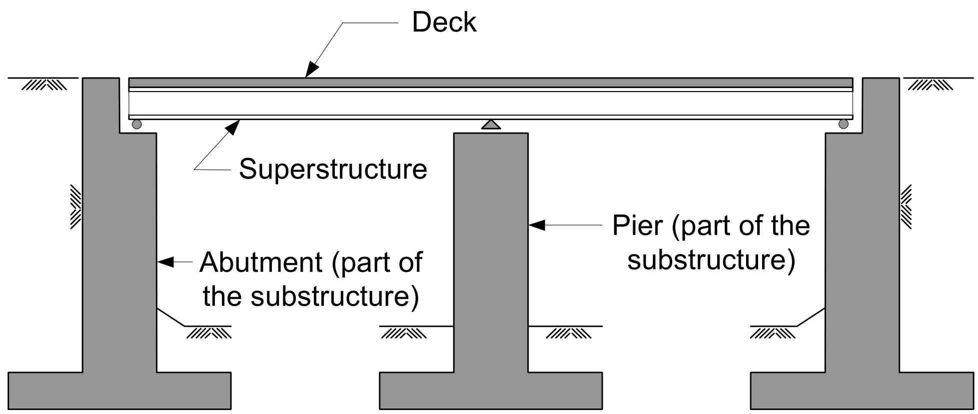


Figure P.2.1A Major Bridge Components

P.2.4 Basic Member Shapes

The ability to recognize and identify basic member shapes requires an understanding of the timber, concrete, and steel shapes used in the construction of bridges.

Every bridge member is designed to carry a unique combination of tension, compression, and shear. These are considered the three basic kinds of member stresses. Bending loads cause a combination of tension and compression in a member. Shear stresses are caused by transverse forces exerted on a member. As such, certain shapes and materials have distinct characteristics in resisting the applied loads. For a review of bridge loadings and member responses, see Topic P.1.

Timber Shapes

Basic shapes, properties, gradings, deteriorations, protective systems, and examination of timber are covered in detail in Topic 2.1.

Timber members are found in a variety of shapes (see Figure P.2.2). The sizes of timber members are generally given in nominal dimensions (such as in Figures P.2.2 through P.2.4). However, timber members are generally seasoned and surfaced from the rough sawn condition, making the actual dimension about 13 to 20 mm (1/2 to 3/4 inches) less than the nominal dimension.

The physical properties of timber enable it to resist both tensile and compressive stresses. Therefore, it can function as an axially-loaded or bending member. Timber bridge members are made into three basic shapes:

- Planks
- Beams
- Piles

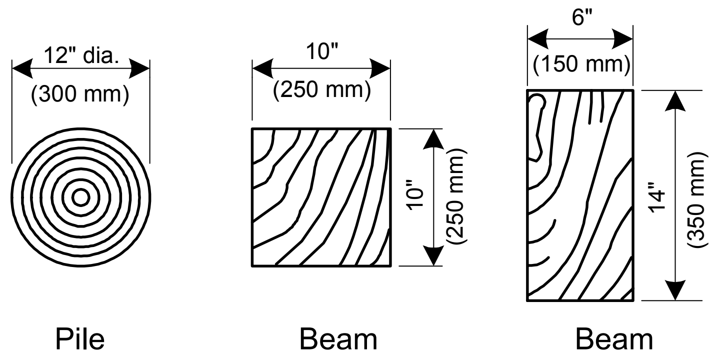


Figure P.2.2 Timber Shapes

Planks

Planks are characterized by elongated, rectangular dimensions determined by the intended bridge use. Plank thickness is dependent upon the distance between the supporting points and the magnitude of the vehicle load. A common dimension for timber planks is a 2” x 12” (50 mm x 300 mm), nominal or rough sawn. Dressed lumber dimensions would be 1 ½” x 11’ 4” (38 mm x 285 mm) (see Figure P.2.3).

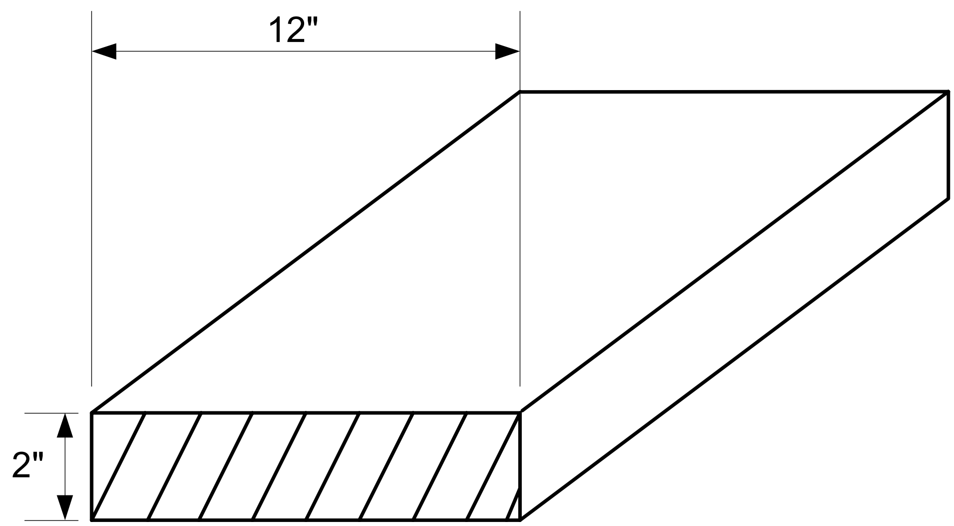


Figure P.2.3 Timber Plank

Planks are most often used for bridge decks on bridges carrying light or infrequent truck traffic. While some shapes and materials are relatively new, the use of timber plank decks has existed for centuries. Timber planks are advantageous in that they are economical, lightweight, readily available, and easy to erect.

Beams

Timber beams have more equal rectangular dimensions than do planks, and they are sometimes square. Common dimensions include 250 mm by 250 mm (10 inches by 10 inches) square timbers, and 150 mm by 350 mm (6 inches by 14 inches) rectangular timbers.

As the differences in the common dimensions of planks and timber beams indicate, beams are larger and heavier than planks and can support heavier loads, as well as span greater distances. As such, timber beams are used in bridge superstructures and substructures to carry bending and axial loads.

Timbers can either be solid sawn or glued-laminated (see Figure P.2.4). Glued-laminated timbers are advantageous in that they can be fabricated from smaller, more readily available pieces. Glued lamination also allows larger rectangular members to be formed without the presence of natural defects such as knots. Glued-laminated timbers are normally manufactured from well-seasoned laminations and display very little shrinkage after they are made.

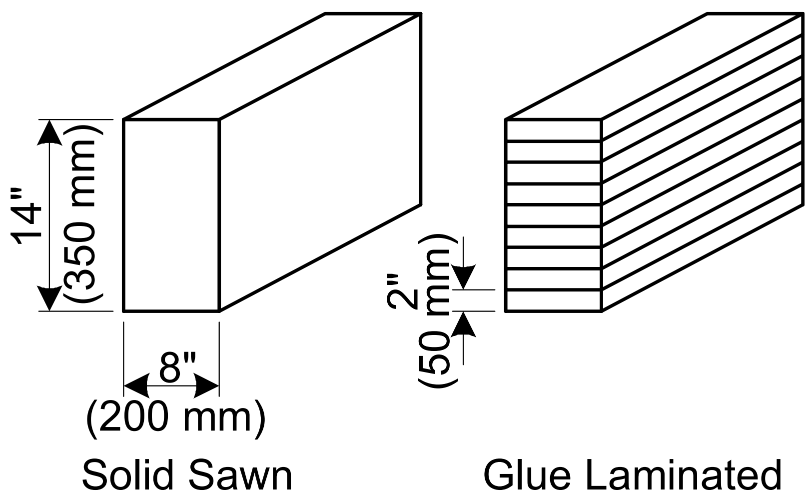


Figure P.2.4 Timber Beams

Piles

Timber can also be used for piles. Piles are normally round, slender columns that support the substructure footing or partially form the substructure. Piles may be partially above ground but are usually completely buried.

Concrete Shapes

Basic ingredients, properties, reinforcement, deterioration, protective systems, and examination of concrete are covered in detail in Topic 2.2.

Figure P.2.5 Unusual Concrete Shapes

Concrete is a unique material for bridge members because it can be formed into an infinite variety of shapes (see Figure P.2.5). Concrete members are used to carry axial loads and loads in bending. Since bending is really a combination of compressive and tensile stresses, plain concrete is a poor material to resist bending. Concrete bending members are typically reinforced with either reinforcing steel (producing reinforced concrete) or with prestressing steel (producing prestressed concrete) in order to carry the tensile stresses in the member. The cost of prestressing steel is greater than that of reinforcing steel. However, because less steel is used in prestressed concrete, it can be more economical to use.

Cast-in-Place Flexural Shapes

The most common shapes of reinforced concrete members are (see Figure P.2.6):

- Slabs
- Rectangular beams
- Tee beams
- Channel beams

Bridges utilizing these shapes and mild steel reinforcement have been constructed and were typically cast-in-place (CIP). Many of the designs are obsolete, but the structures remain in service. Concrete members of this type are used for short and medium span bridges.

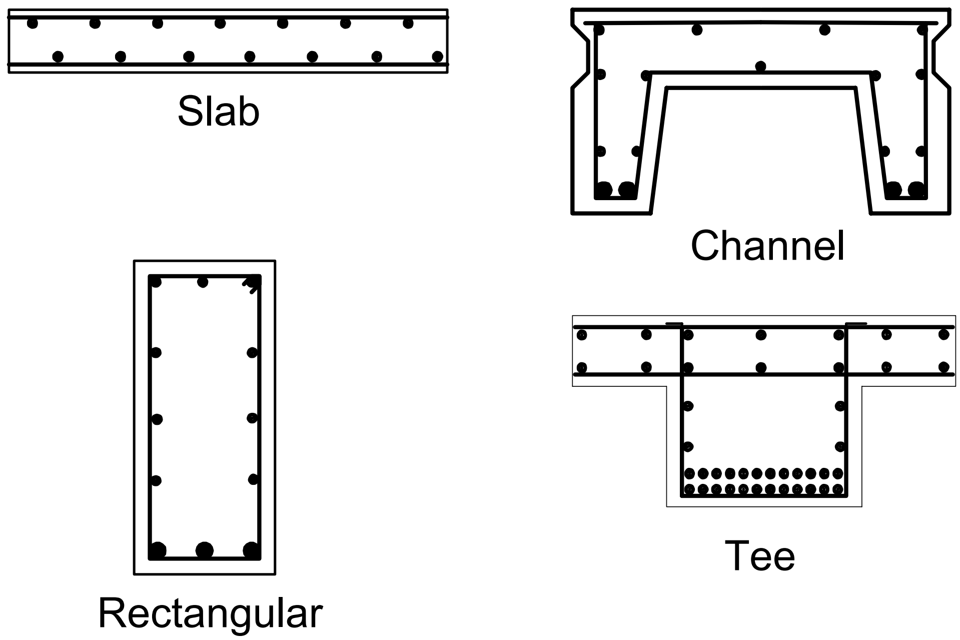


Figure P.2.6 Reinforced Concrete Shapes

Concrete slabs are used for concrete decks and slab bridges. On concrete decks, the concrete spans the distance between superstructure members and is generally 180 to 230 mm (7 to 9 inches thick). On slab bridges, the slab spans the distance between piers or abutments, forming an integral deck and superstructure. Slab bridge elements are usually 300 to 600 mm (12 to 24 inches) thick.

Rectangular beams are used for both superstructure and substructure bridge elements. Concrete pier caps are commonly rectangular beams which support the superstructure.

Bridge use for tee beams is generally limited to superstructure elements. Distinguished by a "T" shape, tee beams combine the functions of a rectangular beam and slab to form an integral deck and superstructure.

Bridge use for channel beams is limited to superstructure elements. This particular shape is precast rather than cast-in-place. Channel beams are formed in the shape of a "C" and placed legs down when erected. They function as both superstructure and deck and are typically used for shorter span bridges. A wearing course is often added to provide the riding surface.

Precast Flexural Shapes

The most common shapes of prestressed concrete members are (see Figure P.2.7):

- I-beams
- Bulb-tees
- Box beams

- Box girders
- Voided slabs

These shapes are used for superstructure members.

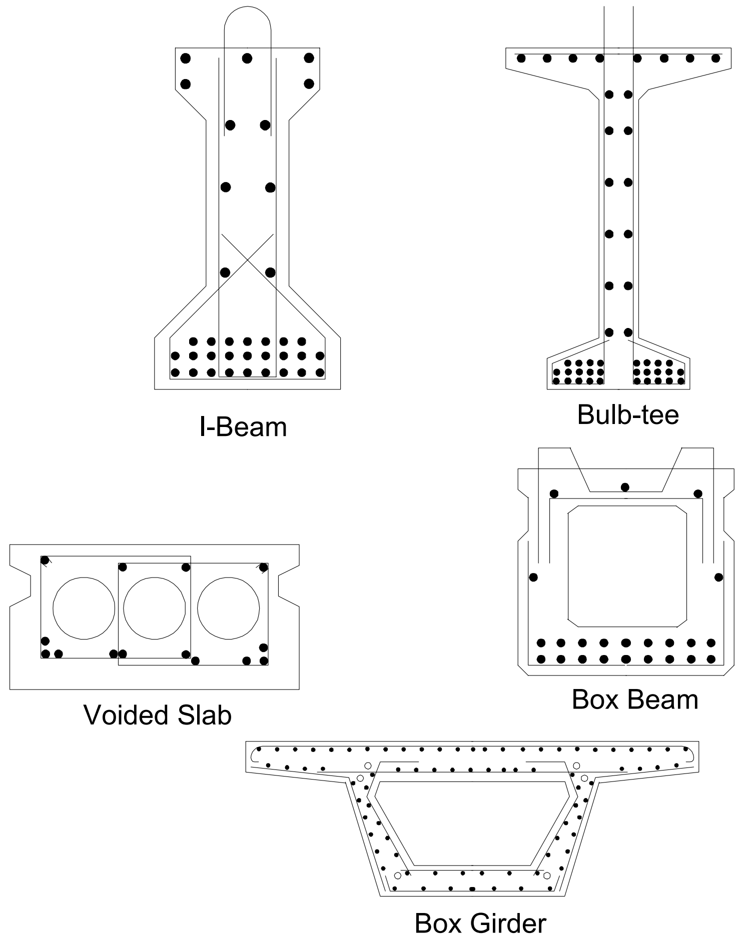


Figure P.2.7 Prestressed Concrete Shapes

Prestressed concrete beams can be precast at a fabricator's plant using high strength concrete. Increased material strengths, more efficient shapes, and the prestress forces allow these members to carry greater loads. Therefore, they are capable of spanning greater distances and supporting heavier live loads. Bridges using members of this type and material have been widely used in the United States since World War II.

Prestressed concrete is generally more economical than conventionally reinforced concrete because the prestressing force lowers the neutral axis, putting more of the concrete section into compression. Also, the prestress steel is very high strength, so fewer pounds of steel are needed (see Figure P.2.8).

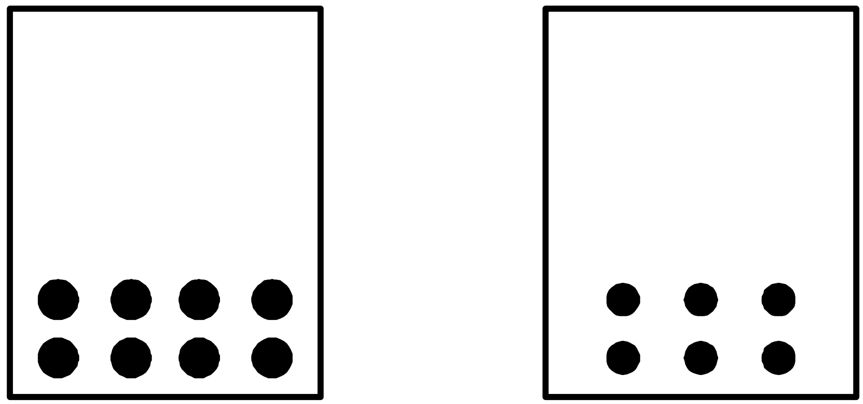


Figure P.2.8 Mild Steel Reinforced Concrete vs. Precast Prestressed Concrete

I-beams, distinguished by their "I" shape, function as superstructure members and support the deck. This type of beam can be used for spans as long as 46 m (150 feet).

Bulb-tee beams are distinguished by their "T" shapes, with a bulb-shaped section (similar to the bottom flange of an I-beam) at the bottom of the vertical leg of the tee. This type of beam can be used for spans as long as 55 m (180 feet).

Box beams, distinguished by a square or rectangular shape, usually have a beam depth greater than 430 mm (17 inches). Box beams can be adjacent or spread, and they are typically used for short and medium span bridges.

Box girders, distinguished by their trapezoidal box shapes, function as both deck and superstructure. Box girders are used for long span or curved bridges. They can be precast and erected in segments or cast in place.

Voided slabs, distinguished by their rectangular shape and their interior voids, are generally precast units placed parallel with the roadway alignment. The interior voids are used to reduce the dead load. Voided slabs can be used for spans of 30 to 9 to 24 m (80 feet).

Axially-Loaded Compression Members

Concrete axially-loaded compression members are used in bridges in the form of:

- Columns
- Arches
- Piles

Because these members also carry varying bending forces, they contain steel reinforcement.

Columns are straight members which can carry axial load, horizontal load, and bending and are used as substructure elements. Columns are commonly square, rectangular, or round.

An arch can be thought of as a curved column and is commonly used as a superstructure element. Concrete superstructure arches are generally square or rectangular in cross section.

Piles are slender columns that support the substructure footing or partially form the substructure. Piles may be partially above ground but are usually completely buried (see Figure P.2.9).

Figure P.2.9 Concrete Pile Bent

Iron Shapes

Iron was used predominately as a bridge material between 1850 and 1900. Stronger and more fire resistant than wood, iron was widely used to carry the expanding railroad system during this period.

There are two types of iron members: cast iron and wrought iron. Cast iron is formed by casting, whereas wrought iron is formed by forging or rolling the iron into the desired form.

Cast Iron

Historically, cast iron preceded wrought iron as a bridge material. The method of casting molten iron to form a desired shape was more direct than that of wrought iron.

Casting allowed iron to be formed into almost any shape. However, because of cast iron's brittleness and low tensile strength, bridge members of cast iron were best used to carry axial compression loads. Therefore, cast iron members were usually cylindrical or box-shaped to efficiently resist axial loads.

Wrought Iron

In the late 1800's, wrought iron virtually replaced the use of cast iron. The two primary reasons for this were that wrought iron was better suited to carry tensile loads and advances in rolling technology made wrought iron shapes easier to obtain and more economical to use. Advances in technology made it possible to form a variety of shapes by rolling, including:

- Rods and wire
- Bars
- Plates
- Angles
- Channels
- Beams

Steel Shapes

Steel bridge members began to be used in the United States in the late 1800's and, by 1900, had virtually replaced iron as a bridge material. The replacement of iron by steel was the result of advances in steel making (see Figure P.2.10). These advances yielded a steel material that surpassed iron in both strength and elasticity. Steel could carry heavier loads and better withstand the shock and vibration of ever-increasing live loads. Since the early 1900's, the quality of steel has continued to improve. Stronger and more ductile A36, A572, and A588 steels have replaced early grades of steel, such as A7.

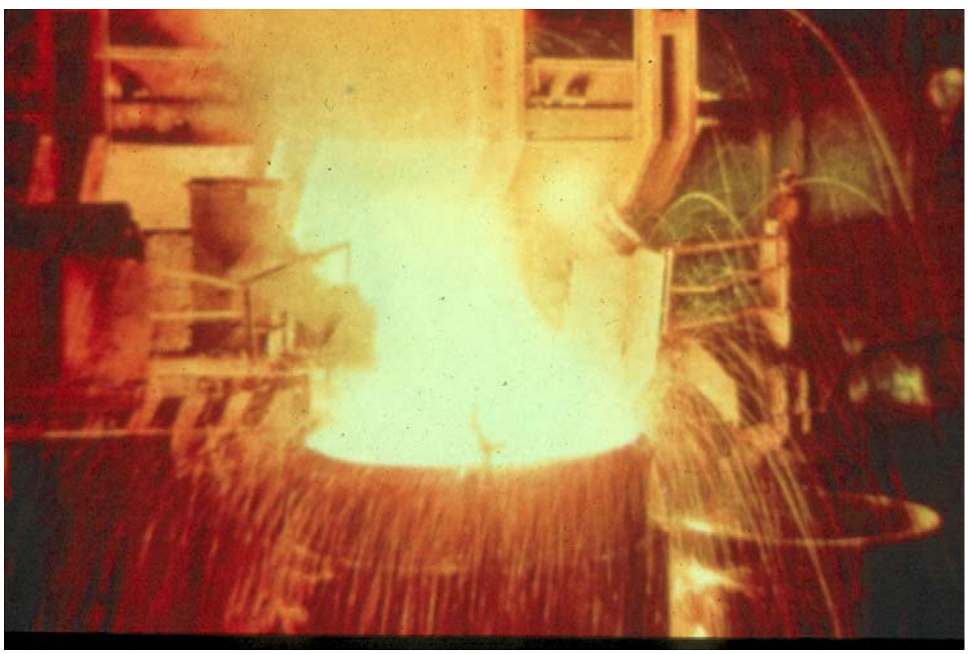


Figure P.2.10 Steel Making Operation

Due to their strength, steel bridge members are used to carry axial forces as well as bending forces. Steel shapes are generally either rolled or built-up.

Rolled Shapes

Rolled steel shapes commonly used on bridges include (see Figure P.2.11):

- Bars and plates
- Angles
- Channels
- S Beams
- W Beams

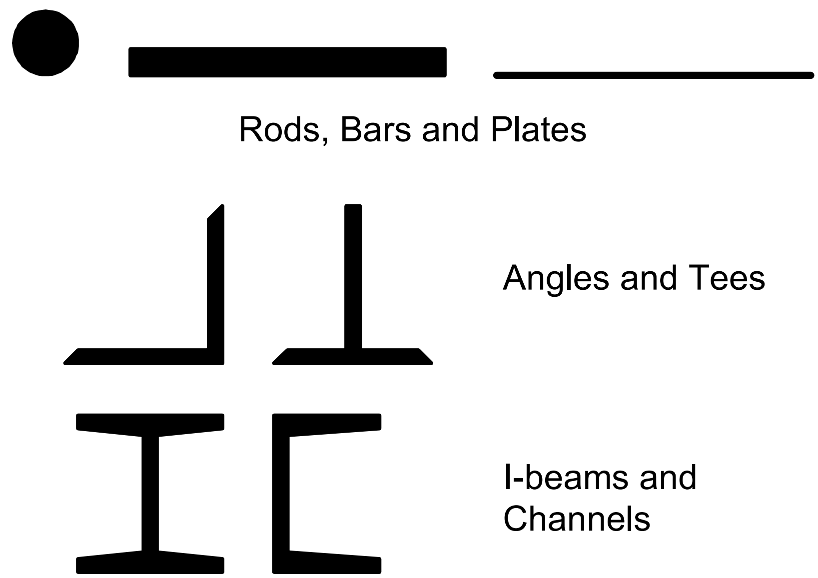


Figure P.2.11 Common Rolled Steel Shapes

The standard weights and dimensions of these shapes can be found in the American Institute of Steel Construction (AISC) *Manual of Steel Construction*.

Bars and plates are formed into flat pieces of steel. Bars are normally considered to be up to 200 mm (8 inches) in width. Common examples of bars include lacing bars on a truss and steel eyebars. Plates are designated as flat plates if they are over 200 mm (8 inches) in width. A common example of a plate is the gusset plate on a truss. Bars and plates are dimensioned as follows: width (in mm or inches) x thickness (in mm or inches) x length (in meters or feet and inches). Examples of bar and plate dimensions include:

- Lacing bar: 50 mm x 10 mm x 1.2 m (2”x3/8”x1’-3”)
- Gusset plate: 530 mm x 12 mm x 1.3 m (21”x1/2”x4’-4”)

Angles are "L"-shaped members, the sides of which are called "legs." Each angle has two legs, and the width of the legs can either be equal or unequal. When dimensioning angles, the two leg widths are given first, followed by the thickness and the length. Examples of angle dimensions include:

- L 4 x 4 x ¼ x 3’-2” (L 102 x 102 x 6.4 x 965)

➢ 2L's 5 x 3 x 3/8 x 1'-1" (2L's 127 x 76 x 9.5 x 330)

Angles range in size from 1"x1"x1/4" to 8"x8"x1-1/8". Angles range in weight from less than 14.6 N/m (1 pound per foot) to almost 880 N/m (60 pounds per foot).

Angles, bars, and plates are commonly connected to form bracing members (see Figure P.2.12).

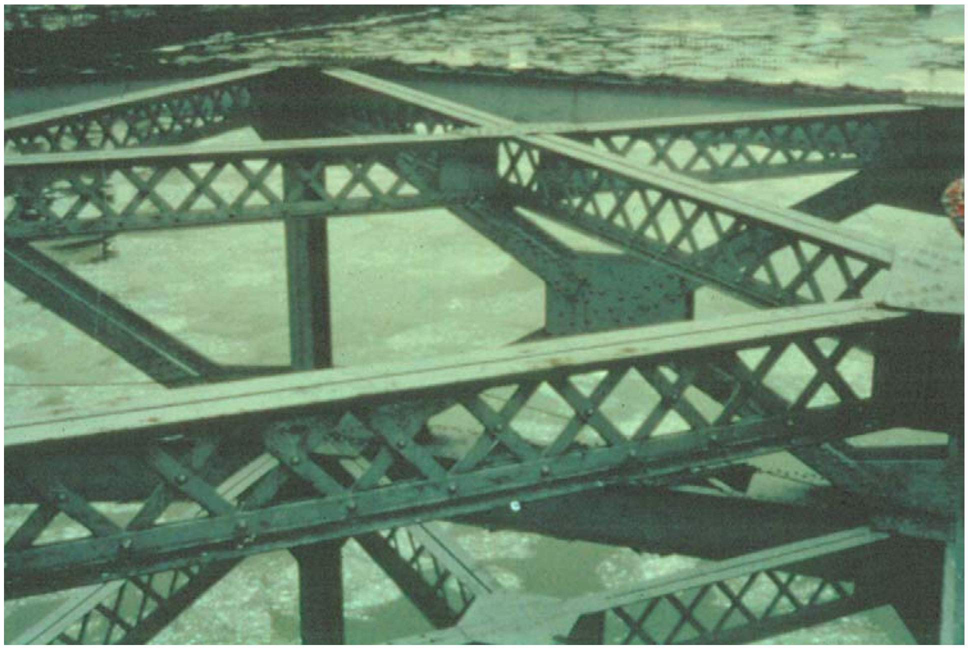


Figure P.2.12 Bracing Members Made from Angles, Bars, and Plates

Channels are squared-off "C"-shaped members and are used as diaphragms, struts, or built-up members. The top and bottom parts of a channel are called the flanges. Channels are dimensioned by the depth (the distance between outside edges of the flanges) in mm or inches, the weight in kg per m or pounds per foot, and the length. Examples of channel dimensions include:

➢ C 9 x 15 x 9'-6" (C 230 x 22 x 2895)
➢ C 12 x 20.7 x 11'-2-1/2" (C 310 x 31 x 3416)

When measuring a channel, it is not possible for the inspector to know how much the channel section weighs. In order to determine the weight, the inspector must record the flange width and the web depth. From this information, the inspector can then determine the true channel designation through the use of reference books.

Standard channels range in depth from 75 mm to 380 mm (3 inches to 15 inches), and weights range from less than 5 pounds per foot to 73 n/m to 730 n/m (50 pounds per foot). Nonstandard sections (called miscellaneous channels or MC) are rolled to depths of up to 600 mm (24 inches), weighing up to 845 n/m (60 pounds per foot).

Beams are "I"-shaped sections used as main load-carrying members. The load-carrying capacity generally increases as the member size increases. The early days of the iron and steel industry saw the various manufacturers rolling beams to their own standards. It was not until 1896 that beam weights and dimensions were standardized when the Association of American Steel Manufacturers adopted the American Standard beam. Because of this, I-beams are referred to by many designations, depending on their dimensions and the time period in which the particular shape was rolled. Today all I-beams are dimensioned according to their depth, weight, and length.

Examples of beam dimensions include:

- S15x50 (S380x74)- an American Standard (hence the "S") beam with a depth of 15 inches and a weight of 50 pounds per foot
- W18x76 (W460x113) - a wide (W) flange beam with a depth of 18 inches and a weight of 76 pounds per foot

Some of the more common designations for rolled I-beams are:

- S = American Standard beam
- W = Wide flange beam
- WF = Wide flange beam
- CB = Carnegie beam
- M = Miscellaneous beam
- HP = H-pile

When measuring an I-beam, the inspector needs to measure the depth, the flange width and thickness, and the web thickness (if possible). With this information, the inspector can then determine the beam designation from reference books.

These beams normally range in depth from 75 to 900 mm (3 to 36 inches) and range in weight from 90 to over 4380 n/m (6 to over 300 pounds) per foot. There are some steel mills that can roll beams up to 1120 mm (44 inches) deep.

Built-up Shapes

Built-up shapes offer a great deal of flexibility in designing member shapes. As such, they allow the bridge engineer to customize the members to their use. Built-up shapes are fabricated by either riveting or welding techniques.

The practice of riveting steel shapes began in the 1800’s and continued through the 1950’s. Typical riveted shapes include girders and boxes.

Riveted girders are large I-beam members fabricated from plates and angles. These girders were fabricated when the largest rolled beams were still not large enough (see Figure P.2.13).

Riveted boxes are large rectangular shapes fabricated from plates, angles, or channels. These boxes are used for cross-girders, truss chord members, and substructure members (see Figure P.2.14).

As technology improved, the need for riveting was replaced by high strength bolts and welding. Popular since the early 1960's, welded steel shapes also include girders and boxes.

Welded girders are large I-beam members fabricated from plates. They are referred to as welded plate girders and have replaced the riveted girder (see Figure P.2.15).

Welded boxes are large, box-shaped members fabricated from plates. Welded boxes are commonly used for superstructure girders, truss members, and cross girders. Welded box shapes have replaced riveted box shapes (see Figure P.2.16).

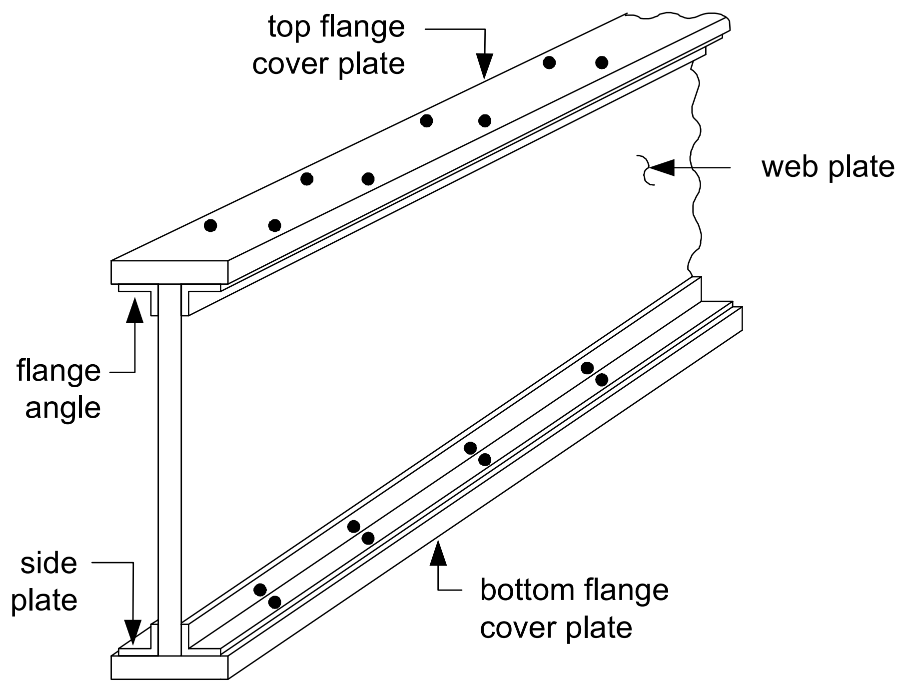


Figure P.2.13 Riveted Plate Girder

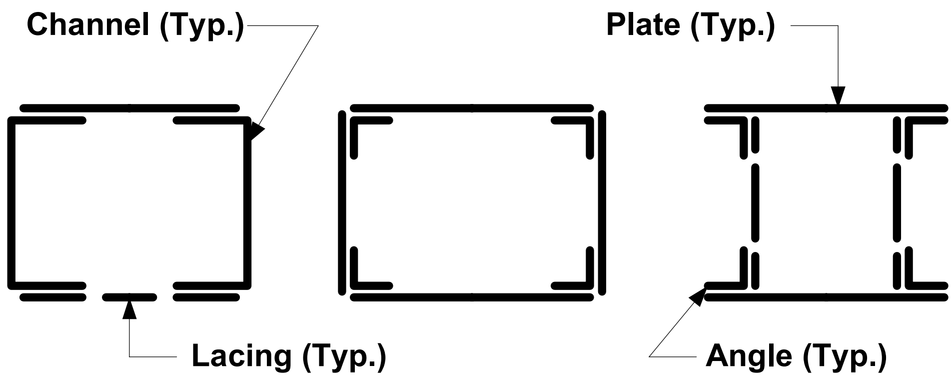


Figure P.2.14 Riveted Box Shapes

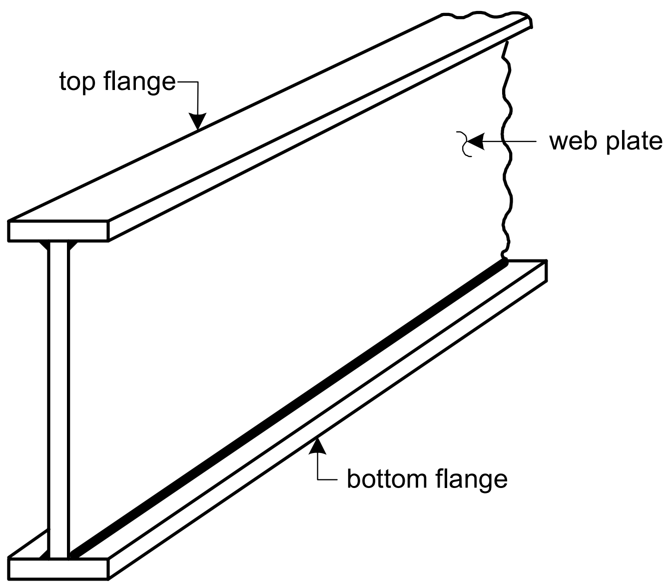


Figure P.2.15 Welded I-Beam

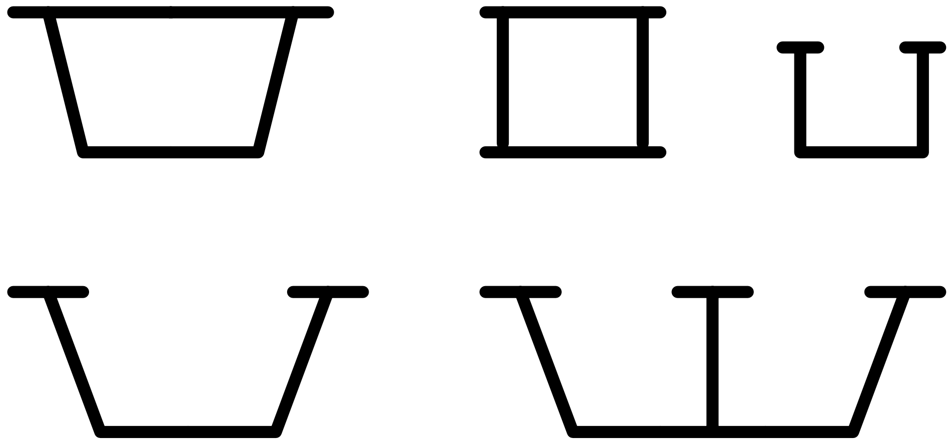


Figure P.2.16 Welded Box Shapes

Cables

Steel cables are tension members and are used in suspension, tied-arch, and cable-stayed bridges. They are used as main cables and hangers of these bridge types (see Figure P.2.17). Refer to Topic 12.1 for a more detailed description of cable-supported bridges.

Figure P.2.17 Cable-Supported Bridge

P.2.5 Connections

Rolled and built-up steel shapes are used to make stringers, floor beams, girders, and truss members. These members require structural joints, or connections, to transfer loads between members. There are several different types of bridge member connections:

- Pin connections
- Riveted connections
- Bolted connections
- Welded connections
- Pin and hanger connections
- Splice connections

Pin Connections

Pins are cylindrical beams produced by either forging, casting, or cold-rolling. The pin sizes and configurations are as follows (see Figure P.2.18):

- A small pin, 32 to 100m (1-1/4 to 4 inches) in diameter, is usually made with a cotter pin hole at one or both ends
- A medium pin, up to 250 mm (10 inches) in diameter, usually has threaded end projections for recessed retainer nuts
- A large pin, over 250 mm (10 inches) in diameter, is held in place by a recessed cap at each end and is secured by a bolt passing completely through the caps and pin

Pins are often surrounded by a protective sleeve, which may also act as a spacer to separate members. Pin connections are commonly used in eyebar trusses, hinged arches, and bearing supports (see Figure P.2.19).

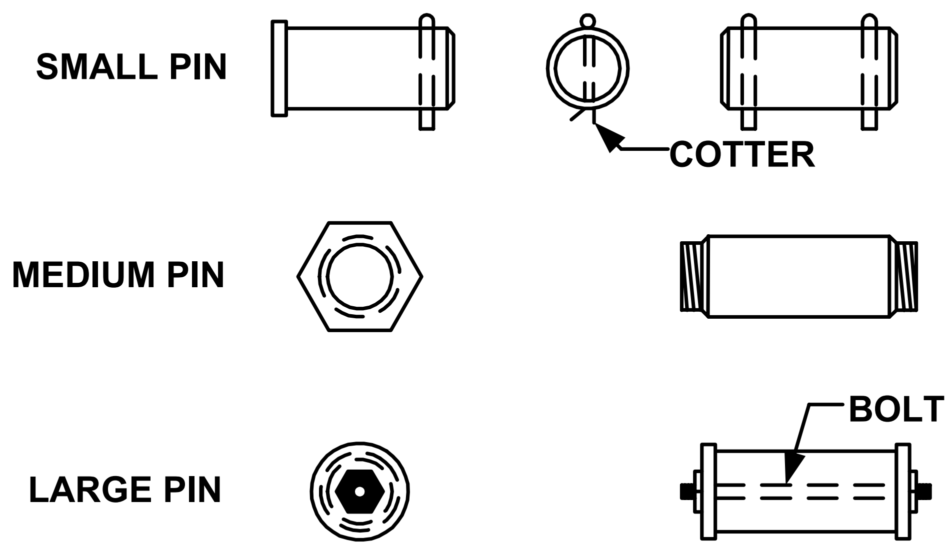


Figure P.2.18 Sizes of Bridge Pins

Figure P.2.19 Pin-Connected Truss Members

The major advantages of using pin connection details are the design simplicity and the ability for free end rotation. The design simplicity afforded by pin connections reduces the amount and complexity of design calculations. By allowing for free end rotation, pin connections reduce the level of stress in the member.

The major disadvantages of pin connection details are the result of vibration, pin wear, unequal eyebar tension, unseen corrosion, and poor inspectability. Vibrations increase with pin connections because they allow more movement than more rigid types of connections. As a result of increased vibration, moving parts are subject to wear.

Pin connections are used both in trusses and at expansion joints. Both truss and girder suspended spans or cantilever joints that permit expansion are susceptible to freezing or fixity of the pinned joints. This results in changes in the structure and undesirable stresses when axially-loaded members become bending members.

Some pins connect multiple eyebars. Since the eyebars may have different lengths, they may experience different levels of tension. In addition, because parts of the pin surface are hidden from view by the eyebars, links, or connected parts, an alternate method of completely inspecting the pin must be used (e.g., ultrasonic or pin removal).

Riveted Connections

The rivet was the primary fastener used in the early days of iron and steel bridges. The use of high strength bolts replaced rivets by the early 1960’s.

The standard head is called a high-button or acorn-head rivet. Flat-head and countersunk-head rivets were also used in areas of limited clearance, such as an eyebar pin connection (see Figure P.2.20).

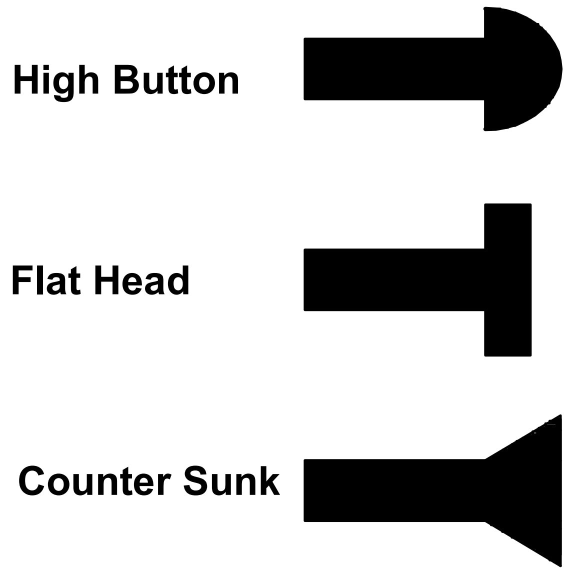


Figure P.2.20 Types of Rivet Heads

There are two grades of rivets typically found on bridges:

- ASTM A502 Grade 1 (formerly ASTM A141) low carbon steel
- ASTM A502 Grade 2 (formerly ASTM A195) high strength steel

The rivet sizes most often used on bridges were 3/4, 7/8, or 1-inch shank diameters. Rivet holes were generally 1/16-inch larger than the rivet shank. While the hot rivet was being driven, the shank would increase slightly, filling the hole. As the rivet cooled, it would shrink in length, clamping together the connected elements.

When the inspector can feel vibration on one head of the rivet while hitting the other head with a hammer, this generally indicates that the rivet is loose. This method may not work with sheared rivets clamped between several plates.

Bolted Connections

Research into the use of high strength bolts began in 1947. The first specifications for the use of bolts were subsequently published in 1951. The economic and structural advantages of bolts over rivets led to their rapid use by bridge engineers. Bridges constructed in the late 1950's may have a combination of riveted (shop) and bolted (field) connections (see Figure P.2.21).

Structural bolts come in three basic types:

- ASTM A307 low carbon steel
- ASTM A325 (AASHTO M 164) high strength steel
- ASTM A490 (AASHTO M 253) high strength alloy steel

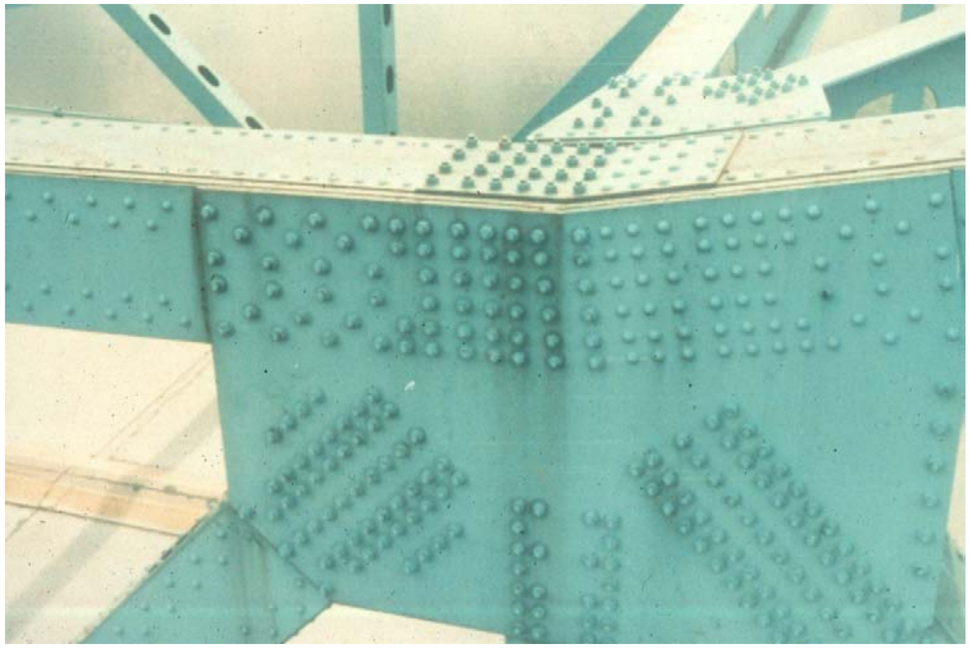


Figure P.2.21 Shop Rivets and Field Bolts

The most commonly used bolts on bridges are 3/4, 7/8, and 1-inch in diameter. Larger bolts are often used to anchor the bearings. Bolt holes are typically 1/16-inch larger than the bolt. However, oversized and slotted holes are also permissible.

The strength of high strength bolts is measured in tension. However, the inspection of high strength bolts on bridges involves many variables. Although the installation inspection of new high strength bolts often requires the use of a torque wrench, this method does not have any merit when inspecting high strength bolts on in-service bridges. The torque is dependent on factors such as bolt diameter, bolt length, connection design (bearing or friction), use of washers, paint and coatings, parallelism of connected parts, dirt, rust, and corrosion.

The inspector must be cautioned that standard tables and formulas relating tension to torque are no longer considered valid.

Simple techniques, such as looking and feeling for loose bolts, are the most common methods used by inspectors when inspecting for loose bolts.

Welded Connections

Pins, rivets, and bolts are examples of mechanical fasteners forming non-rigid joints. A welded connection is not mechanical but rather is rigid one-piece construction. A properly welded joint, in which two pieces are fused together, is as strong as the joined materials.

Similar to mechanical fasteners, welds are used to make structural connections between members and also to connect elements of a built-up member. Welds have also been used in the fabrication and erection of bridges as a way to temporarily hold pieces together prior to field riveting, bolting, or welding. Small temporary erection welds, known as tack welds, can cause serious problems to certain bridge members (see Figure P.2.22). Welding is also used as a means of sealing joints and seams from moisture.

Figure P.2.22 Close-up of Tack Weld on a Riveted Built-up Truss Member

The first specification for using welds on bridges appeared in 1936. Welding eventually replaced rivets for fabricating built-up members. Welded plate girders, hollow box-like truss members, and shear connectors for composite decks are just a few of the advances attributed to welding technology.

Welds need to be carefully inspected for cracks or signs of cracks (e.g., broken paint or rust stains) in both the welds and the adjoining base metal elements.

Pin and Hanger Connections

A pin and hanger connection is a type of hinge consisting of two pins and a hanger. Pin and hanger connections are used in an articulated (continuous bridge with hinges) or a suspended span configuration. The location of the connection varies depending on the type of bridge. In I-beam bridges, a hanger is located on either side of the webs (see Figure P.2.23). In suspended span truss bridges, each connection has a hanger which is similar in shape to the other truss members (with the exception of the pinned ends).

Figure P.2.23 Pin and Hanger Connection

Pin and hanger connections must be carefully inspected for signs of wear and corrosion. A potential problem can occur if corrosion of the pin and hanger causes the connection to "freeze," inhibiting free rotation. This condition violates the design, resulting in additional stresses in the pin and hanger and adjacent girder. The failure of a pin and hanger connection can cause a partial or complete failure of the bridge.

Splice Connections

A splice connection is the joining of two sections of the same member, either in the fabrication shop or in the field. This type of connection can be made using rivets, bolts, or welds. Bolted splices are common in multi-beam superstructures due to the limited allowable shipping lengths (see Figure P.2.24). Welded flange splices are common in large welded plate girders as a means of fabricating the most economical section.

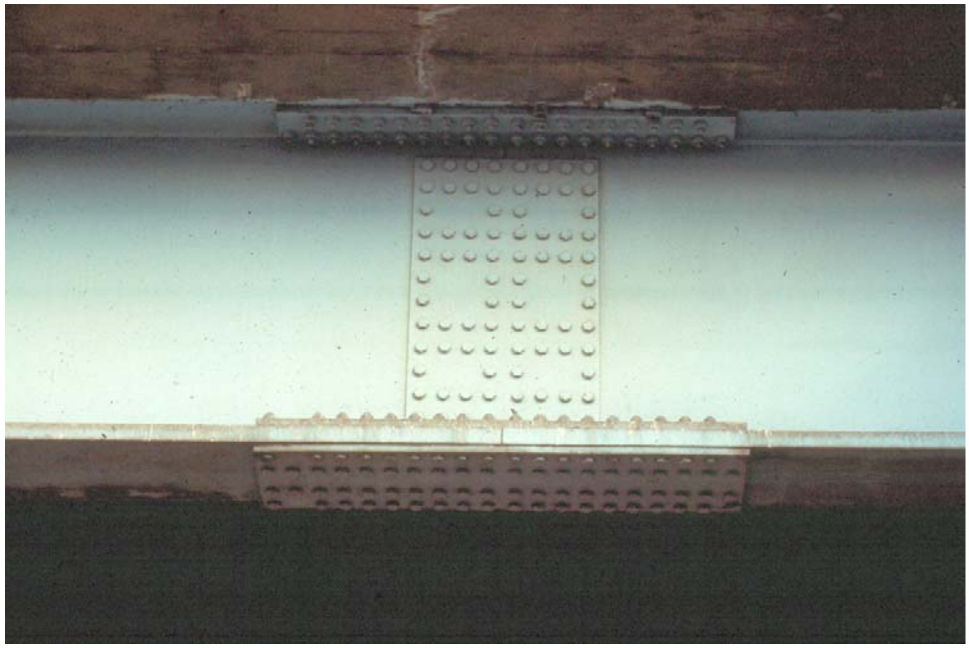


Figure P.2.24 Bolted Field Splice

P.2.6 Decks

The deck is that component of a bridge to which the live load is directly applied. Refer to Section 5 for a detailed explanation on the inspection and evaluation of decks.

Deck Purpose

The purpose of the deck is to provide a smooth and safe riding surface for the traffic utilizing the bridge (see Figure P.2.25).

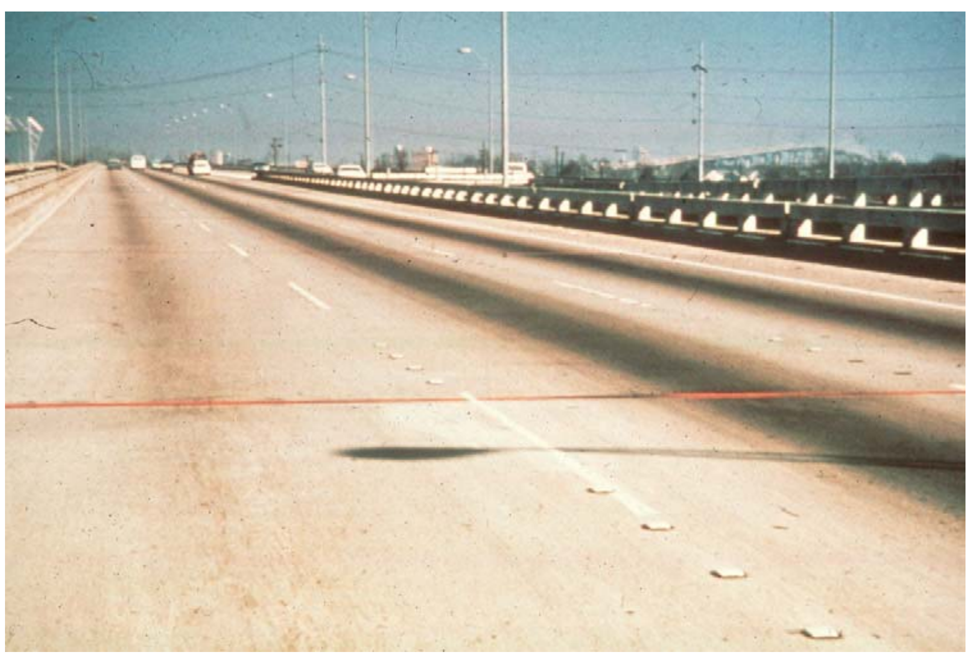


Figure P.2.25 Bridge Deck with a Smooth Riding Surface

Deck Function

The function of the deck is to transfer the live load and dead load of the deck to other bridge components. In most bridges, the deck distributes the live load to the superstructure through a floor system. However, on some bridges (e.g., a concrete slab bridge), the deck and superstructure are one unit which distributes the live load directly to the bridge supports (see Figure P.2.26).

Figure P.2.26 Underside View of a Bridge Deck

Decks function in one of two ways:

- Composite decks - act together with their supporting members and increase superstructure strength (see Figures P.2.27 and P.2.28)

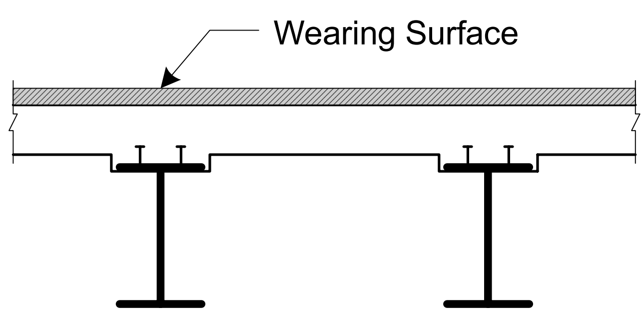


Figure P.2.27 Composite Deck and Steel Superstructure

Figure P.2.28 Shear Studs on Top Flange of Girder before Concrete Deck is Poured

- Non-composite decks - are not integral with their supporting members and do not contribute to structural capacity

Deck Materials

There are three common materials used in the construction of bridge decks:

- Timber
- Concrete
- Steel

Timber Decks

Timber decks are normally referred to as decking or timber flooring, and the term is limited to the roadway portion which receives vehicular loads. Refer to Topic 5.1 for a detailed explanation on the inspection and evaluation of timber decks.

Five basic types of timber decks are:

- Plank deck (see Figure P.2.29)
- Nailed laminated deck
- Glued-laminated deck planks
- Stressed-laminated decks
- Structural composite lumber decks

Figure P.2.29 Plank Deck

Concrete Decks

Concrete permits casting in various shapes and sizes and has provided the bridge designer and the bridge builder with a variety of construction methods. Because concrete is weak in tension, it is used together with reinforcement to resist the tensile stresses (see Figure P.2.30). Refer to Topic 5.2 for a detailed explanation on the inspection and evaluation of concrete decks.

There are several common types of concrete decks:

- Reinforced cast-in-place (CIP) - removable or stay-in-place forms
- Precast
- Precast prestressed deck panels with cast-in-place topping

Figure P.2.30 Concrete Deck

Steel Decks

Steel decks are decks composed of either solid steel plate or steel grids (see Figure P.2.31). Refer to Topic 5.3 for a detailed explanation on the inspection and evaluation of steel decks.

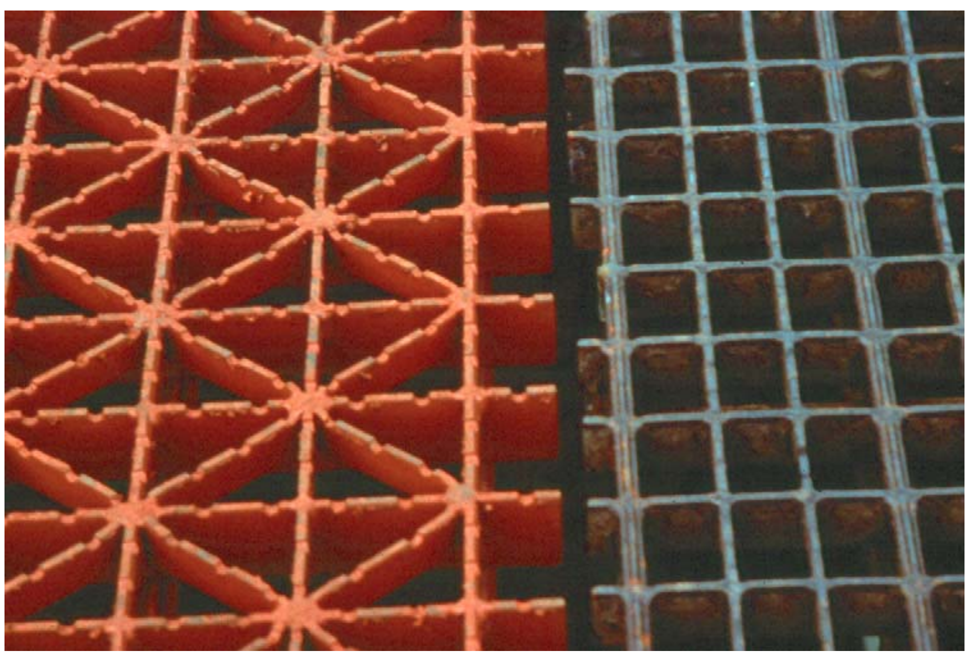


Figure P.2.31 Steel Grid Deck

There are four common types of steel decks:

- Corrugated steel flooring
- Orthotropic deck
- Grid Deck - open, filled, or partially filled
- Buckle plate deck (still exist on some older bridges but are no longer used)

Glass and Plastic

With the rise of technological development, innovative material such as glass and carbon-fiber-reinforced plastic bridge decking has begun replacing existing highway bridge decks. Though plastic bridge material is more expensive than conventional bridge materials such as concrete, it has several other advantages. These include lighter weight for efficient transport, better resistance to earthquakes, and easier installation for workers on site. Plastic and glass bridge decking is also not affected by water or de-icing salts, which corrode steel and deteriorate concrete. In addition, plastic and glass bridges will last longer than concrete or steel bridges and will easily meet the new U.S. highway specifications for a 75-year life span.

Wearing Surfaces

Constant exposure to the elements makes weathering a significant cause of deck deterioration. In addition, vehicular traffic produces damaging effects on the deck surface. For these reasons, a wearing surface is often applied to the surface of the deck. The wearing surface is the topmost layer of material applied upon the deck to provide a smooth riding surface and to protect the deck from the effects of traffic and weathering.

A timber deck may have one of the following wearing surfaces:

- Timber planks
- Bituminous

Concrete decks may have wearing surfaces of:

- Concrete
- Latex modified concrete (LMC)
- Low slump dense concrete (LSDC)
- Asphalt (see Figure P.2.32)
- Epoxy overlay with broadcast aggregate

Figure P.2.32 Asphalt Wearing Surface on a Concrete Deck

Steel decks may have wearing or riding surfaces of:

- Serrated steel
- Concrete
- Asphalt

Deck Joints, Drainage, Appurtenances, Signing and Lighting

Deck Joints

The primary function of a deck joint is to accommodate the expansion, contraction, and rotation of the superstructure. The joint must also provide a smooth transition from an approach roadway to a bridge deck, or between adjoining segments of bridge deck.

There are two major categories of deck joints:

- Open joints
- Sealed joints

Open Joints

Open joints allow water and debris to pass through them. There are two types of unsealed joints:

- Formed joints
- Finger plate joints (see Figure P.2.33)

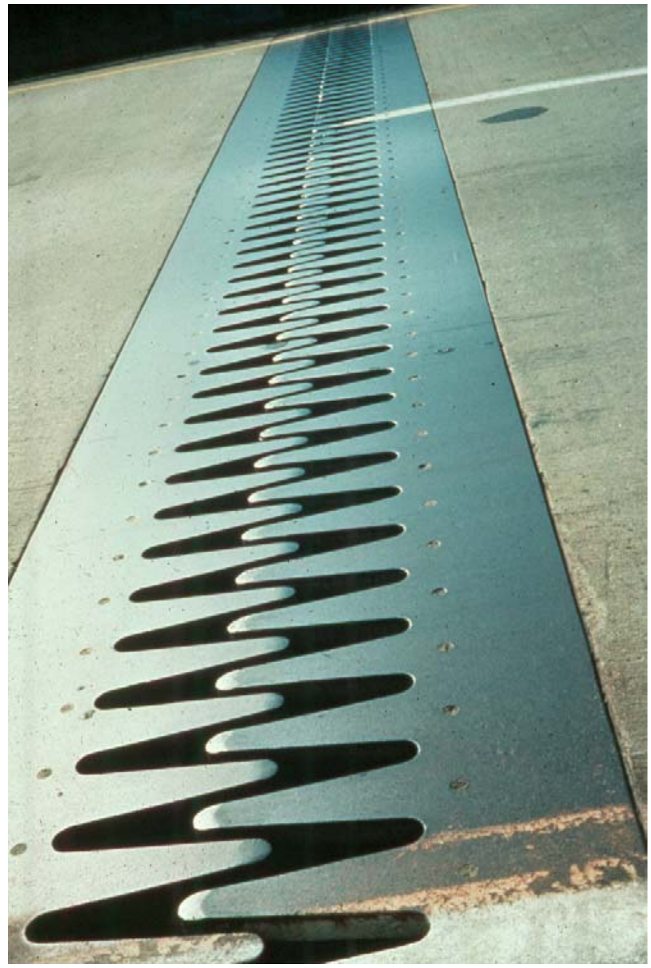


Figure P.2.33 Top View of a Finger Plate Joint

Sealed Joints

Sealed joints are designed so water and debris do not pass through them. There are six types of sealed joints:

- Poured joint seal
- Compression seal (see Figure P.2.34)
- Cellular seal (closed cell foam)
- Sliding plate joint
- Prefabricated elastomeric seal – plank, sheet, or strip seal (see Figure P.2.35)
- Modular elastomeric seal

Figure P.2.34 Top View of an Armored Compression Seal in Place

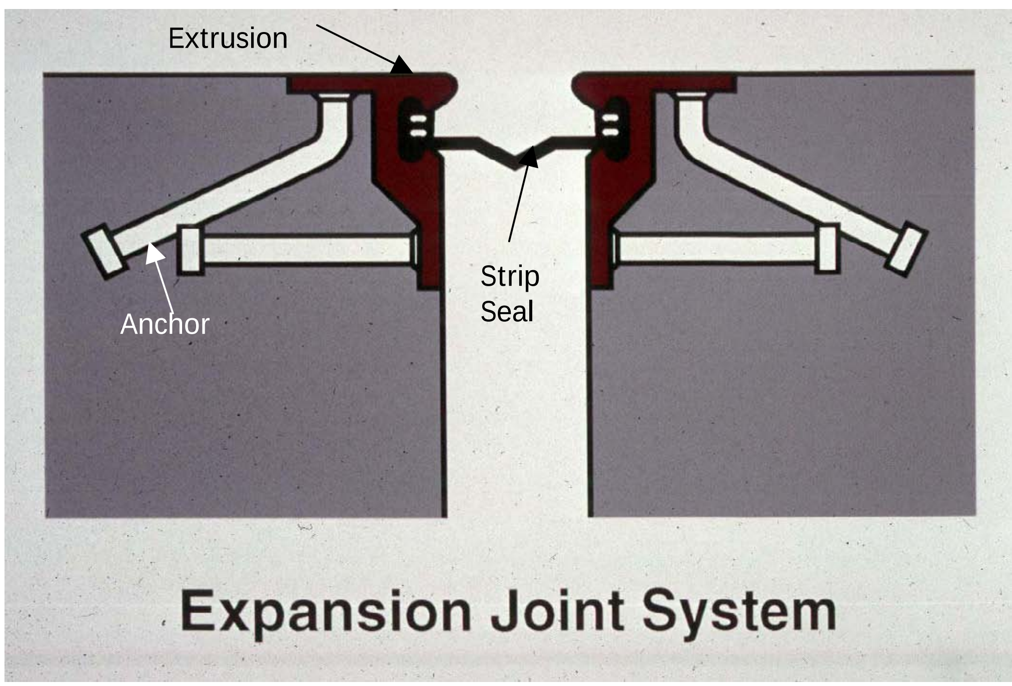


Figure P.2.35 Strip Seal

Drainage Systems

The primary function of a drainage system is to remove water from the bridge deck, from under unsealed deck joints and from behind abutments and wingwalls.

Deck Drainage System

A deck drainage system has the following components:

- Deck drains
- Outlet pipes - to lead water away from drain
- Downspouts pipes - to transport runoff to storm sewers
- Cleanout plugs - for maintenance

Joint Drainage System

A joint drainage system is typically a separate gutter or trough used to collect water passing through a finger plate or sliding plate joint.

Combining all these drainage components forms a complete deck drainage system.

Substructure Drainage Systems

Substructure drainage allows the fill material behind an abutment or wingwall to drain any accumulated water.

Substructure drainage is accomplished with weep holes or substructure drain pipes.

Deck Appurtenances

The proper and effective use of deck appurtenances minimizes hazards for traffic on the highways as well as waterways beneath the bridge.

Bridge Barriers

Bridge barriers can be broken down into two categories:

- Bridge railing - to guide, contain, and redirect errant vehicles
- Pedestrian railing - to protect pedestrians

Examples of railing include:

- Timber plank rail
- Steel angles and bars
- Pigeon hole parapet
- Combination bridge-pedestrian aluminum or steel railing
- New Jersey barrier - a very common barrier which meets current performance requirements (see Figure P.2.36)

Figure P.2.36 New Jersey Barrier

Sidewalks and Curbs

The function of sidewalks and curbs is to provide access to and maintain safety for pedestrians. Curbs serve to lessen the chance of vehicles crossing onto the sidewalk and endangering pedestrians.

Signing

Signing serves to inform the motorist about bridge or roadway conditions that may be hazardous.

Among the various types of signs likely to be encountered are:

- Regulatory
- Weight limit (see Figure P.2.37)
- Speed traffic marker
- Advisory
- Vertical clearance
- Lateral clearance
- Narrow underpass

Figure P.2.37 Weight Limit Sign

Lighting

Types of lighting that may be encountered on a bridge include the following (see Figure P.2.38):

- Highway lighting
- Traffic control lights
- Aerial obstruction lights
- Navigation lights
- Signing lights

Figure P.2.38 Bridge Lighting

Refer to Topic 5.4 for a more detailed explanation on joints, drainage, signing, and lighting of bridge decks. Refer to Topic 5.5 for a more detailed explanation on safety features and barriers of bridge decks.

P.2.7 Superstructure

Superstructure Purpose

The basic purpose of the superstructure is to carry loads from the deck across the span and to the bridge supports. The superstructure is that component of the bridge which supports the deck or riding surface of the bridge, as well as the loads applied to the deck.

Superstructure Function

The function of the superstructure is to transmit loads. Bridges are named for their type of superstructure. Superstructures may be characterized with regard to their function (i.e., how they transmit loads to the substructure). Loads may be transmitted through tension, compression, bending, or a combination of these three.

There are three common materials used in the construction of bridge superstructures:

- Timber
- Concrete
- Steel

Primary Elements

Almost all superstructures are made up of two basic elements:

Floor system - Receives traffic loads from the deck and distributes them to the main supporting elements (see Figure P.2.39)

Figure P.2.39 Floor System

Main supporting elements - Transfer all loads to the substructure units (see Figure P.2.40)

Figure P.2.40 Main Supporting Elements of Deck Arch

Secondary Elements

Secondary elements are elements which do not normally carry traffic loads directly. Typical secondary elements are:

- Diaphragms (see Figure P.2.41)
- Cross or X-bracing (see Figure P.2.42)
- Lateral bracing
- Sway-portal bracing (see Figure P.2.43)

Figure P.2.41 Diaphragms

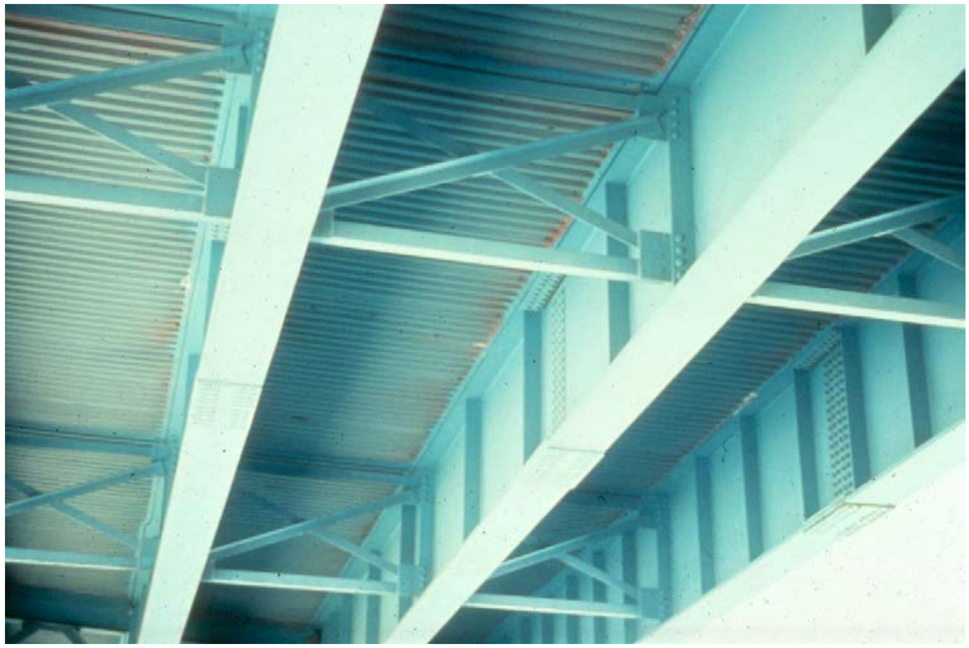


Figure P.2.42 Cross or X-Bracing

Figure P.2.43 Sway Bracing

Superstructure Types

There are three basic types of bridges (see Figure P.2.44):

- Beam
- Arch
- Cable-supported

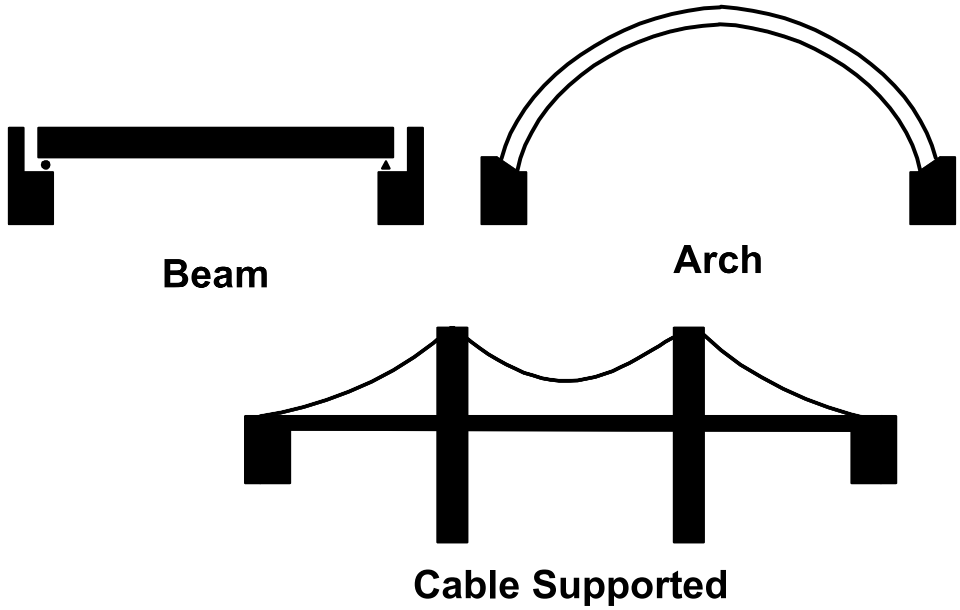


Figure P.2.44 Three Basic Bridge Types

Beam Bridges

In the case of beam bridges, loads from the superstructure are transmitted vertically to the substructure. Examples of beam bridges include:

- Slabs (concrete) (see Figure P.2.45)
- Beams (timber, concrete, or steel) (see Figures P.2.46 to P.2.50)
- Girders (concrete or steel) (see Figure P.2.51)
- Trusses (timber or steel) (see Figures P.2.52 and P.2.53)

Figure P.2.45 Slab Bridge

Figure P.2.46 Timber Beam Bridge

Figure P.2.47 Prestressed Concrete Multi-beam Bridge

Figure P.2.48 Girder Floorbeam Stringer Bridge

Figure P.2.49 Curved Girder Bridge

Figure P.2.50 Tee Beam Bridge

Figure P.2.51 Adjacent Box Beam Bridge

Figure P.2.52 Steel Box Girder Bridge

Figure P.2.53 Deck Truss Bridge

Figure P.2.54 Through Truss Bridge

Arch Bridges

In the case of arch bridges, the loads from the superstructure are transmitted diagonally to the substructure. True arches are in pure compression. Arch bridges can be constructed from timber, concrete, or steel (see Figures P.2.55 and P.2.6).

Figure P.2.55 Deck Arch Bridge

Figure P.2.56 Through Arch Bridge

Cable-Supported Bridges

In the case of cable-supported bridges, the superstructure loads are resisted by cables which act in tension. The cable forces are then resisted by the substructure anchorages and towers. Cable-supported bridges can be either suspension or cable-stayed (see Figures P.2.57 and P.2.58). Refer to Topic 12.1 for a more detailed explanation on cable-supported bridges.

Figure P.2.57 Steel Suspension Bridge

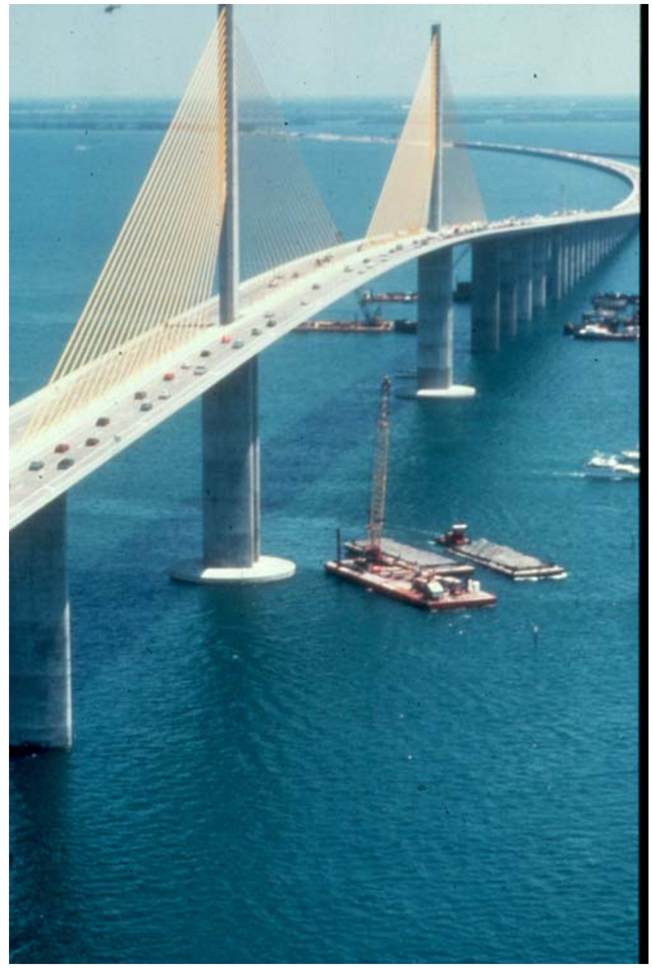


Figure P.2.58 Cable-stayed Bridge

Movable Bridge

Movable bridges are constructed across designated "Navigable Waters of the United States," in accordance with "Permit Drawings" approved by the U.S. Coast Guard. The purpose of a movable bridge is to provide the appropriate channel width and underclearance for passing water vessels when fully opened. Refer to Topic 12.2 for a more detailed explanation on movable bridges.

Movable bridges can be classified into three general groups:

- Bascule (see Figure P.2.59)
- Swing (see Figure P.2.60)
- Lift (see Figure P.2.61)

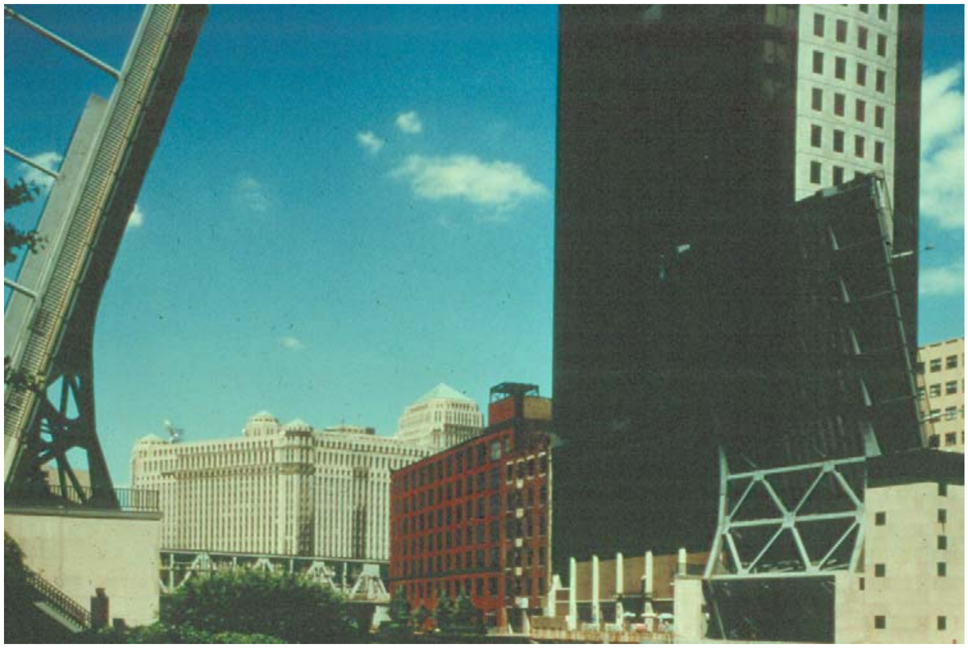


Figure P.2.59 Bascule Bridge

Figure P.2.60 Swing Bridge

Figure P.2.61 Lift Bridge

Floating Bridges

Although uncommon, some states have bridges that are not supported by a substructure. Instead, they are supported by water. The elevation of the bridge will change as the water level fluctuates.

Figure P.2.62 Floating Bridge

Culverts

A culvert is primarily a hydraulic structure, and its main purpose is to transport water flow efficiently.

Culverts are often viewed as small bridges, being constructed entirely below and independent of the roadway surface. However, culverts do not have a deck, superstructure, or substructure (see Figure P.2.63). Refer to Topics 12.3 and 12.4 for a more detailed explanation on culverts.

Figure P.2.63 Culvert

P.2.8 Bearings

Definition

A bridge bearing is a superstructure element which provides an interface between the superstructure and the substructure.

Primary Function

There are three primary functions of a bridge bearing:

- Transmit all loads from the superstructure to the substructure
- Permit longitudinal movement of the superstructure due to thermal expansion and contraction
- Allow rotation caused by dead and live load deflection

Bearings that do not allow for translation or movement of the superstructure are referred to as fixed bearings. Bearings that allow for the displacement of the structure are known as expansion bearings. Both fixed and expansion bearings permit rotation.

Basic Elements

A bridge bearing can be broken down into four basic elements (see Figure P.2.64):

- Sole plate

- Masonry plate
- Bearing or bearing surfaces
- Anchorage

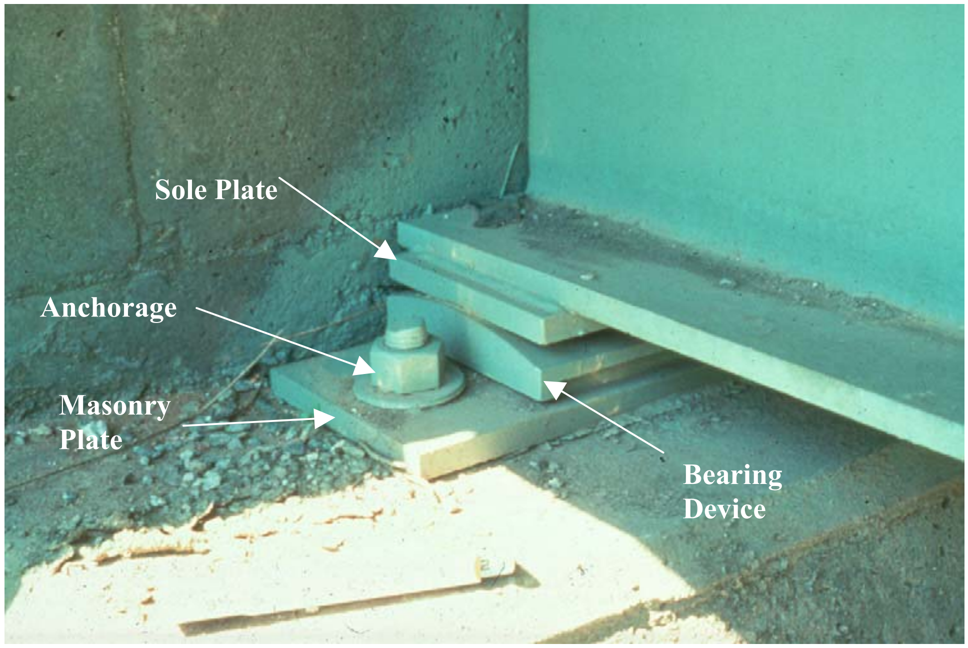


Figure P.2.64 Typical Bearing Showing Four Basic Elements

Bearing Types

Various bearing types have evolved out of the need to accommodate superstructure movement:

- Sliding plate bearings
- Roller bearings
- Rocker bearings
- Pin and link bearings
- Elastomeric bearings
- Pot bearings
- Restraining bearings
- Isolation bearings

Refer to Section 9.1 for a more detailed explanation on bridge bearings.

P.2.9 Substructure

The substructure is that component of a bridge which includes all the elements which support the superstructure.

Substructure Purposes

The purpose of the substructure is to transfer the loads from the superstructure to the foundation soil or rock. Typically the substructure includes all elements below the bearings. The loads are then distributed to the earth or to supporting piles through the footing. The footing is the enlarged base of a substructure unit and is most commonly a thick concrete slab.

Substructure Function

Substructure units function as both axially-loaded and bending members. These units resist both vertical and horizontal loads applied from the superstructure.

Substructures are divided into two basic categories:

- Abutments
- Piers and bents

Abutments provide support for the ends of the superstructure and retain the approach embankment (see Figure P.2.65). Piers and bents provide support for the superstructure at intermediate points along the bridge spans with a minimum obstruction to the flow of traffic or water (see Figure P.2.66).

Figure P.2.65 Concrete Abutment

Figure P.2.66 Concrete Pier

Abutments

Basic types of abutments include:

- Cantilever or full height abutment - extends from the grade line of the roadway or waterway below, to that of the road overhead (see Figure P.2.67).

- Stub, semi-stub, or shelf abutment - located within the topmost portion of the end of an embankment or slope. In the case of a stub, less of the breastwall or stem is visible than in the case of the full height abutment. Most new construction uses this type of abutment. These abutments may be required to be supported on piles (see Figure P.2.68).

- Spill-through or open abutment - consists of columns and has no solid wall, but rather is open to the embankment material. The approach embankment material is usually rock (see Figure P.2.69).

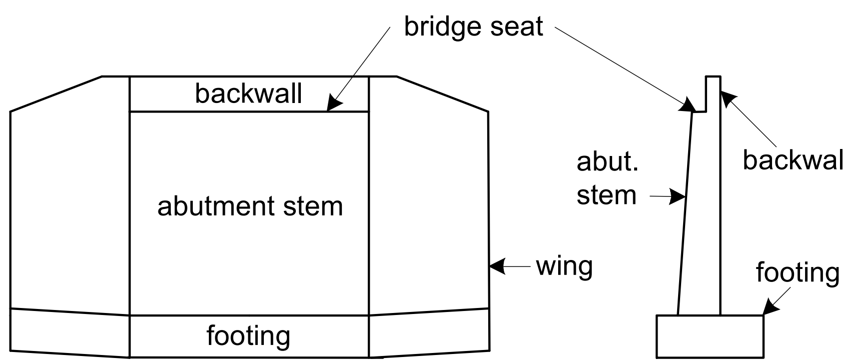


Figure P.2.67 Cantilever Abutment (or Full Height Abutment)

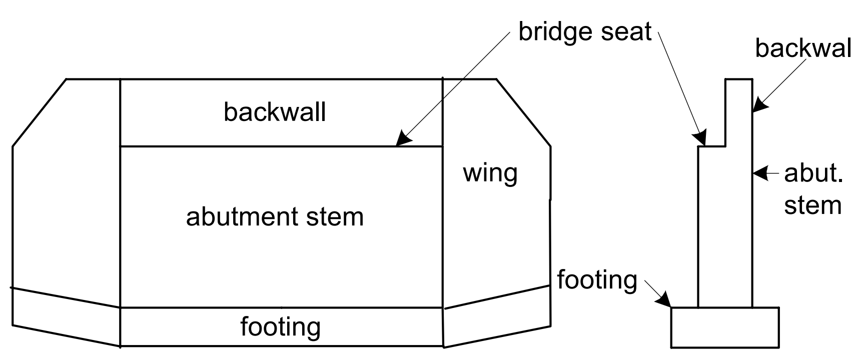


Figure P.2.68 Stub Abutment

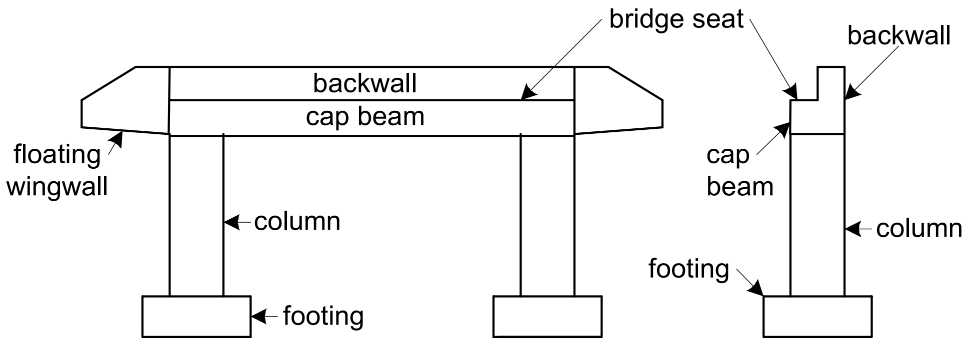


Figure P.2.69 Open Abutment

Refer to Topic 10.1 for a more detailed explanation on bridge abutments.

Piers and Bents

A pier has only one footing at each substructure unit (the footing may serve as a pile cap). A bent has several footings or no footing, as is the case with a pile bent. Refer to Topic 10.2 for a more detailed explanation on bridge piers and bents.

There are four basic types of piers:

- Solid shaft pier (see Figure P.2.70)
- Column pier (see Figure P.2.71)
- Column pier with web wall (see Figure P.2.72)
- Cantilever or hammerhead pier (see Figure P.2.73)

Figure P.2.70 Solid Shaft Pier

Figure P.2.71 Column Pier

Figure P.2.72 Column Pier with Web Wall

Figure P.2.73 Cantilever or Hammerhead Pier

There are two basic types of bents:

- Column bent (see Figure P.2.74)
- Pile bent (see Figure P.2.75)

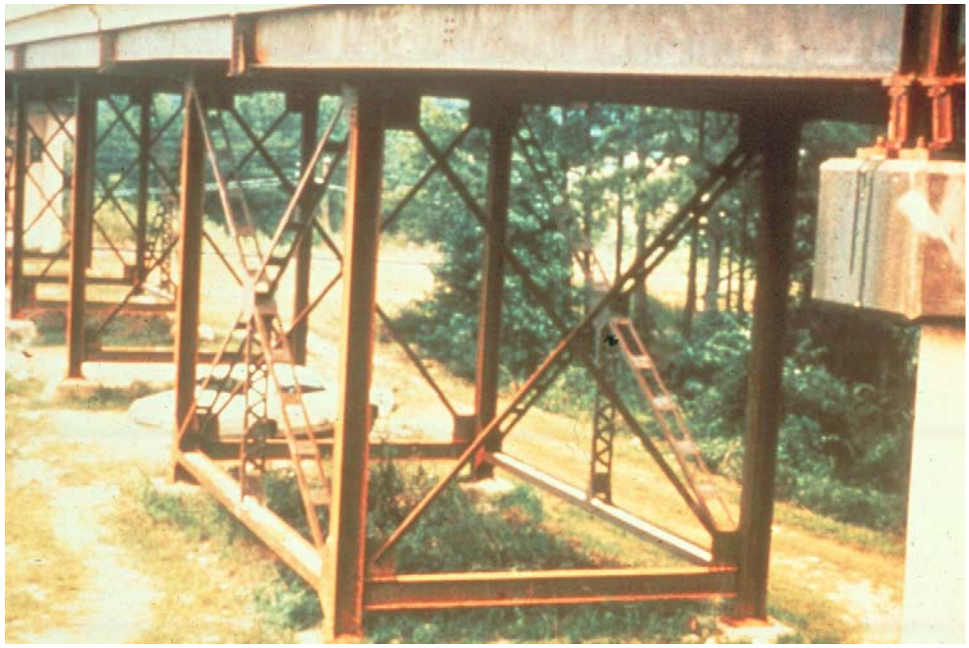


Figure P.2.74 Column Bent

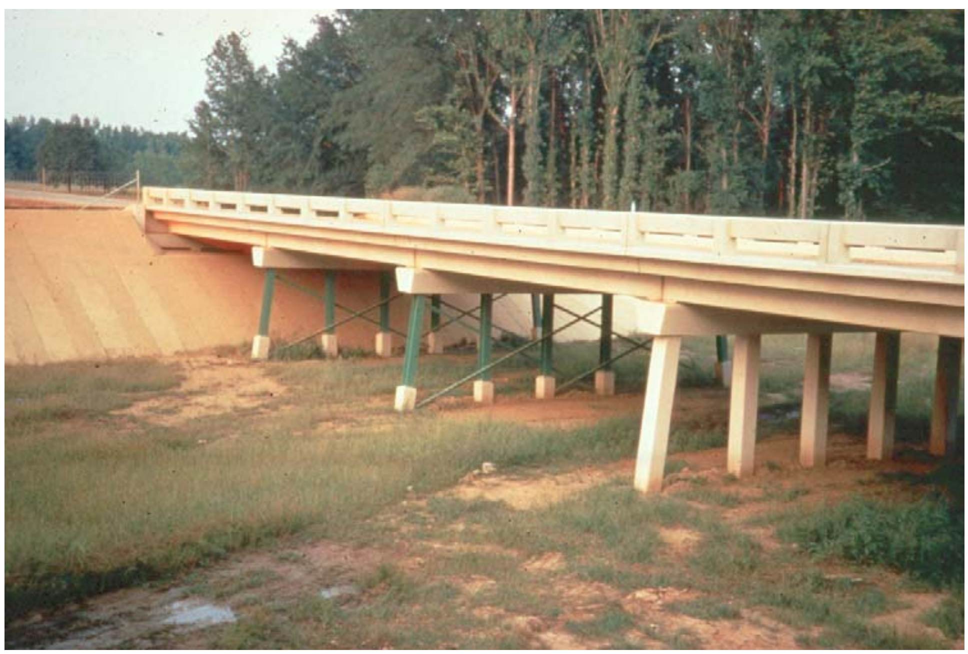


Figure P.2.75 Pile Bent

This page intentionally left blank

Table of Contents

Basic Concepts Primer

Topic P.3 Culvert Characteristics

P.3.1 Introduction

A culvert is a structure designed hydraulically to take advantage of submergence to increase hydraulic capacity. Culverts, as distinguished from bridges, are usually covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Culverts may qualify to be considered “bridge” length.

Over the years, culverts have traditionally received less attention than bridges. Since culverts are less visible it is easy to put them out of mind, particularly when they are performing adequately. Additionally, a culvert usually represents a significantly smaller investment than a bridge and in the event of a failure usually represents much less of a safety hazard.

Since 1967 there has been an increased emphasis on bridge safety and on bridge rehabilitation and replacement programs. In many cases small bridges have been replaced with multiple barrel culverts, box culverts, or long span culverts (see Figure P.3.1). There have also been recent advances in culvert design and analysis techniques. Long span corrugated metal culverts with spans in excess of 12.2 m (40 feet) were introduced in the late 1960's.

Figure P.3.1 Culvert Structure

As a result of these developments, the number, size, complexity, and cost of culvert installations have increased. The failure of a culvert may be more than a mere driving inconvenience. Failure of a major culvert may be both costly and hazardous.

Like bridges, culverts should be inspected regularly to identify potential safety problems and maintenance needs or other actions required to preserve the investment in the structure and to minimize property damage due to improper hydraulic functioning.

Purpose of Culvert Inspection

The National Bridge Inspection Program was designed to insure the safe passage of vehicles and other traffic. The inspection program provides a uniform database from which nationwide statistics on the structural and functional safety of bridges and large culvert-type structures are derived. Although these bridge inspections are essentially for safety purposes, the data collected is also used to develop rehabilitation and replacement priorities.

Bridges with spans over 6.1 m (20 ft) in length are inspected on a two-year cycle in accordance with the National Bridge Inspection Standards (NBIS). According to the American Association of State Highway and Transportation Officials (AASHTO) the definition of bridges includes culverts with openings measuring more than 6.1 m (20 ft) along the centerline of the road and also includes multiple pipes where the distance between openings is less than half of the pipe opening.

Multiple barrel culvert installations with relatively small pipes can therefore meet the definition of a bridge. Structures included in the NBIS are evaluated by utilizing a standardized inventory appraisal process that is based on rating certain structural and functional features. The data obtained is recorded on standardized inspection forms. The minimum data required for bridge length culverts is shown on the Structure Inventory and Appraisal Sheet (SI&A). Procedures for coding these items are provided in the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide)

While the importance of the NBIS inspection program cannot be overemphasized, the SI&A data sheets are oriented toward bridges rather than culverts; thus, they do not allow an inspector to collect either detailed condition data or maintenance data. Additionally, the NBIS program does not specifically address structures where the total opening length is less than 6.1 m (20 feet). However, some type of formal inventory and inspection is needed for culverts that are not bridge length. In many cases, the failure of a culvert or other structure with openings less than 6.1 m (20 ft) long can present a life threatening hazard. Although the primary purpose of this, and other sections relating to culverts is to provide inspection guidelines for culverts included in the NBIS program, the guidelines should also be generally applicable to culverts with openings which are less than 6.1 m (20 feet) long. For culverts (and bridges) less than 6.1 m (20 ft) in length, the state in which the structure is located must incorporate it into their “Local Bridge Inventory”. In this case, the state defines the minimum structure opening to be included in the “Local Bridge Inventory”.

Ideally, all culverts should be inventoried and periodically inspected. Some limitations may be necessary because a considerable effort is required to establish a current and complete culvert inventory. Small culverts may not warrant the same rigorous level of inspection as large culverts. Each agency should define its culvert inspection program in terms of inspection frequency, size, and type of culverts to be inventoried and inspected, and the information to be collected. Culverts larger than 6.1 m (20 ft) must be inspected every two years under the NBIS program. If possible all culverts should be inventoried and inspected to establish a structural adequacy and to evaluate the potential for roadway overtopping or flooding.

The types and amount of condition information to be collected should be based on the purpose for which the information will be used. For example, if small pipes are not repaired but are replaced after failures occur, then the periodic collection of detailed condition data may not be warranted. Documentation of failures as well as the causes of failures, may be all the condition data that is needed. However, the inventory should be updated whenever a replacement is accomplished.

Safety

Safety is the most important reason why culverts should be inspected. To insure that a culvert is functioning safely, the inspector should evaluate structural integrity, hydraulic performance, and roadside compatibility.

- Structural Integrity - The failure of major culverts can present a life threatening safety hazard. The identification of potential structural and material problems requires a careful evaluation of indirect evidence of structural distress as well as actual deterioration and distress in the culvert material.

- Hydraulic Performance - When a culvert's hydraulic performance is inadequate, potential safety hazards may result. The flooding of adjacent properties from unexpected headwater depth may occur. Downstream areas may be flooded by failure of the embankment. The roadway embankment or culvert may be damaged because of erosion.

- Roadside Compatibility - Many culverts, like older bridges, present roadside hazards. Headwalls and wingwalls higher than the road or embankment surface may constitute a fixed obstacle hazard. Abrupt drop-offs over the end of a culvert or steep embankments may represent rollover hazards to vehicles that leave the roadway.

- Hazards of Culvert Inspection – Discussed in Topic 3.2, Safety Practices.

Maintenance Needs

Lack of maintenance is a prime cause of improper functioning in culverts and other drainage structures. Regular periodic inspections allow minor problems to be spotted and corrected before they become serious.

Objectives

The primary objective of this topic as well as Topics 3.1, 3.2, 4.2, 4.3, 7.5, 7.12, 11.2, 12.3, and 12.4 is to provide information that will enable bridge inspectors to do the following tasks:

- Properly inspect an existing culvert.
- Evaluate structural adequacy.
- Evaluate hydraulic adequacy and recognize potential flood hazards.
- Rate the condition of the culvert.
- Correctly document and rate the findings of a culvert inspection using the appropriate coding items.
- Recognize and document traffic safety conditions.
- Recommend corrective actions/maintenance needs.

To meet the primary objective, the sections provide general procedures for conducting, reporting, and documenting a culvert inspection, and guidelines for inspecting and rating specific hydraulic and structural culvert components.

A second objective of these sections is to provide users with the information necessary to understand and evaluate the significance of defects found during an inspection of an existing culvert. To meet this objective, a review of how culverts should function structurally and hydraulically is provided briefly in this section, and covered in more detail in Topics 7.12, 12.3, and 12.4. Durability concepts are also reviewed.

P.3.2 Differentiation Between Culverts and Bridges

Traditional definitions of culverts are based on the span length rather than function or structure type. For example, part of the culvert definition included in the Bridge Inspector's Training Manual 70 states:

"...structures over 20 feet in span parallel to the roadway are usually called bridges; and structures less than 20 feet in span are called culverts even though they support traffic loads directly."

Many structures that measure more than 6.1 m (20 feet) along the centerline of the roadway have been designed hydraulically and structurally as culverts. The structural and hydraulic design of culverts is substantially different from bridges, as are construction methods, maintenance requirements, and inspection procedures. A few of the more significant differences between bridges and culverts are:

Hydraulic

Culverts are usually designed to operate at peak flows with a submerged inlet to improve hydraulic efficiency. The culvert constricts the flow of the stream to cause ponding at the upstream or inlet end. The resulting rise in elevation of the water surface produces a head at the inlet that increases the hydraulic capacity of the culvert. Bridges may constrict flow to increase hydraulic efficiency or be designed to permit water to flow over the bridge or approach roadways during peak flows. However, bridges are generally not designed to take advantage of inlet submergence to the degree that is commonly used for culverts. The effects of localized flooding on appurtenant structures, embankments, and abutting properties are important considerations in the design and inspection of culverts.

Structural

Culverts are usually covered by embankment material. Culverts must be designed to support the dead load of the soil over the culvert as well as live loads of traffic. Either live loads or dead loads may be the most significant load element depending on the type of culvert, type and depth of cover, and amount of live load. However, live loads on culverts are generally not as significant as the dead load unless the cover is shallow. Box culverts with shallow cover are examples of the type of installation where live loads may be significant.

In most culvert designs the soil or embankment material surrounding the culvert plays an important structural role. Lateral soil pressures enhance the culverts ability to support vertical loads. The stability of the surrounding soil is important to the structural performance of most culverts.

Maintenance

Because culverts usually constrict flow there is an increased potential for waterway blockage by debris and sediment, especially for culverts subject to seasonal flow. Multiple barrel culverts may also be particularly susceptible to debris accumulation. Scour caused by high outlet velocity and turbulence at inlet end is a concern. As a result of these factors, routine maintenance for culverts primarily involves the removal of obstructions and the repair of erosion and scour. Prevention of joint leakage may be critical in culverts bedded in pipeable soils to prevent undermining and loss of support.

Traffic Safety

A significant safety advantage of many culverts is the elimination of bridge parapets and railings. Culverts can usually be extended so that the standard roadway cross section can be carried over the culvert to provide a vehicle recovery area. However, when ends are located near traffic lanes or adjacent to shoulders, guardrails may be used to protect the traffic. Another safety advantage of culverts is that less differential icing occurs. Differential icing is the tendency of water on the bridge deck to freeze prior to water on the approaching roadway. Since culverts are under fill material and do not have a bridge deck, the temperature of the roadway over the culvert is at or near the temperature of the roadway approaching the culvert.

Construction

Careful attention to construction details such as bedding, compaction, and trench width during installation is important to the structural integrity of the culvert. Poor compaction or poor quality backfill around culverts may result in uneven settlement over the culvert and possibly structural distress of the culvert.

Durability

Durability of material is a significant problem in culverts and other drainage structures. In very hostile environments such as acid mine drainage and chemical discharge, corrosion and abrasion can cause deterioration of all commonly available culvert materials.

Inspection

The inspection and assessment of the structural condition of culverts requires an evaluation of not only actual distress but circumstantial evidence such as roadway settlement, pavement patches, and embankment condition.

P.3.3

Structural Characteristics of Culverts

Loads on Culverts

In addition to their hydraulic functions, culverts must also support the weight of the embankment or fill covering the culvert and any load on the embankment. There are two general types of loads that must be carried by culverts: dead loads and live loads.

Dead Loads

Dead loads include the earth load or weight of the soil over the culvert and any added surcharge loads such as buildings or additional earth fill placed over an existing culvert. If the actual weight of earth is not known, 1922 kilograms per cubic meter (120 pounds per cubic foot) is generally assumed.

Live Loads

The live loads on a culvert include the loads and forces, which act upon the culvert due to vehicular or pedestrian traffic. The highway wheel loads generally used for analysis are shown in Figure P.3.2. The effect of live loads decreases as the height of cover over the culvert increases. When the cover is more than two feet, concentrated loads may be considered as being spread uniformly over a square with sides 1.75 times the depth of cover. This concept is illustrated in Figure P.3.3 and P.3.4. In fact, for single spans, if the height of earth fill is more than 2.4 meters (8 feet) and exceeds the span length, the effects of live loads can be ignored all together. (see AASHTO)

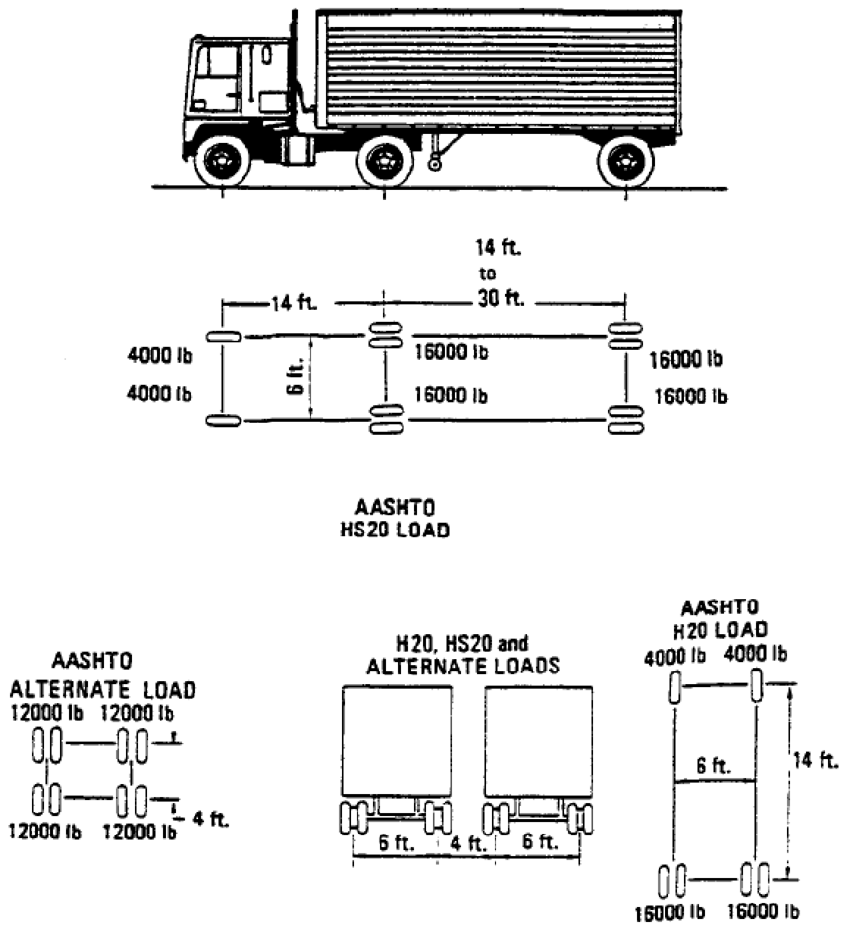


Figure P.3.2 AASHTO Live Load Spacing for Highway Structures

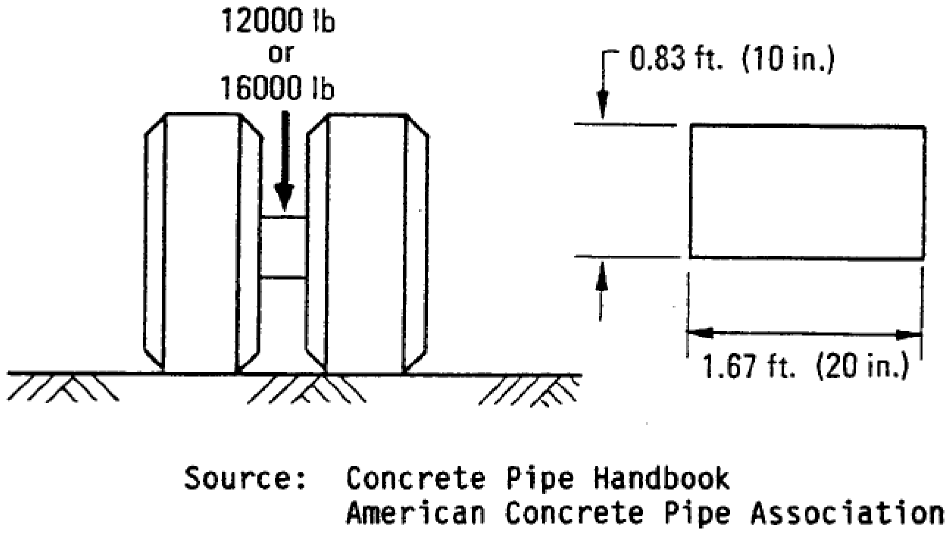


Figure P.3.3 Surface Contact Area for Single Dual Wheel

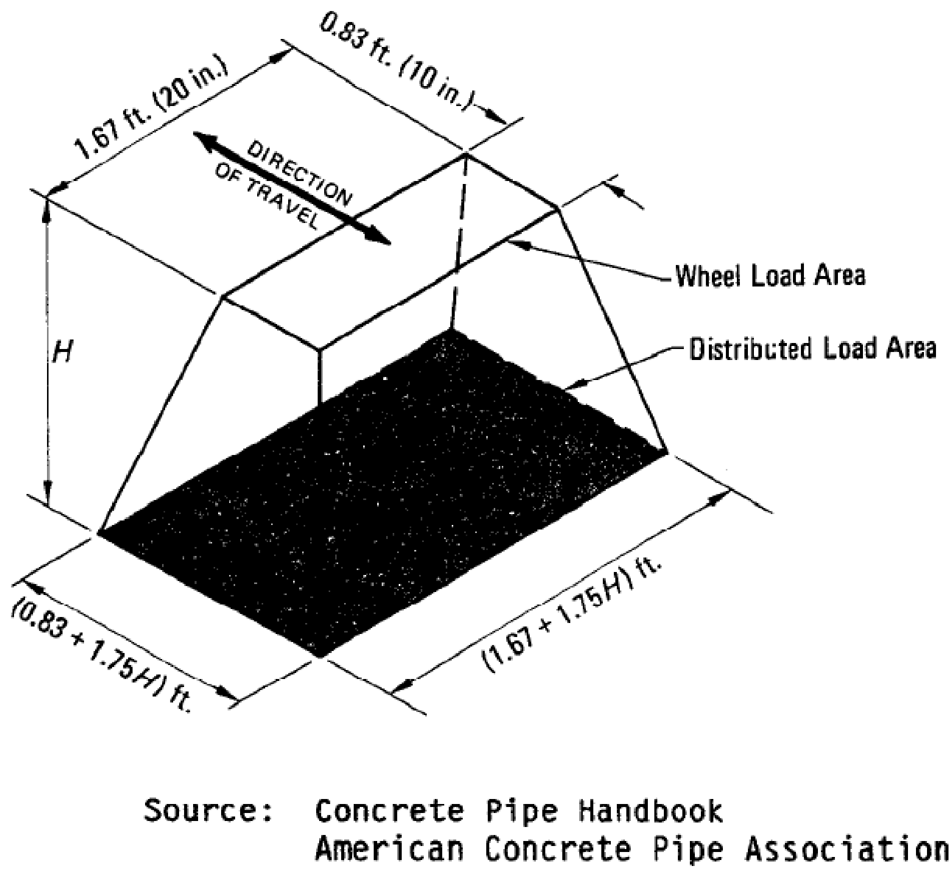


Figure P.3.4 Distribution of Live Load (Single Dual Wheel) for Depth of Cover

Categories of Structural Materials

Based upon material type, culverts can be divided into two broad structural categories: rigid and flexible.

- Rigid Culverts

Culverts made from materials such as reinforced concrete and stone masonry are very stiff and do not deflect appreciably. The culvert material itself provides the needed stiffness to resist loads. In doing this, zones of tension and compression are created. The culvert material is designed to resist the corresponding stresses.

Rigid Culverts are discussed in detail in Sections 7.5, 7.12, and 12.3.

- Flexible culverts

Flexible culverts are commonly made from steel or aluminum. In some states composite materials are used. As stated earlier, flexible culverts rely on the surrounding backfill material to maintain their structural shape. Since they are flexible, they can be deformed significantly with no cracks occurring.

As vertical loads are applied, a flexible culvert will deflect if the surrounding fill material is loose. The vertical diameter decreases while the horizontal diameter increases. Soil pressures resist the increase in horizontal diameter.

For flexible culverts with large openings, sometimes longitudinal and/or circumferential stiffeners are used to prevent excessive deflection. Circumferential stiffeners are usually metal ribs bolted around the circumference of the culvert. Longitudinal stiffeners may be metal or reinforced concrete. This type of stiffener is sometimes called a thrust beam.

Construction and Installation Requirements

The structural behavior of flexible and rigid culverts is often dependent on construction practices during installation (see Figure P.3.5). Items, which require particular attention during construction, are discussed briefly in the following text. This information is provided so that the bridge inspector may gain insight on why certain structural defects are found when inspecting a culvert.

- Compaction and Side Support - Good backfill material and adequate compaction are of critical importance to flexible culverts. A well-compacted soil envelope is needed to develop the lateral pressures required to maintain the shape of flexible culverts. Well-compacted backfill is also important to the performance of rigid culverts. Poorly compacted soils do not provide the intended lateral support.

- Trench Width - Trench width can significantly affect the earth loads on rigid culverts. It is therefore important that trench widths be specified on the plans and that the specified width not be exceeded without authorization from the design engineer.

- Foundations and Bedding - A foundation capable of providing uniform and stable support is important for both flexible and rigid culverts. The foundation must be able to support the structure at the proposed grade and elevation without concentration of foundation pressures. Foundations should be relatively yielding when compared to side fill. Establishing a suitable foundation requires removal and replacement of any hard spots or soft spots. Bedding is needed to level out any irregularities in the foundation and to insure uniform support. When using flexible culverts, bedding should be shaped to a sufficient width to permit compaction of the remainder of the backfill, and enough loose material should be placed on top of the bedding to fill the corrugations. When using rigid culverts, the bedding should conform to the bedding conditions specified in the plans and should be shaped to allow compaction and to provide clearance for the bell ends on bell and spigot type rigid pipes. Adequate support is critical in rigid pipe installations, or shear stress may become a problem.

- Construction Loads - Culverts are generally designed for the loads they must carry after construction is completed. Construction loads may exceed design loads. These heavy loads can cause damage if construction equipment crosses over the culvert installation before adequate fill has been placed or moves too close to the walls, creating unbalanced loading. Additional protective fill may be needed for equipment crossing points.

- Camber - In high fills the center of the embankment tends to settle more than the areas under the embankment side slopes. In such cases it may be necessary to camber the foundation slightly. This should be accomplished by using a flat grade on the upstream half of the culvert and a steeper grade on the downstream half of the culvert. The initial grades should not cause water to pond or pocket.

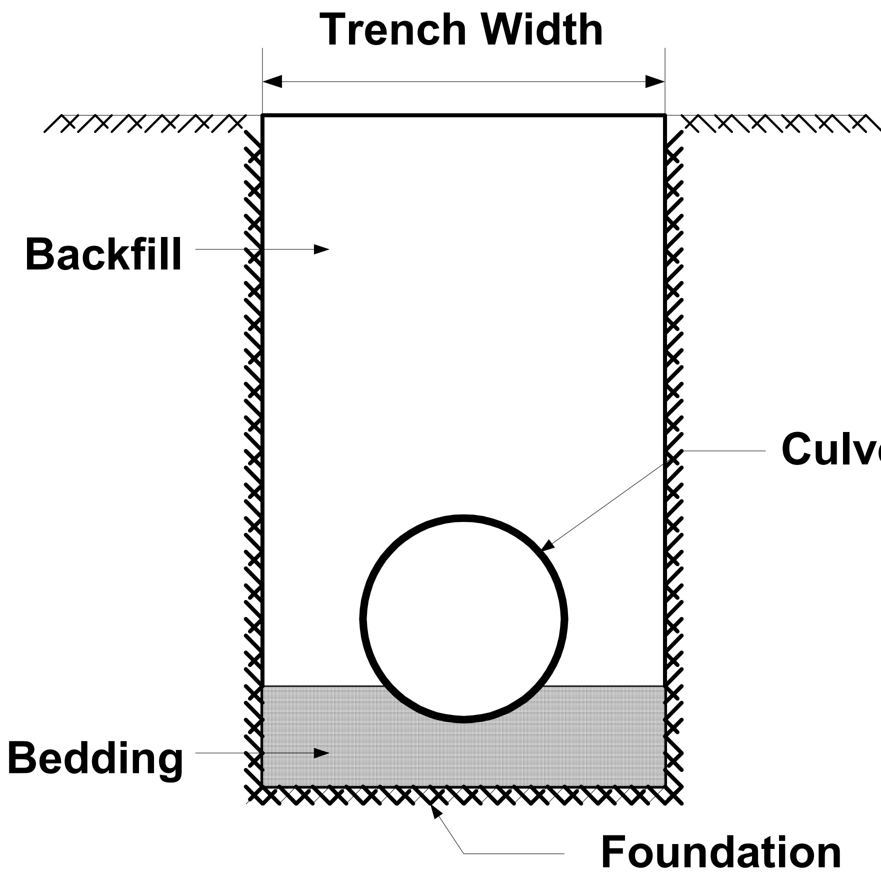


Figure P.3.5 Culvert Construction and Installation Requirements

P.3.4 Culvert Shapes

A wide variety of standard shapes and sizes are available for most culvert materials. Since equivalent openings can be provided by a number of standard shapes, the selection of shape may not be critical in terms of hydraulic performance. Shape selection is often governed by factors such as depth of cover or limited headwater elevation. In such cases a low profile shape may be needed. Other factors such as the potential for clogging by debris, the need for a natural stream bottom, or structural and hydraulic requirements may influence the selection of culvert shape. Each of the common culvert shapes are discussed in the following paragraphs.

Circular

The circular shape is the most common shape manufactured for pipe culverts (see Figure P.3.6). 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 due to the diminishing free surface as the pipe fills beyond the midpoint. With very large diameter corrugated metal pipes, the flexibility of the sidewalls dictates that special care be taken during backfill construction to maintain uniform curvature.

Figure P.3.6 Circular Culvert Structure

Pipe Arch and Elliptical Shapes

Pipe arch and elliptical shapes are often used instead of circular pipe when the distance from channel invert to pavement surface is limited or when a wider section is desirable for low flow levels (see Figure P.3.7). These shapes may also be prone to clogging as the depth of flow increases and the free surface diminishes. Pipe arch and elliptical shapes are not as structurally efficient as a circular shape.

Figure P.3.7 Pipe Arch Culvert

Arches

Arch culverts offer less of an obstruction to the waterway than pipe arches and can be used to provide a natural stream bottom where the stream bottom is naturally erosion resistant (see Figure P.3.8). Foundation conditions must be adequate to support the footings. Riprap is frequently used for scour protection.

Figure P.3.8 Arch Culvert

Box Sections

Rectangular cross-section culverts are easily adaptable to a wide range of site conditions including sites that require low profile structures. Due to the flat sides and top, rectangular shapes are not as structurally efficient as other culvert shapes (see Figure P.3.9). In addition, box sections have an integral floor.

Figure P.3.9 Concrete Box Culvert

Multiple Barrels

Multiple barrels are used to obtain adequate hydraulic capacity under low embankments or for wide waterways (see Figure P.3.10). In some locations they may be prone to clogging as the area between the barrels tends to catch debris and sediment. When a channel is artificially widened, multiple barrels placed beyond the dominant channel are subject to excessive sedimentation. The span or opening length of multiple barrel culverts includes the distance between barrels as long as that distance is less than half the opening length of the adjacent barrels.

Figure P.3.10 Multiple Cell Concrete Culvert

Frame Culverts

Frame culverts are constructed of cast-in-place (see Figure P.3.11) or precast reinforced concrete. This type of culvert has no floor (concrete bottom) and fill material is placed over the structure.

Figure P.3.11 Frame Culvert

P.3.5 Culvert Materials

Precast Concrete

Precast concrete culverts are manufactured in six standard shapes:

- Circular
- Pipe arch
- Horizontal elliptical
- Vertical elliptical
- Rectangular
- Arch

With the exception of box culverts, concrete culvert pipe is manufactured in up to five standard strength classifications. The higher the classification number, the higher the strength. Box culverts are designed for various depths of cover and live loads. All of the standard shapes are manufactured in a wide range of sizes. Circular and elliptical pipes are available with standard sizes as large as 3.7 m (144 inches) in diameter, with larger sizes available as special designs. Standard box sections are also available with spans as large as 3.7 m (144 inches). Precast concrete arches on cast-in-place footings are available with spans up to 12.8 m (42 feet). A listing of standard sizes is provided at the end of Topics 7.12, 12.3, and 12.4.

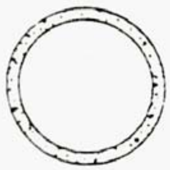
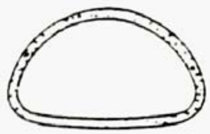
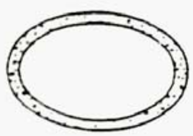
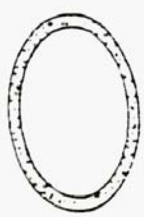
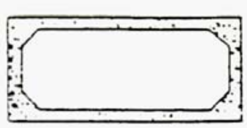
SHAPE	RANGE OF SIZES	COMMON USES
CIRCULAR 	12 to 180 inches reinforced 4 to 36 inches non-reinforced	Culverts, storm drains, and sewers.
PIPE ARCH 	15 to 132 inches equivalent diameter	Culverts, storm drains, and sewers. Used where head is limited.
HORIZONTAL ELLIPSE 	Span x Rise 18 to 144 inches equivalent diameter	Culverts, storm drains, and sewers. Used where head is limited.
VERTICAL ELLIPSE 	Span x Rise 36 to 144 inches equivalent diameter	Culverts, storm drains, and sewers. Used where lateral clearance is limited.
RECTANGULAR (box sections) 	Span 3ft to 12ft	Culverts, storm drains, and sewers. Used for wide openings with limited head.
ARCH 	Span 24 ft to 41 ft	Culvert and storm drains. For low, wide waterway enclosures.

Figure P.3.12 Standard Concrete Pipe Shapes

Cast-in-Place Concrete

Culverts that are reinforced cast-in-place concrete are typically either rectangular or arch-shaped. The rectangular shape is more common and is usually constructed with multiple cells (barrels) to accommodate longer spans. One advantage of cast-in-place construction is that the culvert can be designed to meet the specific requirements of a site. Due to the long construction time of cast-in-place culverts, precast concrete or corrugated metal culverts are sometimes selected. However, in many areas cast-in-place culverts are more practical and represent a significant number of installations.

Metal Culverts

Flexible culverts are typically either steel or aluminum and are constructed from factory-made corrugated metal pipe or field assembled from structural plates. Structural plate products are available as plate pipes, box culverts, or long span structures (see Figure 12.4.2). Several factors such as span length, vertical and horizontal clearance, peak stream flow and terrain determine which flexible culvert shape is used.

Masonry

Stone and brick are durable, low maintenance materials. Prior to the 1920's, both stone and brick were used frequently in railroad and road construction projects because they were readily available from rock cuts or local brickyards. Currently stone and brick are seldom used for constructing culvert barrels. Stone is used occasionally for this purpose in locations which have very acidic runoff, but the most common use of stone is for headwalls where a rustic or scenic appearance is desired. A stone masonry arch culvert is shown in Figure P.3.13. Refer to Topic 2.4 for a detailed discussion of stone masonry.

Figure P.3.13 Typical Stone Masonry Arch Culvert

Timber

There are a limited amount of timber culverts throughout the nation.

Timber culverts are generally box culverts and are constructed from individual timbers similar to railroad ties. Timber culverts are also analagous to a short span timber bridge on timber abutments (see Figure P.3.14).

An inspection of a timber culvert should be conducted in the same manner as a timber bridge, including sounding and drilling to determine the extent of decay. The inspector should accurately describe the construction of the timber culvert and make note of the following timber defects and their location and extent:

- Defects from Checks, Splits, and Shakes
- Decay by Fungi
- Damage by Parasites
- Damage from chemical attack
- Damage from fire
- Damage from Impact/Collisions
- Damage from Abrasion/Wear
- Damage from Overstress
- Damage from Weathering/Warping

Refer to Topic 2.1.5 for a more detailed presentation of the types and causes of timber deterioration.

Bulging of the walls and any shape deformations may indicate unstable soil conditions. These problems and their location and extent should be recorded.

The vast majority of these culverts do not have floors. The inspector should check carefully at the footings for any scour or undermining. A probing rod should be used since scour holes can and do fill up with sediment.

Figure P.3.14 Timber Box

Other Materials

Aluminum, steel, concrete, and stone masonry are the most commonly found materials for existing culverts. There are several other materials which may be encountered during culvert inspections, including cast iron, stainless steel, terra cotta, asbestos cement, and plastic. These materials are not commonly found because they are either relatively new (plastic), labor intensive (terra cotta), or used for specialized situations (stainless steel and cast iron).

P.3.6 Culvert End Treatments

Culverts may have end treatments or end structures. End structures are used to control scour, support backfill, retain the embankment, improve hydraulic efficiency, protect the culvert barrel, and provide additional stability to the culvert ends.

The most common types of end treatments are:

- Projecting - The barrel simply extends beyond the embankment. No additional support is used (see Figure P.3.15).

- Mitered - The end of the culvert is cut to match the slope of the embankment. This type of treatment is also referred to as beveling and is commonly used when the embankment has some sort of slope paving (see Figure P.3.16).

- Skewed - Culverts, which are not perpendicular to the roadway, may have their ends cut parallel to the roadway (see Figure P.3.17).

- Pipe end section - A section of pipe is added to the ends of the culvert barrel. These are typically used on relatively smaller culverts.

- Headwalls - Used along with wingwalls to retain the fill, resist scour, and improve the hydraulic capacity of the culvert. Headwalls are usually reinforced concrete (see Figure P.3.18), but can be constructed of timber or masonry. Metal headwalls are usually found on metal box culverts.

Figure P.3.15 Culvert End Projection

Figure P.3.16 Culvert Mitered End

Figure P.3.17 Culvert Skewed End

Figure P.3.18 Culvert Headwall

Miscellaneous Appurtenance Structures may also be used with end treatments to improve hydraulic efficiency and reduce scour. Typical appurtenances are:

- Aprons - Used to reduce streambed scour at the inlets and outlets of culverts (see Figure P.3.19). Aprons are typically concrete slabs, but they may also be riprap. Most aprons include an upstream cutoff wall to protect against undermining.

- Energy Dissipators - Used when outlet velocities are likely to cause streambed scour downstream from the culvert. Stilling basins, riprap or other devices that reduce flow velocity can be considered energy dissipators (see Figure P.3.20).

Figure P.3.19 Apron

Figure P.3.20 Energy Dissipator

P.3.7 Hydraulics of Culverts

Culverts are primarily constructed to convey water under a highway, railroad, or other embankment. A culvert which does not perform this function properly may jeopardize the throughway, cause excessive property damage, or even loss of life. The hydraulic requirements of a culvert usually determine the size, shape, slope, and inlet and outlet treatments. Culvert hydraulics can be divided into two general design elements:

- Hydrologic Analysis
- Hydraulic Analysis

A hydrologic analysis is the evaluation of the watershed area for a stream and is used to determine the design discharge or the amount of runoff the culvert should be designed to convey.

A hydraulic analysis is used to select a culvert, or evaluate whether an existing culvert is capable of adequately conveying the design discharge. To recognize whether a culvert is performing adequately the inspector should understand the factors that influence the amount of runoff to be handled by the culvert as well as the factors which influence the culvert's hydraulic capacity.

Hydrologic Analysis

Most culverts are designed to carry the surface runoff from a specific drainage area. While the selection and use of appropriate methods of estimating runoff requires a person experienced in hydrologic analysis and would usually not be performed by the inspector, the inspector should understand how changes in the topography of the drainage area can cause major changes in runoff. Climatic and topographic factors are briefly discussed in the following sections.

Climatic Factors

Climatic factors that may influence the amount of runoff include:

- Rainfall intensity
- Storm duration
- Rainfall distribution within the drainage area
- Soil moisture
- Snow melt
- Rain-on-snow
- Rain-hail
- Other factors

Topographic Factors

Topographic factors that may influence runoff include:

- The land use within the drainage area
- The size, shape, and slope of the drainage area
- Other factors such as the type of soil, elevation, and orientation of the area

Land use is the most likely characteristic to change significantly during the service life of a culvert. Changes in land use may have a considerable effect on the amount and type of runoff. Some surface types will permit more infiltration than other surface types. Practically all of the rain falling on paved surfaces will drain off while much less runoff will result from undeveloped land. If changes in land use were not planned during the design of a culvert, increased runoff may exceed the capacity of an existing culvert when the land use does change.

The size, shape, and slope of a culvert's drainage area influence the amount of runoff that may be collected and the speed with which it will reach the culvert. The amount of time required for water to flow to the culvert from the most remote part of a drainage area is referred to as the time of concentration. Changes within the drainage area may influence the time of concentration.

Straightening or enclosing streams and eliminating temporary storage by replacing undersized upstream pipes are examples of changes which may decrease time of concentration. Land use changes may also decrease time of concentration since water will flow more quickly over paved surfaces. Since higher rainfall intensities occur for shorter storm durations, changes in time of concentration can have a significant impact on runoff. Drainage areas are sometimes altered and flow diverted from one watershed to another.

Hydraulic Capacity

The factors affecting capacity may include headwater depth, tailwater depth, inlet geometry, the slope of the culvert barrel, and the roughness of the culvert barrel. The various combinations of the factors affecting flow can be grouped into two types of conditions in culverts:

- Inlet control
- Outlet control

Inlet Control

Under inlet control the discharge from the culvert is controlled at the entrance of the culvert by headwater depth and inlet geometry (see Figure P.3.21). Inlet geometry includes the cross-sectional area, shape, and type of inlet edge. Inlet control governs the discharge as long as water can flow out of the culvert faster than it can enter the culvert.

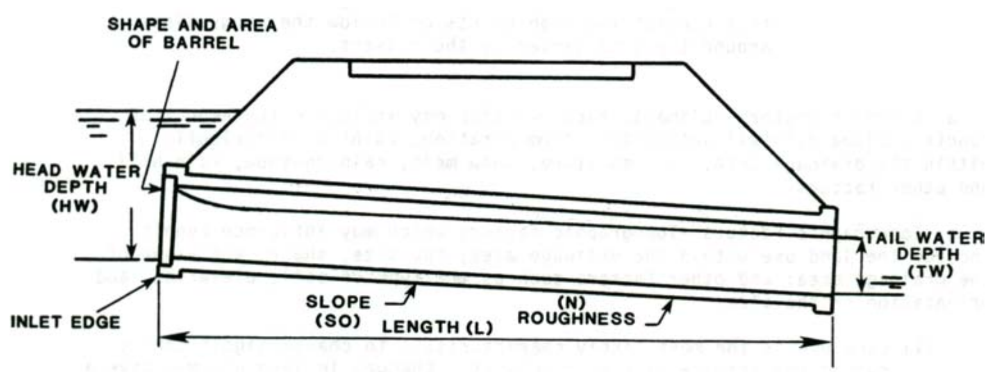


Figure P.3.21 Factors Affecting Culvert Discharge (Source: Adapted from Concrete Pipe Handbook, American Concrete Pipe Association)

Most culverts, except those in flat terrain, are designed to operate under inlet control during peak flows. Since the entrance characteristics govern, minor modifications at the culvert inlet can significantly effect hydraulic capacity. For example, change in the approach alignment of the stream may reduce capacity, while the improvement of the inlet edge condition, or addition of properly designed headwalls and wingwalls, may increase the capacity.

Outlet Control

Under outlet control water can enter the culvert faster than water can flow through the culvert. The discharge is influenced by the same factors as inlet control plus the tailwater depth and barrel characteristics (slope, length, and roughness). Culverts operating with outlet control usually lie on flat slopes or have high tailwater.

When culverts are operating with outlet control, changes in barrel characteristics or tailwater depth may effect capacity. For example, increased tailwater depth or debris in the culvert barrel may reduce the capacity.

Special Hydraulic Considerations

Inlet and Outlet Protection

The inlets and outlets of culverts may require protection to withstand the hydraulic forces exerted during peak flows. Inlet ends of flexible pipe culverts, which are not adequately protected or anchored, may be subject to entrance failures due to buoyant forces. The outlet may require energy dissipators to control erosion and scour and to protect downstream properties. High outlet velocities may cause scour which undermines the endwall, wingwalls, and culvert barrel. This erosion can cause end-section drop-off in rigid sectional pipe culverts.

Protection Against Piping

Seepage along the outside of the culvert barrel may remove supporting material. This process is referred to as “piping”, since a hollow cavity similar to a pipe is often formed. Piping can also occur through open joints. Piping is controlled by reducing the amount and velocity of water seeping along the outside of the culvert barrel. This may require watertight joints and in some cases anti-seep collars. Good backfill material and adequate compaction of that material are also important.

P.3.8

Factors Affecting Culvert Performance

Some of the common factors that can affect the performance of a culvert include the following:

- Construction Techniques - Specifically, how well the foundation was prepared, the bedding placed, and the backfill compacted.
- The characteristics of the stream flow - water depth, velocity, turbulence.
- Structural Integrity - how well the structure can withstand the loads to which it is subjected, especially after experiencing substantial deterioration and section loss.
- Suitability of the Foundation - Can the foundation material provide

adequate support?

- Stability of the embankment in relationship to other structures on the upstream or downstream side.

- Hydraulic capacity - if the culvert cross section is insufficient for flow, upstream ponding could result and damage the embankment.

- The presence of vegetation - can greatly affect the means and efficiency of the flow through the culvert.

- The possibility of abrasion and corrosion caused by substances in the water, the surrounding soil or atmosphere.

P.3.9 Types and Locations of Culvert Distress

Types of Distress

The combination of high earth loads, long pipe-like structures and running water tends to produce the following types of distress:

- Shear or bending failure - High embankments may impose very high loads on all sides of a culvert and can cause shear or bending failure (see Figure P.3.22).

- Foundation failure - Either a smooth sag or differential vertical displacement at construction or expansion joints (settlement). Tipping of wingwalls. Lateral movement of precast or cast-in-place box sections (see Figure P.3.23).

- Hydraulic failure - Full flow design conditions result in accelerated scour and undermining at culvert ends as well as at any irregularities within the culvert due to foundation problems (see Figure P.3.24).

- Debris accumulation - Branches, sediment and trash can often be trapped at the culvert entrance restricting the channel flow and causing scour and embankment erosion (see Figure P.3.25).

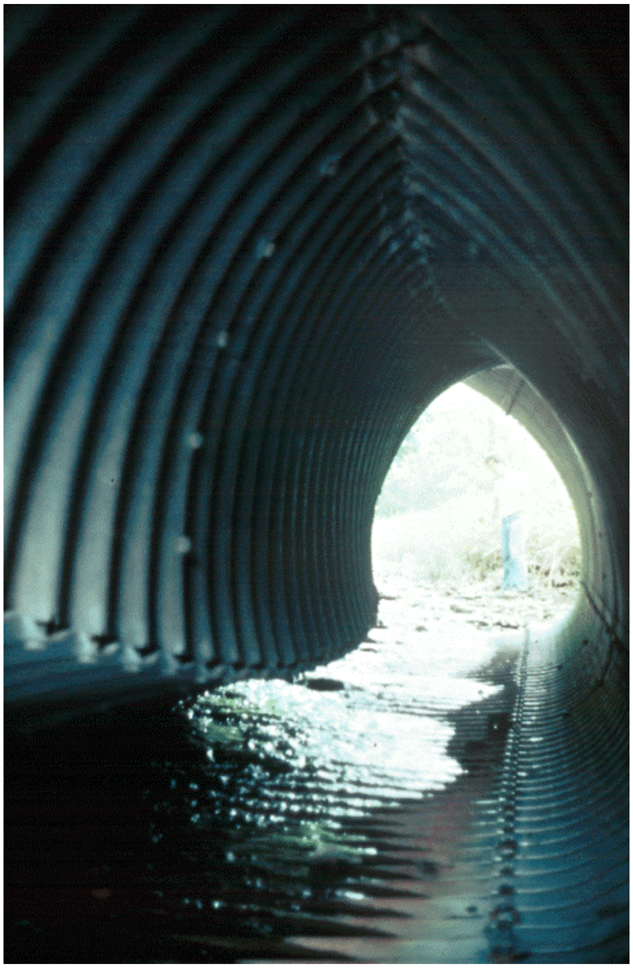


Figure P.3.22 Bending or Shear Failure

Figure P.3.23 Cracking of Culvert Due to Foundation Settlement

Figure P.3.24 Scour and Undermining at Culvert Inlet

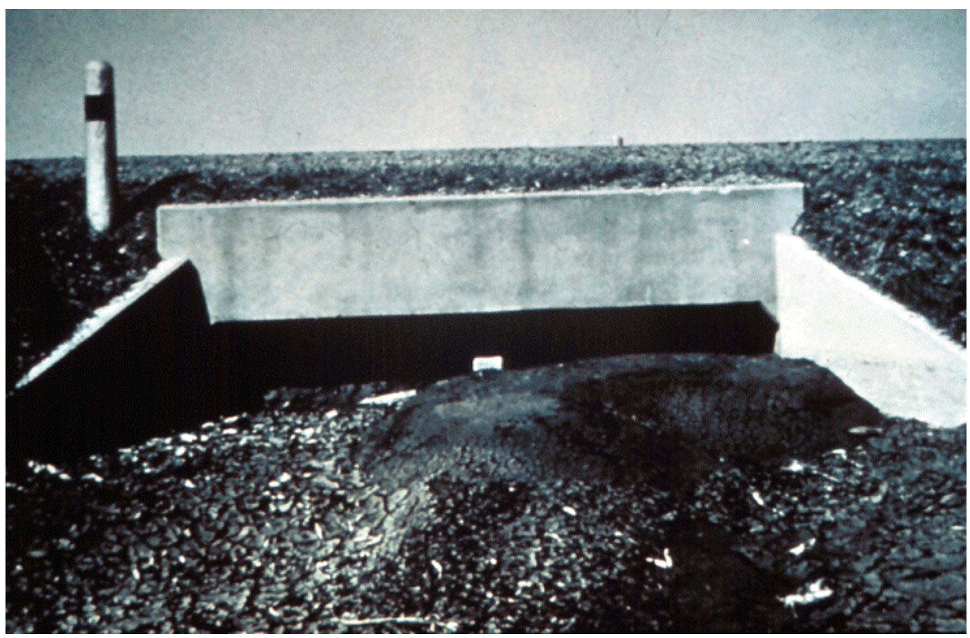


Figure P.3.25 Debris and Sediment Buildup

Inspection Locations

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, 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:

- Overall condition
- Approach roadway and embankment settlement
- Waterway (see in Topic 11.2)
- End treatments
- Appurtenance structures
- Culvert barrel

Overall Condition

General observations of the condition of the culvert should be made while approaching the culvert area. The purpose of these initial observations is to familiarize the inspector with the structure. They may also point out a need to modify the inspection sequence or indicate areas requiring special attention. The inspector should also be alert for changes in the drainage area that might affect runoff characteristics.

Approach Roadway and Embankment

Inspection of the approach roadway and embankment includes an evaluation of the functional adequacy (see Figure P.3.26).

The approach roadway and embankment should also be inspected for the following functional requirements:

- Signing
- Alignment
- Clearances
- Adequate shoulder profile
- Safety features

Figure P.3.26 Approach Roadway at a Culvert Site

Defects in the approach roadway and embankment may be indicators of possible structural or hydraulic problems in the culvert. The approach roadway and embankment should be inspected for the following conditions:

- Sag in roadway or guardrail
- Cracks in pavement
- Pavement patches or evidence that roadway has settled
- Erosion or failure of side slopes

Approach roadways should be examined for sudden dips, cracks, and sags in the pavement (see Figure P.3.27). These usually indicate excessive deflection of the culvert or inadequate compaction of the backfill material.

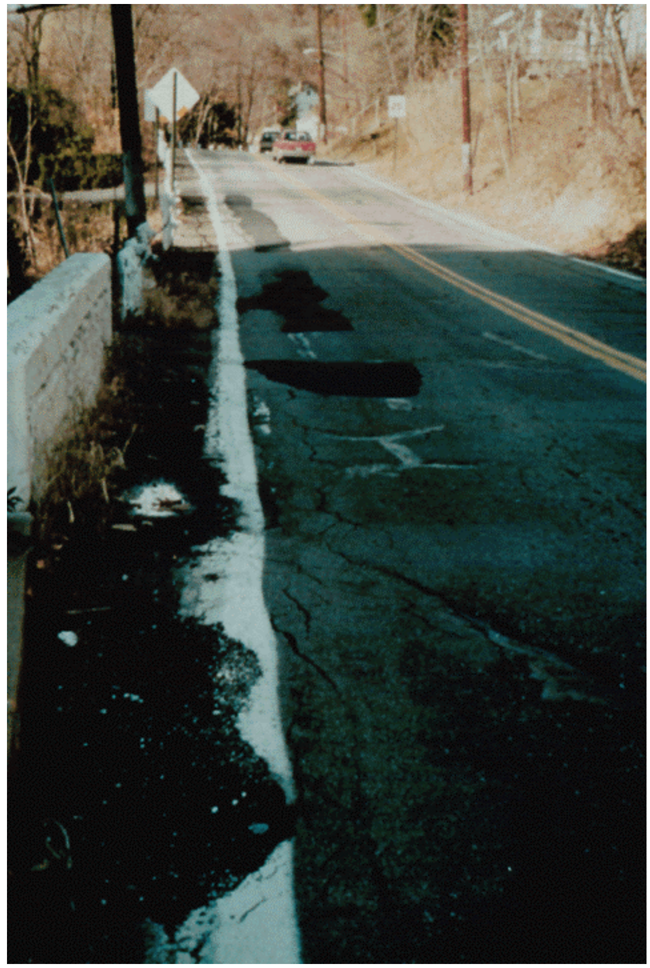


Figure P.3.27 Roadway Over a Culvert

New pavement can temporarily hide approach problems. It is advisable for the inspector to have previous inspection reports that may indicate the age of the present overlay.

It is important to note that not all defects in the approach roadways have an adverse affect on the culvert. Deterioration of the pavement may be due to excessive traffic and no other reason.

Embankment

The embankment around the culvert entrance and exit should be inspected for slide failures in the fill around the box (see Figure P.3.28). Check for debris at the inlet and outlet and within the culvert. Also note if vegetation is obstructing the ends.

Figure P.3.28 Slide Failure

End Treatments

The SI&A Inspection Sheet does not specifically address end treatments in terms of inventory data or condition. The condition rating of end treatments is part of SI&A Item 62, Culvert Condition, and can have an impact on SI&A Item 67, Structural Evaluation.

Inspections of end treatments primarily involve visual inspection, although hand tools should be used such as a plumb bob to check for misalignment, a hammer to sound for defects, and a probing rod to check for scour and undermining. In general, headwalls should be inspected for movement or settlement, cracks, deterioration, and traffic hazards. Culvert ends should be checked for undermining, scour, and evidence of piping.

Figure P.3.29 Headwall and Wingwall End Treatment on Box Culvert

The most common types of box culvert end treatments are:

- Skewed Ends
- Headwalls

Both end treatment types use wingwalls to retain the embankment around the opening.

Wingwalls should be inspected to ensure they are in proper vertical alignment (see Figure P.3.30). 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.

Figure P.3.30 Potential for Tilted Wingwalls

Skewed Ends - Skewing the end of a culvert has nearly the same effect on structural capacity as does mitering (see Figure P.3.31). Stresses increase because a full box shape is not present at the end.

Figure P.3.31 Skewed End

Headwalls – Headwalls and wingwalls should be inspected for undermining and settlement. Cracking, tipping or separation of culvert barrel from the headwall and wingwalls is usually good evidence of undermining. (see Figure P.3.32 and P.3.33).

Figure P.3.32 Culvert Headwall

Appurtenance Structures

Typical appurtenance structures are:

- Aprons
- Energy Dissipators

Aprons – should be checked for any undermining or settlement. The joints between the apron and headwalls should be inspected to see if it is watertight. (see Figure P.3.33)

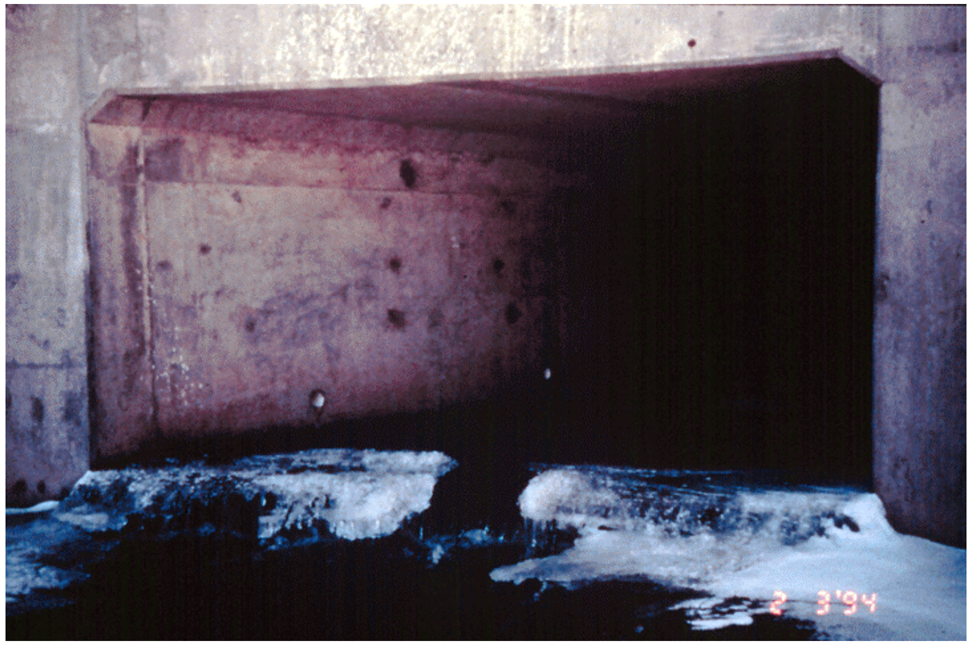


Figure P.3.33 Apron

Energy Dissipaters – are used when outlet velocities are likely to cause streambed scour downstream from the culvert (see Figure P.3.34). Energy dissipaters may include stilling basins, riprap or other devices. Energy dissipaters should be inspected for material defects and overall effectiveness.

Figure P.3.34 Energy Dissipater

Culvert Barrel

The full length of the culvert should be inspected from the inside. All components of the culvert barrel should be visually examined, including walls, floor, top slab, and joints. The concrete should be sounded by tapping with a hammer particularly around cracks and other defects. It is important to time the inspection so that water levels are low. Culverts with small diameters can be inspected by looking through the culvert from both ends or by using a small movable camera. The condition of the culvert barrel is rated under SI&A Item 62, which covers all structural components of a culvert.

For concrete box culverts, the culvert barrels should be inspected primarily for defects such as misalignment, joint defects, cracking, spalling, and other material defects.

P.3.10 Durability

Although the structural condition is a very important element in the performance of culverts, durability problems are probably the most frequent cause of replacement. Culverts are more likely to "wear away" than fail structurally. Durability is affected by two mechanisms: corrosion and abrasion.

Corrosion

Corrosion affects all metals and alloys, although the rates can vary widely depending both upon the chemical and physical properties of the metal and upon the environmental condition to which it is exposed. When a metal corrodes a very low voltage electrical current is established between two parts of a metal surface that have different voltage potential. The difference in voltage potential may be caused by slight variations in the material, changes in surface condition, or the presence of foreign materials. The current removes metallic ions from one location and deposits them at another location, causing corrosion. The chemicals present in the water greatly influence its effectiveness as an electrolyte.

Corrosion is the deterioration of culvert materials by chemical or electrochemical reaction to the environment. Culvert corrosion may occur in many different soils and waters. These soils and waters may contain acids, alkalis, dissolved salts, organics, industrial wastes or other chemicals, mine drainage, sanitary effluents, and dissolved or free gases. However, culvert corrosion is generally related to water and the chemicals that have reacted to, become dissolved in, or been transported by the water.

Corrosion can attack the inside or outside of the culvert barrel. The chemicals in drainage water can attack the material on the interior of the culvert. Culverts subject to continuous flows or standing water with aggressive chemicals are more likely to be damaged than those with intermittent flows. The exterior of culverts can be attacked by chemicals in the ground water which can originate in the soil, be introduced through contaminates in the backfill soil, or be transported by subsurface flow.

Although less common than with metal pipe, corrosion can occur in concrete culverts. Metallic corrosion can take place in the reinforcing steel when it is exposed by cracking or spalling, when the concrete cover is inadequate or when the concrete is porous enough to allow water to contact the reinforcing steel.

If the steel corrodes, the corrosion products expand and may cause spalling of the concrete. Corrosion can also take place in the concrete itself. It is not, however, the same type of electrochemical reaction that occurs in metal. Other reactions between the concrete materials and the chemicals present in the stream flow or ground water are involved and can result in deterioration of the concrete.

Abrasion

Abrasion is the process of wearing down or grinding away surface material as water laden with sand, gravel, or stones flows through a culvert. Abrasive forces increase as the velocity of the water flowing through a culvert increases; for example, doubling the velocity of a stream flow can cause the abrasive power to become approximately four-fold.

Often corrosion and abrasion operate together to produce far greater deterioration than would result from either alone. Abrasion can accelerate corrosion by removing protective coatings and allowing water-borne chemicals to come into contact with corrodible culvert materials.

P.3.11

Soil and Water Conditions that Affect Culverts

Certain soil and water conditions have been found to have a strong relationship to accelerated culvert deterioration. These conditions are referred to as "aggressive" or "hostile." The most significant conditions of this type are:

pH Extremes

pH is a measure of the relative acidity or alkalinity of water. A pH of 7.0 is neutral; values of less than 7.0 are acid, and values of more than 7.0 are alkaline. For culvert purposes, soils or water having a pH of 5.5 or less are strongly acid and those of 8.5 or more are strongly alkaline.

Acid water stems from two sources, mineral and organic. Mineral acidity comes from sulfurous wells and springs, and drainage from coal mines. These sources contain dissolved sulfur and iron sulfide which may form sulfurous and sulfuric acids. Mineral acidity as strong as pH 2.3 has been encountered. Organic acidity usually found in swampy land and barnyards rarely produces a pH of less than 4.0. Alkalinity in water is caused by strong alkali-forming minerals and from limed and fertilized fields. Acid water (low pH) is more common to wet climates and alkaline water (high pH) is more common to dry climates. As the pH of water in contact with culvert materials, either internally or externally, deviates from neutral, 7.0, it generally becomes more hostile.

Electrical Resistivity

This measurement depends largely on the nature and amount of dissolved salts in the soil. The greater the resistance the less the flow of electrical current associated with corrosion. High moisture content and temperature lower the resistivity and increase the potential for corrosion. Soil resistivity generally decreases as the depth increases. The use of granular backfill around the entire pipe will increase electrical resistivity and will reduce the potential for galvanic corrosion.

Several states rely on soil and water resistivity measurements as an important index of corrosion potential. Some states and the FHWA have published guidelines that use a combination of the pH and electrical resistivity of soil and water to indicate the corrosion potential at proposed culvert sites. The collection of pH and electrical resistivity data during culvert inspections can provide valuable information for developing local guidelines.

Soil Characteristics

The chemical and physical characteristics of the soil, which will come into contact with a culvert, can be analyzed to determine the potential for corrosion. The presence of base-farming and acid-forming chemicals is important. Chlorides and other dissolved salts increase electrical conductivity and promote the flow of corrosion currents. Sulfate soils and water can be erosive to metals and harmful to concrete. The permeability of soil to water and to oxygen is another variable in the corrosion process.

P.3.12 Culvert Protective Systems

There are several protective measures that can be taken to increase the durability of culverts. The more commonly used measures are:

Extra Thickness

For some aggressive environments, it may be economical to provide extra thickness of concrete or metal.

Bituminous Coating

This is the most common protective measure used on corrugated steel pipe. This procedure can increase the resistance of metal pipe to acidic conditions if the coating is properly applied and remains in place. Careful handling during transportation, storage, and placement is required to avoid damage to the coating. Bituminous coatings can also be damaged by abrasion. Field repairs should be made when bare metal has been exposed. Fiber binding is sometimes used to improve the adherence of bituminous material to the metallic-coated pipe.

Bituminous Paved Inverts

Paving the inverts of corrugated metal culverts to provide a smooth flow and to protect the metal has sometimes been an effective protection from particularly abrasive and corrosive environments. Bituminous paving is usually at least 3 mm (1/8-inch) thick over the inner crest of the corrugations. Generally only the lower quadrant of the pipe interior is paved. Fiber binding is sometimes used to improve the adherence of bituminous material to the metallic-coated pipe.

Other Coatings

There are several other coating materials that are being used to some degree throughout the country. Polymeric, epoxy, fiberglass, clay, and concrete field paving, have all been used as protection against corrosion. Galvanizing is the most common of the metallic coatings used for steel. It involves the application of a thin layer of zinc on the metal culvert. Other metallic coatings used to protect steel culverts are aluminum and aluminum-zinc.

This page intentionally left blank

Table of Contents

Section 1
Bridge Inspection Programs

Abbreviations Used in this Section

AASHO	-	American Association of State Highway Officials (1921 to 1973)
AASHTO	-	American Association of State Highway and Transportation Officials (1973 to present)
AASHTO Manual	-	*Manual for Maintenance Inspection of Bridges*
BIRM	-	*Bridge Inspector's Reference Manual*
BMS	-	Bridge Management System
Coding Guide	-	FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*
DOT	-	Department of Transportation
FCM	-	fracture critical member
FHWA	-	Federal Highway Administration
HBRR	-	Highway Bridge Replacement & Rehabilitation
HEC	-	*Hydraulic Engineering Circular*
ISTEA	-	Intermodal Surface Transportation Efficiency Act
Manual 70	-	*Bridge Inspector's Training Manual 70*
Manual 90	-	*Bridge Inspector's Training Manual 90*
MR&R	-	maintenance, repair and rehabilitation
NBI	-	National Bridge Inventory
NBIS	-	National Bridge Inspection Standards
NCHRP	-	National Cooperative Highway Research Program
NDT	-	nondestructive testing
NHI	-	National Highway Institute
NHS	-	National Highway System
NICET	-	National Institute for Certification in Engineering Technologies
TEA-21	-	Transportation Equity Act of the 21st Century
TRB	-	Transportation Research Board
TWG	-	Technical Working Group

Section 1

Bridge Inspection Programs

Topic 1.1 History of the National Bridge Inventory Program

1.1.1 Introduction

In the years since the Federal Highway Administration's landmark publication, *Bridge Inspector's Training Manual 90* (*Manual 90*), bridge inspection and inventory programs of state and local governments have formed an important basis for formal bridge management programs. During the 1990's, the state DOT's implemented comprehensive bridge management systems, which rely heavily on accurate, consistent bridge inspection data.

This manual (*Bridge Inspector's Reference Manual*) updates *Manual 90* and reflects over ten years of change.

Advances in technology and construction have greatly enhanced current bridge design. However, the emergence of previously unknown problem areas and the escalating cost of replacing older bridges make it imperative that existing bridges be evaluated properly to be kept open and safe.

There are four letters that define the scope of bridge inspections in this country: NBIS, meaning National Bridge Inspection Standards. The **National Bridge Inspection Standards (NBIS)** are Federal regulations establishing requirements for:

- Inspection Procedures
- Frequency of Inspections
- Qualifications of Personnel
- Inspection Reports
- Maintenance of Bridge Inventory

The **National Bridge Inventory (NBI)** is the aggregation of structure inventory and appraisal data collected by each state to fulfill the requirements of NBIS.

To better understand the National Bridge Inventory Program, it is helpful if we take a look back at the development of the program.

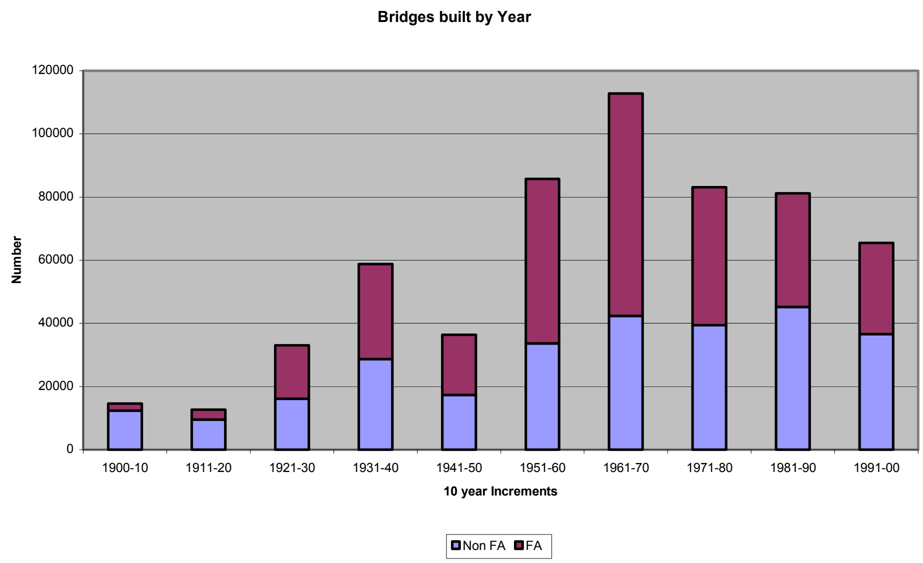


Table 1.1.1 Number of Bridges Built since 1900

1.1.2 History of the National Bridge Inventory Program

Background

During the bridge construction boom of the 1950's and 1960's, little emphasis was placed on safety inspection and maintenance of bridges. This changed when the 681 m (2,235-foot) Silver Bridge, at Point Pleasant, West Virginia, collapsed into the Ohio River on December 15, 1967, killing 46 people (see Figure 1.1.1).

Figure 1.1.1 Collapse of the Silver Bridge

This tragic collapse aroused national interest in the safety inspection and maintenance of bridges. The U.S. Congress was prompted to add a section to the "Federal Highway Act of 1968" which required the Secretary of Transportation to establish a national bridge inspection standard. The Secretary was also required to develop a program to train bridge inspectors.

The 1970's

Thus, in 1971, the National Bridge Inspection Standards (NBIS) came into being. The NBIS established national policy regarding:

- Inspection procedures
- Frequency of inspections
- Qualifications of personnel
- Inspection reports
- Maintenance of state bridge inventory (NBI)

Three manuals were subsequently developed. These manuals were vital to the early success of the NBIS. The first manual was the Federal Highway Administration (FHWA) *Bridge Inspector's Training Manual 70* (*Manual 70)*. This manual set the standard for inspector training.

The second manual was the American Association of State Highway Officials (AASHO) *Manual for Maintenance Inspection of Bridges*, released in 1970. This manual served as a standard to provide uniformity in the procedures and policies for determining the physical condition, maintenance needs and load capacity of highway bridges.

The third manual was the FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*Coding Guide*), released in July 1972. It provided thorough and detailed guidance in evaluating and coding specific bridge data.

With the publication of *Manual 70*, the implementation of national standards and guidelines, the support of AASHO, and a newly available FHWA bridge inspector's training course for use in individual states, improved inventory and appraisal of the nation's bridges seemed inevitable. Several states began in-house training programs, and the 1970's looked promising. Maintenance and inspection problems associated with movable bridges were also addressed. In 1977, a supplement to *Manual 70*, the *Bridge Inspector's Manual for Movable Bridges*, was added.

However, the future was not to be trouble free. Two predominant concerns were identified during this period. One concern was that bridge repair and replacement needs far exceeded available funding. The other was that NBIS activity was limited to bridges on the Federal Aid highway systems. This resulted in little incentive for inspection and inventory of bridges not on Federal Aid highway systems.

These two concerns were addressed in the "Surface Transportation Assistance Act of 1978." This act provided badly needed funding for rehabilitation and new construction and required that all public bridges over 20 feet (6.1 m) in length be inspected and inventoried in accordance with the NBIS by December 31, 1980.

Any bridge not inspected and inventoried in compliance with NBIS would be ineligible for funding from the special replacement program.

In 1978, the American Association of State Highway and Transportation Officials (AASHTO) revised their *Manual for Maintenance Inspection of Bridges* (*AASHTO Manual*). In 1979, the NBIS and the FHWA *Coding Guide* were also revised. These publications, along with *Manual 70*, provided state agencies with definite guidelines for compliance with the NBIS.

The 1980's

The National Bridge Inspection Program was now maturing and well positioned for the coming decade. Two additional supplements to *Manual 70* were published. First, culverts became an area of interest after several tragic failures. The 1979 NBIS revisions also prompted increased interest in culverts. The *Culvert Inspection Manual* was published July 1986. Then, an emerging national emphasis on fatigue and fracture critical bridges was sharply focused by the collapse of Connecticut's Mianus River Bridge in June 1983. *Inspection of Fracture Critical Bridge Members* was published in September 1986. These manuals were the products of ongoing research in these problem areas.

With the April 1987 collapse of New York's Schoharie Creek Bridge, national attention turned to underwater inspection. Of the 592,000 bridges in the national inventory, approximately 86% are over waterways. The FHWA responded with *Scour at Bridges*, a technical advisory published in September 1988. This advisory provided guidance for developing and implementing a scour evaluation program for the:

- Design of new bridges to resist damage resulting from scour
- Evaluation of existing bridges for vulnerability to scour
- Use of scour countermeasures
- Improvement of the state-of-practice of estimating scour at bridges

Further documentation is available on this topic in the *Hydraulic Engineering Circular No. 18* (*HEC-18*).

In September 1988, the NBIS was modified, based on suggestions made in the "1987 Surface Transportation and Uniform Relocation Assistance Act," to require states to identify bridges with fracture critical details and establish special inspection procedures. The same requirements were made for bridges requiring underwater inspections. The NBIS revisions also allowed for adjustments in the frequency of inspections and the acceptance of National Institute for Certification in Engineering Technologies (NICET) Level III and IV certification for inspector qualifications.

In December 1988, the FHWA issued a revision to the *Coding Guide*. This time the revision would be one of major proportions, shaping the National Bridge Inspection Program for the next decade. The *Coding Guide* provided inspectors with additional direction in performing uniform and accurate bridge inspections.

The 1990's

The 1990's was the decade for bridge management systems (BMS). Several states, including New York, Pennsylvania, North Carolina, Alabama and Indiana, had their own comprehensive bridge management systems.

In 1991, the FHWA sponsored the development of a bridge management system called "Pontis" which is derived from the Latin word for bridge. The Pontis system has sufficient flexibility to allow customization to any agency or organization responsible for maintaining a network of bridges.

Simultaneously, the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board (TRB) developed a BMS software called "Bridgit." Bridgit is primarily targeted to smaller bridge inventories or local highway systems.

As more and more bridge needs were identified, it became evident that needed funding for bridge maintenance, repair and rehabilitation (MR&R) far exceeded the available funding from federal and state sources. Even with the infusion of financial support provided by the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, funding for bridge MR&R projects was difficult to obtain. This was due in part to the enormous demand from across the nation. An October 1993 revision to NBIS permitted bridge owners to request approval from FHWA of extended inspection cycles of up to four years for bridges meeting certain requirements.

In 1994, the American Association of State Highway and Transportation Officials (AASHTO) revised their *Manual for Condition Evaluation of Bridges* (*AASHTO Manual*). In 1995, the FHWA *Coding Guide* was also revised. These publications, along with *Manual 90, Revised July 1995*, provided state agencies with continued definite guidelines for compliance with the NBIS and conducting bridge inspection.

Although later rescinded in the next transportation bill, the ISTEA legislation required that each state implement a comprehensive bridge management system by October 1995. This deadline represented a remarkable challenge since few states had previously implemented a BMS that could be considered to meet the definition of a comprehensive BMS. In fact, prior to the late 1980's, there were no existing management systems adaptable to the management of bridge programs nor was there any clear, accepted definition of key bridge management principles or objectives.

This flexibility in the system was the result of developmental input by a Technical Working Group (TWG) comprised of representatives from the FHWA, the Transportation Research Board (TRB) and the following six states: California, Minnesota, North Carolina, Tennessee, Vermont and Washington. The TWG provided guidance drawing on considerable experience in bridge management and engineering.

The National Highway System (NHS) Act of 1995 rescinded the requirement for bridge management systems. However, many of the states continued to implement the Pontis BMS.

The Transportation Equity Act of the 21st Century (TEA-21) was signed into law in June 1998. TEA-21 builds on and improves the initiatives established in ISTEA and, as mentioned earlier, rescinded the mandatory BMS requirement.

The 2000's

In 2002, *Manual 90* was revised and updated as a part of a complete overhaul of the FHWA Bridge Safety Inspection training program. The new manual was named the *Bridge Inspector's Reference Manual* (BIRM) and incorporated all of *Manual 90.* The BIRM also incorporates manual 70 Supplements for culvert inspection and Fracture Critical Members, and course curriculum material that was not specifically part of the revised course objectives. Over the years, varying amounts of federal funds have been spent on bridge projects, depending on the demands of the transportation infrastructure. Table 1.1.2 illustrates the fluctuations in federal spending and shows current trends.

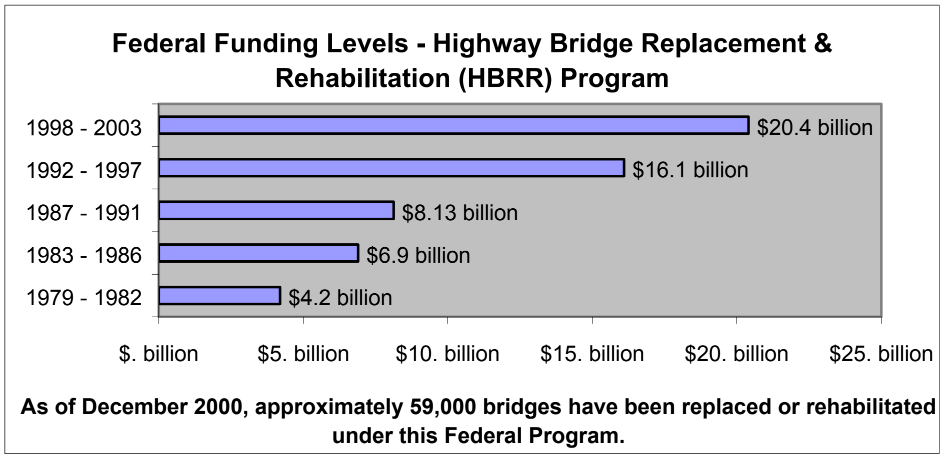


Table 1.1.2 Federal Funding Levels (1979 – 2003)

1.1.3 Today's National Bridge Inspection Program

Much has been learned in the field of bridge inspection, and a national Bridge Inspection Training Program is now fully implemented. State and federal inspection efforts are more organized, better managed and much broader in scope. The technology used to inspect and evaluate bridge members and bridge materials has significantly improved.

Areas of emphasis in bridge inspection programs are changing and expanding as new problems become apparent, as newer bridge types become more common, and as these newer bridges age enough to have areas of concern. Guidelines for inspection ratings have been refined to increase uniformity and consistency of inspections. Data from bridge inspections has become critical input into a variety of analyses and decisions by state agencies and the Federal Highway Administration.

The NBIS has kept current with the field of bridge inspection. The 1995 National Bridge Inspection Standards appear in Appendix A. The standards are divided into the following sections:

- Application of standards
- Inspection procedures
- Frequency of inspections
- Qualifications of personnel

- Inspection report
- Inventory

The FHWA has made a considerable effort to make available to the nation's bridge inspectors the information and knowledge necessary to accurately and thoroughly inspect and evaluate the nation's bridges.

FHWA Training

The FHWA has developed and now offers the following training courses relative to bridge inspection through the National Highway Institute (NHI):

- "Bridge Inspector's Training Course, Part I - Engineering Concepts for Bridge Inspectors" (NHI Course Number 130054)

  This one-week course presents engineering concepts, as well as inspection procedures and information about bridge types, bridge components, and bridge materials. The one-week course is for new inspectors with little or no practical bridge inspection experience.

- "Bridge Inspector's Training Course, Part II - Safety Inspection of In-Service Bridges" (NHI Course Number 130055)

  This two-week course is for experienced inspectors or engineers who perform or manage bridge inspections. Emphasis is on inspection applications and procedures. The uniform coding and rating of bridge elements and components is also an objective of the two-week course. A unique feature of this course allows for customization of the course content by the host agency. Some states use component rating based on NBIS while some states use element condition level based on Pontis. Optional topics can be scheduled, and their level of coverage can be selected. These topics include identification and inspection of fracture critical members (FCM's), underwater inspection, culverts, field trips, case studies, movable bridges, and coatings. Several special bridge types may also be discussed at the host agency's request.

- "Fracture Critical Inspection Techniques for Steel Bridges" (NHI Course Number 130078)

  This three and one-half day course provides an understanding of fracture critical members (FCM's), FCM identification, failure mechanics and fatigue in metal. Emphasis is placed on inspection procedures and reporting of common FCM's and nondestructive testing (NDT) methods most often associated with steel highway bridges.

- "Bridge Inspection Refresher Training" (NHI Course Number 130053)

  This three-day course provides a review of the National Bridge Inventory (NBI) inspection methods and includes discussions on structure inventory items, structure types, and the appropriate codes for the Federal Structure, Inventory and Appraisal reporting.

- "Stream Stability and Scour at Highway Bridges for Bridge Inspectors"

(NHI Course Number 135047)

This one-day course concentrates on visual keys to detecting scour and stream instability problems. The course emphasizes inspection guidelines to complete the hydraulic and scour-related coding requirements of the National Bridge Inspection Standards (NBIS).

- "Bridge Coatings Inspection" (NHI Course Number 130079)

  This four-day course provides information on the inspection of surface preparation and application of protective coating systems for bridge and highway structures. The course provides a basic overview of the theory of corrosion and its control and the characteristics of various bridge coating types.

Throughout all the expansions and improvements in bridge inspection programs and capabilities, one factor remains constant: the overriding importance of the inspector's ability to effectively inspect bridge components and materials and to make sound evaluations with accurate ratings. The validity of all analyses and decisions based on the inspection data is dependent on the quality and the reliability of the data collected in the field.

Across the nation, the duties, responsibilities, and qualifications of bridge inspectors vary widely. The two keys to a knowledgeable, effective inspection are training and experience in performing actual bridge inspections. Training of bridge inspectors has been, and will continue to be, an active process within state highway agencies for many years. This manual is designed to be an integral part of that training process.

Current FHWA Reference Material

- NBIS. *Code of Federal Regulations.* 23 Highways Part 650, Subpart C – National Bridge Inspection Standards.

- AASHTO. *LRFD Bridge Design Specifications, 2nd Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, 1998.

- FHWA. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.* Washington, D.C.: United States Department of Transportation, 1995.
  - http://www.fhwa.dot.gov/bridge/mtguide.pdf

- FHWA. *Bridge Inspector's Reference Manual.* Washington, D.C.: United States Department of Transportation, 2002.

- AASHTO. *Manual for Condition Evaluation of Bridges, 2nd Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2000.

- AASHTO. *CoRe Elements.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2000.

Table of Contents

This page intentionally left blank

Topic 1.2 Responsibilities of the Bridge Inspector

1.2.1 Introduction

Bridge inspection has played, and will continue to play, an increasingly important role in providing a safe infrastructure for our nation. As our nation's bridges continue to age and deteriorate, an accurate and thorough assessment of each bridge's condition is critical in maintaining a safe, functional and reliable highway system.

This section presents the responsibilities of the bridge inspector. It also describes how the inspector can prepare for the inspection and some of the major inspection procedures.

1.2.2 Responsibilities of the Bridge Inspector and Engineer

There are five basic responsibilities of the bridge inspector and engineer:

- Maintain public safety and confidence
- Protect public investment
- Provide bridge inspection program support
- Provide accurate bridge records
- Fulfill legal responsibilities

1. Maintain Public Safety and Confidence

The primary responsibility of the bridge inspector is to maintain public safety and confidence. This is also a prime concern to everyone in the highway agency. The general public travels our highways and bridges without hesitation. However, when a bridge fails, the public's confidence in our bridge system is violated (see Figure 1.2.1). The design engineer's role in assuring bridge safety is:

- To incorporate safety factors.
- To provide cost-effective designs.

  Engineers provide a margin of safety to compensate for a lack of precise calculations, variations in the quality of material, erection loading conditions, and uncertain maintenance. This is particularly evident in older bridges, especially those designed prior to the use of computers. The bridge design engineer must be as confident as possible that the bridge will never fail under natural or man-made loads.

The inspector's role is:

- To provide thorough inspections identifying bridge conditions and defects,
- To prepare condition reports documenting these deficiencies and alerting supervisors or engineers of any findings which might impact the safety of the roadway user or the integrity of the structure.

Figure 1.2.1 Bridge Failure

2. Protect Public Investment

Another responsibility is to protect public investment in bridges. The inspector must be on guard for minor problems that can be corrected before they lead to costly major repairs. The inspector must also be able to recognize bridge elements that need repair in order to maintain bridge safety and avoid replacement costs.

As stated before, the funding available to rehabilitate and replace deficient bridges is not adequate to meet all of the needs. It is important that preservation activities be a part of the bridge program to extend the performance life of as many bridges as possible and minimize the need for costly repairs or replacement.

The inspector's role is to:

- Continually be on guard for minor problems that can become costly repairs.
- Recognize bridge components that need repair in order to maintain bridge safety and avoid the need for costly replacement.

The engineer's role is to:

- Continually upgrade design standards to promote longevity of bridge performance.

3. Provide Bridge Inspection Program Support

Subpart C of the National Bridge Inspection Standards (NBIS) of the *Code of Federal Regulations*, 23 Highways Part 650, mandates:

- Inspection procedures
- Frequency of inspections
- Qualifications of personnel
- Reporting
- Inventory

Bridge Inspection Programs are funded by public tax dollars. Therefore, the bridge inspector is financially responsible to the public.

The "Surface Transportation Act of 1978" established the funding mechanism for providing federal funds for bridge replacement. The Act also established criteria for bridge inspections and requirements for compliance with the NBIS.

The "Intermodal Surface Transportation Efficiency Act" (ISTEA) of 1991 and the Transportation Equity Act for the 21st Century (TEA-21) of 1998 establish funding mechanisms for tolled and free bridges for bridge maintenance, rehabilitation and replacement to adequately preserve the bridges and their safety to all users.

4. Provide Accurate Bridge Records

There are three major reasons why accurate bridge records are required:

a. To establish and maintain a structure history file.

For example, two bridge abutments are measured for tilt during several inspection cycles, and the results are as follows:

Year	Abutment A	Abutment B
2000	106 mm (4-3/16")	89 mm (3-1/2")
1998	106 mm (4-3/16")	57 mm (2-1/4")
1996	105 mm (4-1/8")	29 mm (1-1/8")
1994	102 mm (4")	25 mm (1")

Looking at year 2000 measurements only would indicate that Abutment A has a more severe problem. However, examining the changes each year, we see that the movement of A is slowing and may have stopped, while B is changing at a faster pace each inspection cycle. At the rate it is moving, B will probably surpass A by the next inspection.

b. To identify and assess bridge deficiencies and to identify and assess bridge repair requirements. An individual should be able to readily determine, from the records, what repairs are needed as well as a good estimate of quantities.

c. To identify and assess minor bridge deficiencies and to identify and assess bridge maintenance needs in a similar manner to the repair requirements.

To ensure accurate bridge records, proper record keeping needs to be maintained. A system should be developed to review bridge data and evaluate quality of bridge inspections.

5. Fulfill Legal Responsibilities

A bridge inspection report is a legal document. Descriptions must be specific, detailed, quantitative (where possible), and complete. Vague adjectives such as good, fair, poor, and general deterioration, without concise descriptions to back them up, should not be used. To say "the bridge is OK" is just not good enough.

Example of inspection descriptions:
Bad description: "Fair beams"
Good description: "Stringers in fair condition with light scaling on bottom flanges

of Beams B and D for their full length"
Bad description: "Deck in poor condition"
Good description: "Deck in poor condition with spalls covering 50% of the deck as indicated on field sketch, see Figure 42"

Any visual assessments should include phrases such as "no other apparent defects" or "no other defects observed."

Original inspection notes should not be altered without consultation with the inspector who wrote the notes.

A bridge inspection report implies that the inspection was performed in accordance with the National Bridge Inspection Standards, unless specifically stated otherwise in the report. Proper equipment, techniques, and personnel must be used. If the inspection is a special or interim inspection, this must be explained explicitly in the report.

1.2.3 Qualifications of Bridge Inspectors

The NBIS are very specific with regard to the qualifications of bridge inspectors. The *Code of Federal Regulations*, Title 23, Chapter 1, Section 650-307, (23 CFR 1.650.307), lists the qualifications of personnel for the National Bridge Inspection Standards. These are minimum standards; therefore, state or local highway agencies can implement higher requirements.

Inspection Program Manager

(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 Reference Manual," which has been developed by a joint Federal-State task force, and subsequent additions to the manual.

Inspection Team Leader

(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 Reference Manual," which has been developed by a joint Federal-State task force.

(3) Current certification as a Level III or IV Bridge Safety Inspector under the National Society of Professional Engineers' program for the National Institute For Certification in Engineering Technologies (NICET) is an alternate acceptable means for establishing that a bridge inspection team leader is qualified.

Qualifications and responsibilities of inspection personnel can be found in Section 3.4 of the AASHTO Manual for Condition Evaluation of Bridges, 2nd Ed.

1.2.4 Consequence of Irresponsibility

The dictionary defines tort as "a wrongful act for which a civil action will lie except one involving a breach of contract."

In the event of negligence in carrying out the basic responsibilities described above, an individual, including department heads, engineers, and inspectors, is subject to personal liability. An inspector should strive to be as objective and complete as possible. Accidents that result in litigation are generally related, but not necessarily limited, to the following:

- Deficient safety features
- Failed members
- Failed substructure elements
- Failed joints or decks
- Potholes or other hazards to the traveling public
- Improper or deficient load posting

Anything said or written in the bridge file could be used in litigation cases held against you. In litigation involving a bridge, the inspection notes and reports may be used as evidence. A subjective report may have negative consequences for the highway agency involved in lawsuits involving bridges. The report will be scrutinized to determine if conditions are documented thoroughly and for the "proper" reasons. An inspector should, therefore, strive to be as objective and complete as possible. State if something could not be inspected.

Example of liabilities:

In a recent case, a consulting firm was found liable for negligent inspection practices. A tractor-trailer hit a large hole in a bridge deck, swerved, went through the guard rail, and fell 9.1 m (30 feet) to the ground. Ten years prior to the accident, the consulting firm had noted severe deterioration of the deck and had recommended tests to determine the need for replacement. Two years prior to the accident, their annual inspection report did not show the deterioration or recommend repairs. One year before the accident, inspectors from the consultant checked 345 bridges in five days, including the bridge on which the accident occurred. The court found that the consulting firm had been negligent in its inspection, and assessed the firm 75% of the ensuing settlement.

In another case, four cars drove into a hole 3.7 m (12 feet) deep and 9.1 m (30 feet) across during the night. Five people were killed and four were injured. The hole was the result of a collapse of a multi-plate arch. Six lawsuits were filed and, defendants included the county, the county engineer, the manufacturer, the supplier, and the consulting engineers who inspected the arch each year. The arch was built and backfilled, with mostly clay, by a county maintenance crew 16 years prior to the accident. Three years later, the county engineer found movement of 75 to 100 mm (3 to 4 inches) at one headwall. The manufacturer sent an inspector, who determined that the problem was backfill-related and recommended periodic measurements. These measurements were done once, but the arch was described as "in good condition" or "in good condition with housekeeping necessary" on subsequent inspections. Inspection reports documented a 150 mm (6 inch) gap between the steel plate and the headwall. A contractor examined the arch at the county engineer's request to provide a proposal for shoring. The county engineer discussed the proposal with the consulting engineers a month before the accident. Thirteen inspections in all were conducted on the structure. An engineering report accuses the county engineer of poor engineering practice.

Table of Contents

This page intentionally left blank

Table 4C Design Values for Mechanically Graded Dimension Lumber[1,2,3]

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 2.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4C ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi) Bending F_b	Tension parallel to grain F_t	Compression parallel to grain F_c	Modulus of Elasticity E	Grading Rules Agency
MACHINE STRESS RATED (MSR) LUMBER						
900f-1.0E	2"& less in thickness 2"& wider	900	350	1050	1,000,000	WCLIB, WWPA
1200f-1.2E		1200	600	1400	1,200,000	NLGA, WCLIB, WWPA
1250f-1.4E		1250	800	1475	1,400,000	WCLIB
1350f-1.3E		1350	750	1600	1,300,000	NLGA, WCLIB, WWPA
1400f-1.2E		1400	800	1600	1,200,000	NLGA
1450f-1.3E		1450	800	1625	1,300,000	NLGA, WCLIB, WWPA
1500f-1.3E		1500	900	1650	1,300,000	WWPA
1500f-1.4E		1500	900	1650	1,400,000	NLGA, WCLIB, WWPA
1600f-1.4E		1600	950	1675	1,400,000	NLGA
1650f-1.3E		1650	1020	1700	1,300,000	NLGA, WWPA
1650f-1.5E		1650	1020	1700	1,500,000	NLGA, SPIB, WCLIB, WWPA
1650f-1.6E		1650	1175	1700	1,600,000	WCLIB, WWPA
1700f-1.6E		1700	1175	1725	1,600,000	WCLIB
1750f-2.0E		1750	1125	1725	2,000,000	WCLIB
1800f-1.5E		1800	1300	1750	1,500,000	NLGA, WWPA
1800f-1.6E		1800	1175	1750	1,600,000	NLGA, SPIB, WCLIB, WWPA
1950f-1.5E		1950	1375	1800	1,500,000	SPIB, WWPA
1950f-1.7E		1950	1375	1800	1,700,000	NLGA, SPIB, WCLIB, WWPA
2000f-1.6E		2000	1300	1825	1,600,000	NLGA
2100f-1.8E		2100	1575	1875	1,800,000	NLGA, SPIB, WCLIB, WWPA
2250f-1.7E		2250	1750	1925	1,700,000	NLGA, WWPA
2250f-1.8E		2250	1750	1925	1,800,000	NLGA, WCLIB, WWPA
2250f-1.9E		2250	1750	1925	1,900,000	NLGA, SPIB, WCLIB, WWPA
2400f-1.8E		2400	1925	1975	1,800,000	NLGA, WWPA
2400f-2.0E		2400	1925	1975	2,000,000	NLGA, SPIB, WCLIB, WWPA
2500f-2.2E		2500	1750	2000	2,200,000	WCLIB
2550f-2.1E		2550	2050	2025	2,100,000	NLGA, SPIB, WCLIB, WWPA
2700f-2.0E		2700	1800	2100	2,000,000	WCLIB, WWPA
2700f-2.2E		2700	2150	2100	2,200,000	NLGA, SPIB, WCLIB, WWPA
2850f-2.3E		2850	2300	2150	2,300,000	NLGA, SPIB, WCLIB, WWPA
3000f-2.4E		3000	2400	2200	2,400,000	NLGA, SPIB
MACHINE EVALUATED LUMBER (MEL)						
M-5	2"& less in thickness 2"& wider	900	500	1050	1,100,000	SPIB
M-6		1100	600	1300	1,000,000	SPIB
M-7		1200	650	1400	1,100,000	SPIB
M-8		1300	700	1500	1,300,000	SPIB
M-9		1400	800	1600	1,400,000	SPIB
M-10		1400	800	1600	1,200,000	NLGA, SPIB
M-11		1550	850	1675	1,500,000	NLGA, SPIB
M-12		1600	850	1675	1,600,000	NLGA, SPIB
M-13		1600	950	1675	1,400,000	NLGA, SPIB
M-14		1800	1000	1750	1,700,000	NLGA, SPIB
M-15		1800	1100	1750	1,500,000	NLGA, SPIB
M-16		1800	1300	1750	1,500,000	SPIB
M-17[4]		1950	1300	2050	1,700,000	SPIB
M-18		2000	1200	1825	1,800,000	NLGA, SPIB
M-19		2000	1300	1825	1,600,000	NLGA, SPIB
M-20[4]		2000	1600	2100	1,900,000	SPIB
M-21		2300	1400	1950	1,900,000	NLGA, SPIB
M-22		2350	1500	1950	1,700,000	NLGA, SPIB
M-23		2400	1900	1975	1,800,000	NLGA, SPIB
M-24		2700	1800	2100	1,900,000	NLGA, SPIB
M-25		2750	2000	2100	2,200,000	NLGA, SPIB
M-26		2800	1800	2150	2,000,000	NLGA, SPIB
M-27[4]		3000	2000	2400	2,100,000	SPIB
M-28		2200	1600	1900	1,700,000	SPIB
M-29		1550	850	1650	1,700,000	SPIB

Table 4D Design Values for Visually Graded Timbers (5" x 5" and larger)

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 2.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4D ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi): Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Grading Rules Agency
BALSAM FIR								
Select Structural	Beams and Stringers	1350	900	65	305	950	1,400,000	NELMA NSLB
No.1	Beams and Stringers	1100	750	65	305	800	1,400,000	
No.2	Beams and Stringers	725	350	65	305	500	1,100,000	
Select Structural	Posts and Timbers	1250	825	65	305	1000	1,400,000	
No.1	Posts and Timbers	1000	675	65	305	875	1,400,000	
No.2	Posts and Timbers	575	375	65	305	400	1,100,000	
BEECH-BIRCH-HICKORY								
Select Structural	Beams and Stringers	1650	975	90	715	975	1,500,000	NELMA
No.1	Beams and Stringers	1400	700	90	715	825	1,500,000	
No.2	Beams and Stringers	900	450	90	715	525	1,200,000	
Select Structural	Posts and Timbers	1550	1050	90	715	1050	1,500,000	
No.1	Posts and Timbers	1250	850	90	715	900	1,500,000	
No.2	Posts and Timbers	725	475	90	715	425	1,200,000	
COAST SITKA SPRUCE								
Select Structural	Beams and Stringers	1150	675	60	455	775	1,500,000	NLGA
No.1	Beams and Stringers	950	475	60	455	650	1,500,000	
No.2	Beams and Stringers	625	325	60	455	425	1,200,000	
Select Structural	Posts and Timbers	1100	725	60	455	825	1,500,000	
No.1	Posts and Timbers	875	575	60	455	725	1,500,000	
No.2	Posts and Timbers	525	350	60	455	500	1,200,000	
DOUGLAS FIR-LARCH								
Dense Select Structural	Beams and Stringers	1900	1100	85	730	1300	1,700,000	WCLIB
Select Structural	Beams and Stringers	1600	950	85	625	1100	1,600,000	
Dense No.1	Beams and Stringers	1550	775	85	730	1100	1,700,000	
No.1	Beams and Stringers	1350	675	85	625	925	1,600,000	
No.2	Beams and Stringers	875	425	85	625	600	1,300,000	
Dense Select Structural	Posts and Timbers	1750	1150	85	730	1350	1,700,000	
Select Structural	Posts and Timbers	1500	1000	85	625	1150	1,600,000	
Dense No.1	Posts and Timbers	1400	950	85	730	1200	1,700,000	
No.1	Posts and Timbers	1200	825	85	625	1000	1,600,000	
No.2	Posts and Timbers	750	475	85	625	700	1,300,000	
Dense Select Structural	Beams and Stringers	1850	1100	85	730	1300	1,700,000	WWPA
Select Structural	Beams and Stringers	1600	950	85	625	1100	1,600,000	
Dense No.1	Beams and Stringers	1550	775	85	730	1100	1,700,000	
No.1	Beams and Stringers	1350	675	85	625	925	1,600,000	
Dense No.2	Beams and Stringers	1000	500	85	730	700	1,400,000	
No.2	Beams and Stringers	875	425	85	625	600	1,300,000	
Dense Select Structural	Posts and Timbers	1750	1150	85	730	1350	1,700,000	
Select Structural	Posts and Timbers	1500	1000	85	625	1150	1,600,000	
Dense No.1	Posts and Timbers	1400	950	85	730	1200	1,700,000	
No.1	Posts and Timbers	1200	825	85	625	1000	1,600,000	
Dense No.2	Posts and Timbers	800	550	85	730	550	1,400,000	
No.2	Posts and Timbers	700	475	85	625	475	1,300,000	
DOUGLAS FIR-LARCH (NORTH)								
Select Structural	Beams and Stringers	1600	950	85	625	1100	1,600,000	NLGA
No.1	Beams and Stringers	1300	675	85	625	925	1,600,000	
No.2	Beams and Stringers	875	425	85	625	600	1,300,000	
Select Structural	Posts and Timbers	1500	1000	85	625	1150	1,600,000	
No.1	Posts and Timbers	1200	825	85	625	1000	1,600,000	
No.2	Posts and Timbers	725	475	85	625	700	1,300,000	
DOUGLAS FIR-SOUTH								
Select Structural	Beams and Stringers	1550	900	85	520	1000	1,200,000	WWPA
No.1	Beams and Stringers	1300	625	85	520	850	1,200,000	
No.2	Beams and Stringers	825	425	85	520	525	1,000,000	
Select Structural	Posts and Timbers	1400	950	85	520	1050	1,200,000	
No.1	Posts and Timbers	1150	775	85	520	925	1,200,000	
No.2	Posts and Timbers	650	400	85	520	425	1,000,000	

Table 5A Design Values for Structural Glued Laminated Softwood Timber

(Members stressed primarily in bending) [1,2,3,4,12] (Tabulated design values are for normal load duration and dry service conditions. See NDS 2.3 for a comprehensive description of design value adjustment factors.)

Use with Table 5A Adjustment Factors

		Design values in pounds per square inch (psi)													
		BENDING ABOUT X-X AXIS (Loaded Perpendicular to Wide Faces of Laminations)						BENDING ABOUT Y-Y AXIS (Loaded Parallel to Wide Faces of Laminations)					AXIALLY LOADED		
		Bending		Compression Perpendicular to Grain											
Combination Symbol[4]	Species Outer Lams/ Core Lams[5]	Tension Zone Stressed in Tension F_{bxx}	Compression Zone Stressed in Tension[6] F_{bxx}	Tension Face[9,10] $F_{c\perp xx}$	Compression Face[9,10] $F_{c\perp xx}$	Shear Parallel to Grain[10] F_{vxx}	Modulus of Elasticity E_{xx}	Bending F_{byy}	Compression Perpendicular to Grain (Side Faces) $F_{c\perp yy}$	Shear Parallel to Grain F_{vyy}	Shear Parallel to Grain (For Members With Multiple Piece Laminations Which are Not Edge Glued)[13] F_{vyy}	Modulus of Elasticity E_{yy}	Tension Parallel to Grain F_t	Compression Parallel to Grain F_c	Modulus of Elasticity E
VISUALLY GRADED WESTERN SPECIES															
16F-V1	DF/WW	1600	800	560[9,10]	560[10]	140	1,300,000	950	255	130[14]	65[14]	1,100,000	675	975	1,100,000
16F-V2	HF/HF	1600	800	500[10]	375[10]	155	1,400,000	1250	375	135	70	1,300,000	875	1300	1,300,000
16F-V3	DF/DF	1600	800	560[9,10]	560[10]	190	1,500,000	1450	560	165	85	1,500,000	950	1550	1,500,000
16F-V4[7]	DF/WW	1600	800	650	560[10]	90[10]	1,500,000	900	255	130[14]	65[14]	1,300,000	650	600	1,300,000
16F-V5[7]	DF/DF	1600	800	650	560[10]	90[10]	1,600,000	1000	470	135	70	1,500,000	750	875	1,500,000
16F-V6[8]	DF/DF	1600	1600	560[9,10]	560[10]	190	1,500,000	1450	560	165	85	1,400,000	950	1550	1,400,000
16F-V7[8]	HF/HF	1600	1600	375[10]	375[10]	155	1,400,000	1200	375	135	70	1,300,000	850	1350	1,300,000
20F-V1	DF/WW	2000	1000	650	560[10]	140	1,400,000	1000	255	130[14]	65[14]	1,200,000	750	1000	1,200,000
20F-V2	HF/HF	2000	1000	500[10]	375[10]	155	1,500,000	1200	375	135	70	1,400,000	950	1350	1,400,000
20F-V3	DF/DF	2000	1000	650	560[10]	190	1,600,000	1450	560	165	85	1,500,000	1000	1550	1,500,000
20F-V4	DF/DF	2000	1000	590[9,10]	560[10]	190	1,600,000	1450	560	165	85	1,600,000	1000	1550	1,600,000
20F-V5[7]	DF/WW	2000	1000	650	560[10]	90[10]	1,600,000	1000	255	135[14]	70[14]	1,300,000	750	725	1,300,000
20F-V7[8]	DF/DF	2000	2000	650	650	190	1,600,000	1450	560	165	85	1,600,000	1000	1600	1,600,000
20F-V8[8]	DF/DF	2000	2000	590[9,10]	590[9,10]	190	1,700,000	1450	560	165	85	1,600,000	1000	1600	1,600,000
20F-V9[8]	HF/HF	2000	2000	500[10]	500[10]	155	1,500,000	1400	375	135	70	1,400,000	975	1400	1,400,000
20F-V12	AC/AC	2000	1000	560	560	190	1,500,000	1200	470	165	80	1,400,000	900	1500	1,400,000
22F-V1	DF/WW	2200	1100	650	560[10]	140	1,600,000	1050	255	130[14]	65[14]	1,300,000	850	1100	1,300,000
22F-V3	DF/DF	2200	1100	650	560[10]	190	1,700,000	1450	560	165	85	1,600,000	1050	1500	1,600,000
22F-V8[8]	DF/DF	2200	2200	590[9,10]	590[9,10]	190	1,700,000	1450	560	165	85	1,600,000	1050	1650	1,600,000
22F-V10	DF/DFS	2200	1100	650	560[10]	190	1,600,000	1600	500	165	85	1,300,000	1000	1400	1,300,000
24F-V1	DF/WW	2400	1200	650	650	140	1,700,000	1250	255	135[14]	70[14]	1,400,000	1000	1300	1,400,000
24F-V2	HF/HF	2400	1200	500[10]	500[10]	155	1,500,000	1250	375	135	70	1,400,000	950	1300	1,400,000
24F-V4	DF/DF	2400	1200	650	650	190	1,800,000	1500	560	165	85	1,600,000	1150	1650	1,600,000
24F-V5	DF/HF	2400	1200	650	650	155	1,700,000	1350	375	140	70	1,500,000	1100	1450	1,500,000
24F-V8[8]	DF/DF	2400	2400	650	650	190	1,800,000	1450	560	165	85	1,600,000	1100	1650	1,600,000
24F-V10[8]	DF/HF	2400	2400	650	650	155	1,800,000	1400	375	140	70	1,600,000	1150	1600	1,600,000
24F-V11	DF/DFS	2400	1200	650	560[10]	190	1,700,000	1600	500	165	85	1,400,000	1150	1700	1,400,000

Section 2

Bridge Materials

Topic 2.1 Timber

2.1.1 Introduction

Approximately 7% of the bridges listed in the National Bridge Inventory (NBI) are classified as timber bridges. Another 7% of the total have a timber deck supported by a steel superstructure. Many of these bridges are very old, but the use of timber structures is gaining new popularity with the use of engineered wood products. (see Figure 2.1.1). To preserve and maintain them, it is important that the bridge inspector understand the basic characteristics of wood. Timber Bridges Design, Construction, Inspection and Maintenance August 1992 manual published by the United States Department of Agriculture, Forest Service is an excellent reference to supplement timber information in this manual. To order call (304) 285-1591. The National Wood in Transportation website is www.fs.fed.us/na/wit.

Figure 2.1.1 Glued-laminated Modern Timber Bridge

2.1.2

Basic Shapes Used in Bridge Construction

Depending on the required structural capacities and geometric constraints, wood can be cut into various shapes.

Round

Because sawmills were not created yet, most early timber bridge members were made out of solid round logs. Logs were generally used as beams, or stacked and used as abutments and foundations. In some parks, log bridges can still be seen. Round timber members have been used as piles driven into the ground or waterway bed. Logs have also been used as retaining devices for embankment material.

Rectangular

Once sawmill operations gained prominence, rectangular timber members became commonplace. Rectangular timber members were easier to connect together due to the flat sides and can be used for decking, superstructure beams, arches and truss elements, curbs or railings, and retaining devices (see Figure 2.1.2).

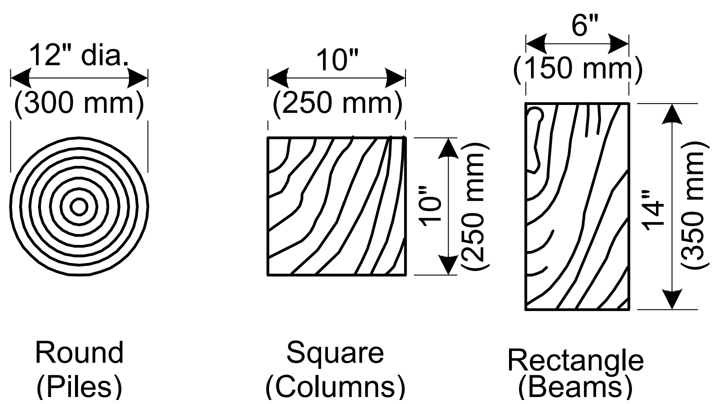


Figure 2.1.2 Timber Shapes

Built-up Shapes

Modern timber bridge members are fabricated from basic rectangular shapes to create built-up shapes, which perform at high capacities. A fundamental example of this is the slab-shaped beam. Two other common examples are T-shaped and box-shaped beams (see Figure 2.1.3). Using glue-laminate technology and stress timber design, these shapes enable modern timber bridges to carry current legal loads.

Refer to Section 6 for further information on timber superstructures and Topic 5.1 for timber decks.

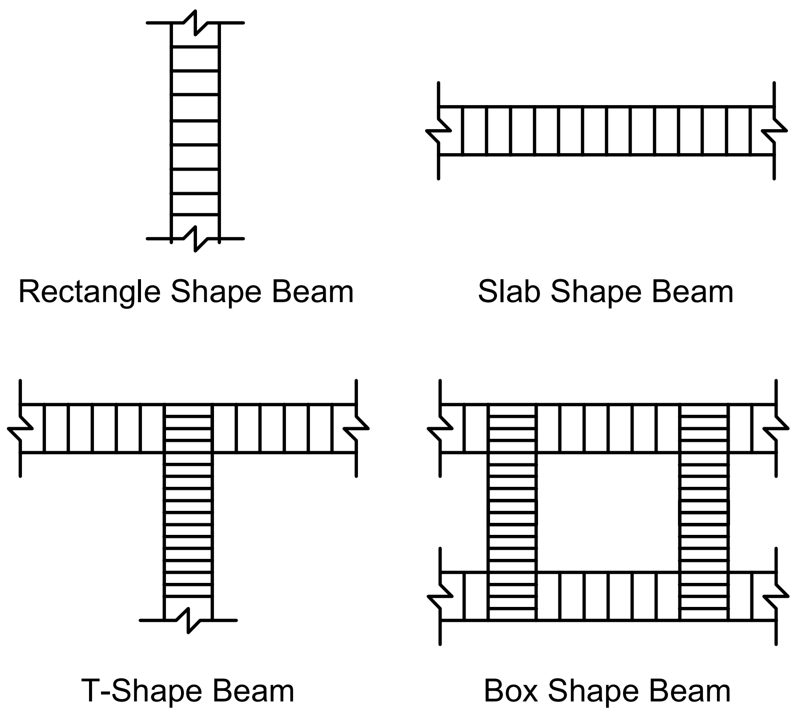


Figure 2.1.3 Built-up Timber Shapes

2.1.3 Properties of Timber

Because of its physical characteristics, wood is in many ways an excellent engineering material for use in bridges. Perhaps foremost is that it is a renewable resource. In addition, wood is:

- Strong, with a high strength to weight ratio
- Economical
- Aesthetically pleasing
- Readily available in many locations
- Easy to fabricate and construct
- Resistant to deicing agents
- Resistant to damage from freezing and thawing
- Able to sustain overloads for short periods of time (shock resistant)

However, wood also has some negative properties:

- Excessive creep under sustained loads
- Vulnerable to insect attack
- Vulnerable to fire

These characteristics stem from the unique physical and mechanical properties of wood, which vary with the species and grade of the timber.

Physical Properties

There are four basic physical properties that define timber behavior. These properties are classification, anatomy, growth features, and moisture content.

Timber Classification

Wood may be classified as hardwood or softwood. Hardwoods have broad leaves and lose their leaves at the end of each growing season. Softwoods, or conifers, have needle-like or scale-like leaves and are evergreens. The terms "hardwood" and "softwood" are misleading because they do not necessarily indicate the hardness or softness of the wood. Some hardwoods are softer than certain softwoods and vice versa.

Timber Anatomy

Wood is a non-homogeneous material. Wood, although an extremely complex organic material, has dominant and fundamental patterns to its cell structure. Some of the physical properties of this cell structure include (see Figure 2.1.4):

- Hollow cell composition - cell walls consist of cellulose and lignin, and are formed in an oval or rectangular shape which accounts for the high strength-to-weight ratio of wood; wood with thick cell walls is dense and strong; lignin bonds the cells together
- Growth rings - revealed in the cross section of a tree; they are distinct annual rings of wood, denser toward the end of each session, sometimes darker in color in that part of each ring (as in Douglas fir and southern pine), sometimes with little color difference (spruces and true firs); depends on species
- Sapwood - the active, outer part of the tree that conducts sap and stores food throughout the tree; is generally permeable and easier to treat with preservatives; sapwood is of lighter color than heartwood
- Heartwood - the inactive, inner part of the tree which serves to support the tree; may be resistant to decay due to toxic materials deposited in the heartwood cells; usually of darker color than sapwood
- Wood rays - groups of cells, running from the center of the tree horizontally to the bark, which are responsible for cross grain strength
- Grain - the wood fibers oriented along the long axis of logs and timbers; the direction of greatest strength

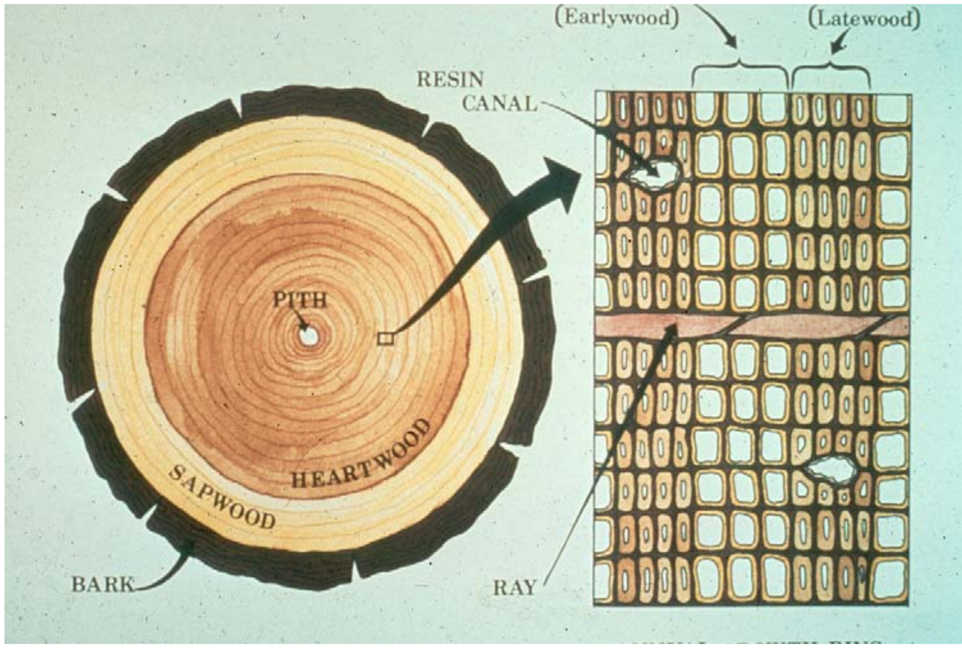


Figure 2.1.4 Anatomy of Timber

Growth Features

A variety of growth features adversely affect the strength of wood. Some of these features include:

- Knots and knot holes - due to intergrown limbs and associated grain deviation
- Sloping grain - caused by the normal taper of a tree or by sawing in a direction other than parallel to the grain
- Splits, checks, and shakes - separation of the cells along the grain, primarily due to rapid or uneven drying and differential shrinkage in the radial and tangential directions during seasoning; checks and splits occur across the growth rings; a shake is a type of check which occurs between the growth rings, peculiar to a few species
- Reaction wood - a type of abnormal wood that is formed in leaning trees; the pith is off center; the wood is gelatinous and displays cross grain shrinkage checks when seasoned

Moisture Content

Moisture content affects wood. It causes dimensional instability and fluctuations of weight and affects the strength and decay resistance of wood. It is most desirable for wood to have the least moisture content as is possible. This is done naturally over time (seasoning) or using kiln drying.

Mechanical Properties

In addition to the physical properties of timber, there are also four important mechanical properties which govern the use of timber in structures.

Orthotropic Behavior

Wood is considered a non-homogeneous and an orthotropic material. It is non-homogeneous because of the random occurrences of knots, splits, checks, and the variance in cell size and shape. It is orthotropic because wood has mechanical properties that are unique to its three principal axes of anatomical symmetry (longitudinal, radial, and tangential). This orthotropic behavior is due to the orientation of the cell fibers in wood (see Figure 2.1.5).

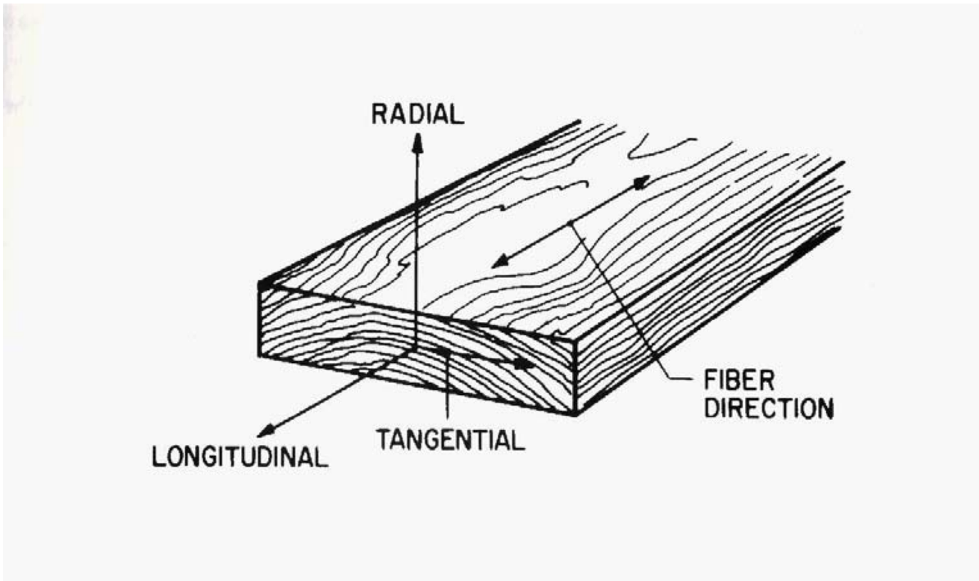


Figure 2.1.5 Three Principal Axes of Wood

As a result of its orthotropy, wood has three distinct sets of strength properties. Because timber members are longitudinal sections of wood, strength properties are commonly defined for the longitudinal axis. However, an exception is bearing strength perpendicular to the grain. American Society for Testing and Materials (ASTM) standards are issued which present strength properties for various types of wood.

Fatigue Characteristics

Because wood is a fibrous material, it tends to be less sensitive than steel or iron to repeated loads. Therefore, it is somewhat fatigue resistant. The presence of knots and sloping grain reduces the strength of wood considerably more than does fatigue; therefore, fatigue is generally not a limiting factor in timber design.

Impact Resistance

Wood is able to sustain short-term loads of about twice the level it can bear on a permanent basis, provided the cumulative duration of such loads is limited.

Creep Characteristics

Creep occurs when a load is maintained on wood. That is, the initial deflection of the member increases with time. Green timbers may sag appreciably, if allowed to season under load. Initial deflection of unseasoned wood under permanent loading can be expected to double with the passage of time. Therefore, to accommodate creep, twice the initial elastic deformation is often assumed for design. Partially seasoned material may also creep to some extent. However, thoroughly seasoned wood members will exhibit little permanent increase in deflection with time.

2.1.4 Timber Grading

The most widely used species of wood for bridge construction are Douglas fir and southern pines. The southern pines include several species graded and marketed under identical grading rules. Other species, such as western hemlock and eastern spruce, are suitable for bridge construction if appropriate allowable stresses are used. Some hardwoods are also used for bridge construction.

Timber is given a grading so that the following can be established:

- Modulus of elasticity
- Tensile stress parallel to grain
- Compressive stress parallel to grain
- Compressive stress perpendicular to grain
- Shear stress parallel to grain (horizontal shear)
- Bending stress

Timber used for outdoor applications needs to be designed for wet service condition. Refer to *Timber Bridges: Design, Construction, Inspection, and Maintenance*, Forest Service, United States Department of Agriculture.

The ultimate strength properties of wood in the tables at the beginning of this topic are for air-dried wood, which is clear, straight grained, and free of strength-reducing defects. Reduction factors need to be applied to these values based on use.

Preservative treatment for decay resistance does not alter the allowable stresses for design, provided any moisture associated with the treatment process is removed.

Unlike steel, the elastic modulus of wood varies with the grades and species.

Sawn Lumber

The grading of sawn timber is accomplished by either a visual grading or a mechanical stress grading (MSR). Refer to the tables at the beginning of this topic.

Visual Grading

This type of grading is the most common and is performed by a certified lumber grader. The lumber grader inspects each sawn and surfaced piece of lumber. The individual pieces of lumber must meet particular grade description requirements in order to be classified at a certain grade. If the requirements are not met, the piece of sawn and surfaced lumber is compared to lower grade description requirements until the piece of lumber fits into the appropriate grade. Mechanical properties are predetermined for each grade. Therefore, once the piece of lumber has been graded, the mechanical properties are known.

Mechanical Stress Grading

Mechanical stress grading or mechanical stress rating (MSR) grades lumber by the relationship between the modulus of elasticity and the bending strength of lumber. The machine measures the bending strength and then assigns an elastic modulus. The grading mainly depends on the elastic modulus but can be changed by visual observance of edge knots, checks, shakes, splits, and warps. Mechanical stress grading has a different set of grading symbols than visual grading.

Glued-Laminated Lumber

Glued-laminated lumber or glulam grades use a combination symbol that describes the combination of lamination grades. Bending and axial are the two types of combination grades. A glulam member will be graded under the bending combination if it is designed for use as a flexure member (see the tables at the beginning of this topic), or it will be graded under the axial combination if it is designed for use as an axial loaded member.

2.1.5 Types and Causes of Timber Deterioration

Although wood is an excellent material for use in bridges, untreated wood is vulnerable to damage from fungi, parasites, and other sources. The untreated inner cores of treated timbers and poles are vulnerable to these predators if they can gain access through the outer treated shell. The degree of vulnerability varies with the species and grade of the timber. Bridge inspectors must be able to recognize the signs of the various types of damage and be able to evaluate their effect on the structure.

Natural Defects

Defects that form from abnormal growth or from the lumber drying process include (see Figure 2.1.6):

- Checks - separations of the wood fibers, normally occurring across or through the annual growth rings, and generally parallel to the grain direction

- Splits - similar to checks except the separations of the wood fibers extend completely through the piece of wood; a split is also known as a through check

- Shakes - separations along the grain which occur between the annual growth rings

These three defects provide openings for decay to begin and in some cases indicate reduced strength in the member when the defect is in an advanced state.

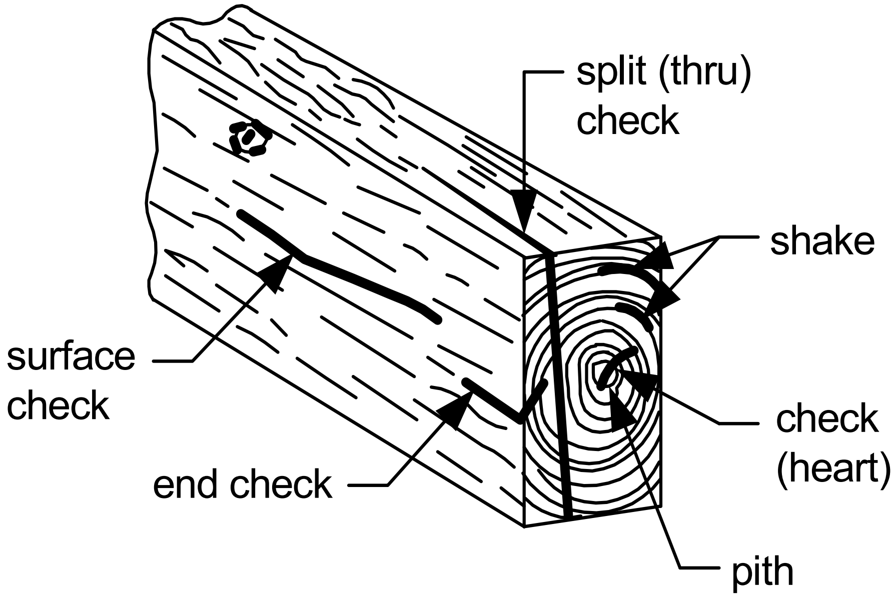


Figure 2.1.6 Natural Timber Defects

Fungi

Decay is the primary cause of timber bridge replacement. Decay is the process of living fungi, which are plants feeding on the cell walls of wood (see Figure 2.1.7). The initial process is started by the deposition of spores or microscopic seeds. Fruiting bodies (e.g., mushrooms and conks) produce these spores by the billions. The spores are distributed by wind, water, or insects.

Figure 2.1.7 Decay of Wood by Fungi

Spores that survive and experience favorable growth conditions can penetrate timber bridge members in a few weeks. Favorable conditions for fungi to grow can only occur when these four requirements exist:

- Oxygen - Sufficient oxygen must be available for the fungi to breathe. A minimal amount of free oxygen can sustain them in a dormant state, but at least 20 percent of the volume of wood must be occupied by air for fungi to become active. The air we breathe contains about 21 percent oxygen. Absence of oxygen in bridge members would only occur in piling or bents placed below the permanent low water elevation or water table, or buried in the ground.

- Temperature - A favorable temperature range must be available for the growth of fungi to occur. Below 0ºC (32ºF), the fungi become dormant but resumes its growth as the temperature rises above freezing to the 24ºC to 29ºC (75ºF to 85ºF) range, where growth is at its maximum. Above 32ºC (90ºF), growth tapers off rapidly, and temperatures in excess of 49ºC (120ºF) become lethal to the fungi. These killing temperatures could only occur in bridge members during kiln drying or preservative treating.

- Food - An adequate food supply must be available for the fungus to feed on. As the entire bridge serves as the food supply, the only prevention is to poison the wood supply with preservatives.

- Moisture - The fourth and probably the most controlling requirement is an adequate supply of moisture. The term "dry-rot" is misleading because dry wood will not rot. Wood must have a moisture content of 20 percent or greater for the growth of fungi to become active. Rain or snow is the main source of wood wetting. Secondary sources are condensation, ground water, and stream water. Exposed surfaces allow moisture to evaporate harmlessly. However seasoning checks, interfaces between timber members, and fastener holes are ideal for localized moisture accumulation which allow fungi to grow.

Although there are numerous types and species of fungi, only a few cause decay in timber bridge members. Some fungi types that do not cause damage include:

- Molds - cottony or powdery circular growths varying from white or light colors to black; molds themselves do not cause decay but their presence is an indication that conditions favorable to the growth of fungi exist (see Figure 2.1.8)
- Stains - specks, spots, streaks, or patches, varying in color, which penetrate the sap wood; sapstain is harmless to wood; it is usually a surface phenomenon and, like molds, implies conditions where harmful fungi can flourish
- Soft rot - attacks the wood, making it soft and spongy; only the surface wood is affected, and thus it does not significantly weaken the member; occurs mostly in wood of high water content and high nitrogen content

Some fungi types that weaken the wood include:

- Brown rot - degrades the cellulose and hemi-cellulose leaving the lignin as a framework which makes the wood dark brown and crumbly (see Figure 2.1.9)
- White rot - feeds upon the cellulose, hemi-cellulose, and the lignin and makes the wood white and stringy (see Figure 2.1.9)

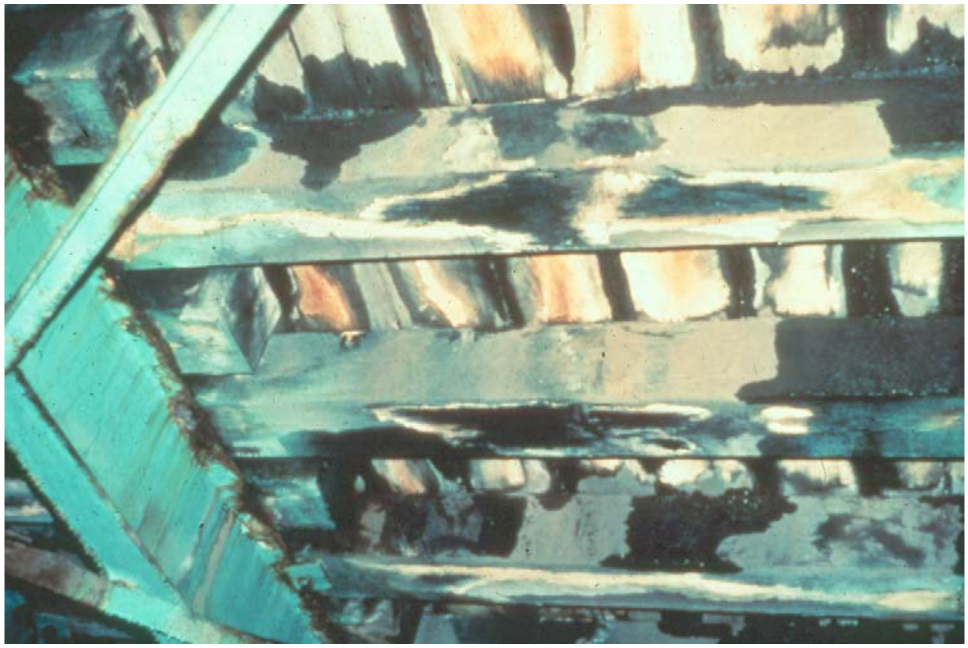


Figure 2.1.8 Mold and Stain on Underside of Timber Bridge

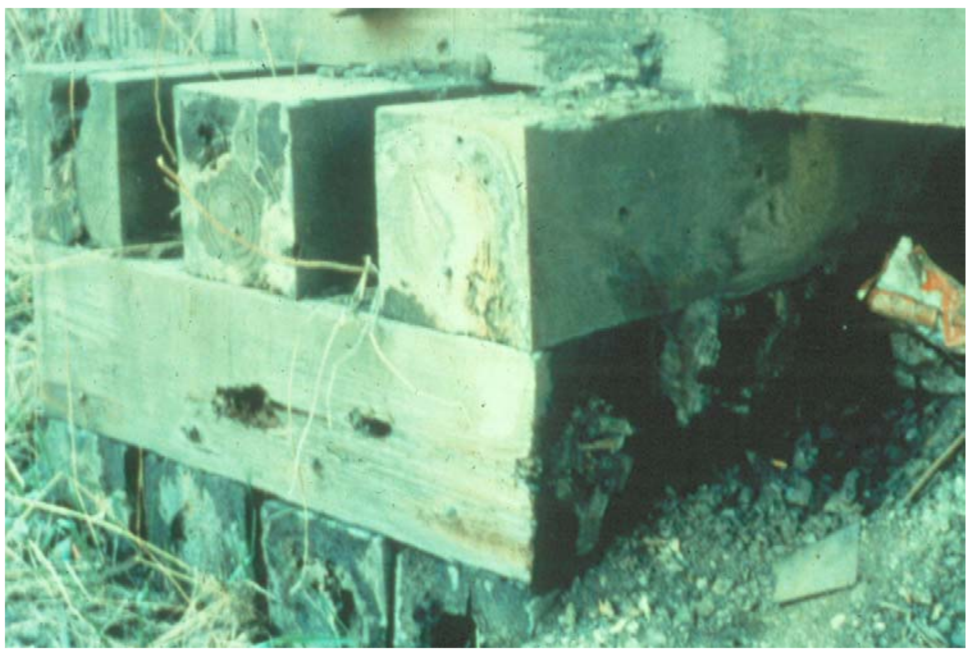


Figure 2.1.9 Brown and White Rot

Brown and white rots are responsible for structural damage to wood.

The natural decay resistance of wood exposed under conditions favorable for decay is distinctly variable, and it can be an important factor in the service life of wood bridges.

The heartwood of many tree species possesses a considerable degree of natural decay resistance, while the sapwood of all commercial species is vulnerable to decay.

Each year, when an inner layer or ring of sapwood dies and becomes heartwood, fungi-toxic compounds are deposited. These compounds provide natural decay resistance and are not present in living sapwood.

Most existing wood bridges in this country have been constructed from either Douglas fir or southern pine. Older bridges may contain such additional species as larch, various pines, and red oak. The above named species are classified as moderately decay resistant. Western red cedar and white oak are considered very decay resistant.

In the last 25 years, wood bridge materials have been obtained increasingly from smaller trees in young-growth timber stands. As a result, recent supplies of lumber and timbers have contained increased percentages of decay-susceptible sapwood.

Insects

Insects tunnel in and hollow out the insides of timber members for food and shelter. Some common types of insects include:

- Termites
- Carpenter ants
- Powder-post beetles or lyctus beetles
- Caddisflies

Termites

Termites are pale-colored, soft-bodied insects that feed on wood (see Figure 2.1.10). All damage is inside the surface of the wood; hence, it is not visible. The only visible signs of infestation are white mud shelter tubes or runways extending up from the earth to the wood and on the sides of masonry substructures. Termite attack of bridge members, however, is rare or nonexistent in bridges throughout most of the country due to the constant vibration caused by traffic travelling over timber bridges.

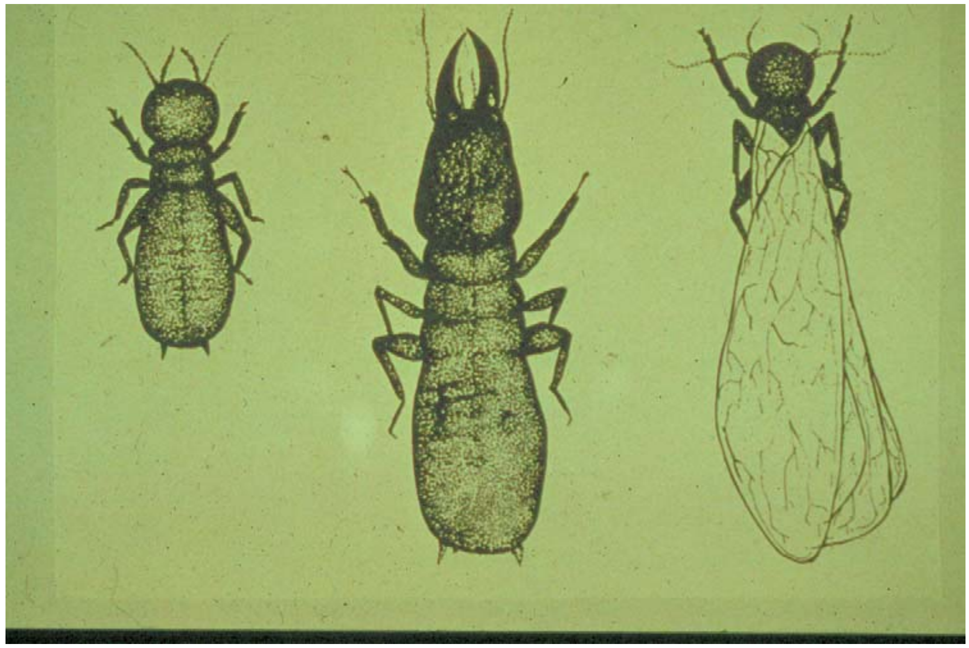


Figure 2.1.10 Termites

Carpenter Ants

Carpenter ants are large, black ants up to 3/4 inches (19 mm) long that gnaw galleries in soft or decayed wood (see Figure 2.1.11). The ants may be seen in the vicinity of the infested wood, but the accumulation of sawdust on the ground at the base of the timber is also an indicator of their presence. The ants do not use the wood for food but build their galleries in the moist and soft or partially decayed wood.

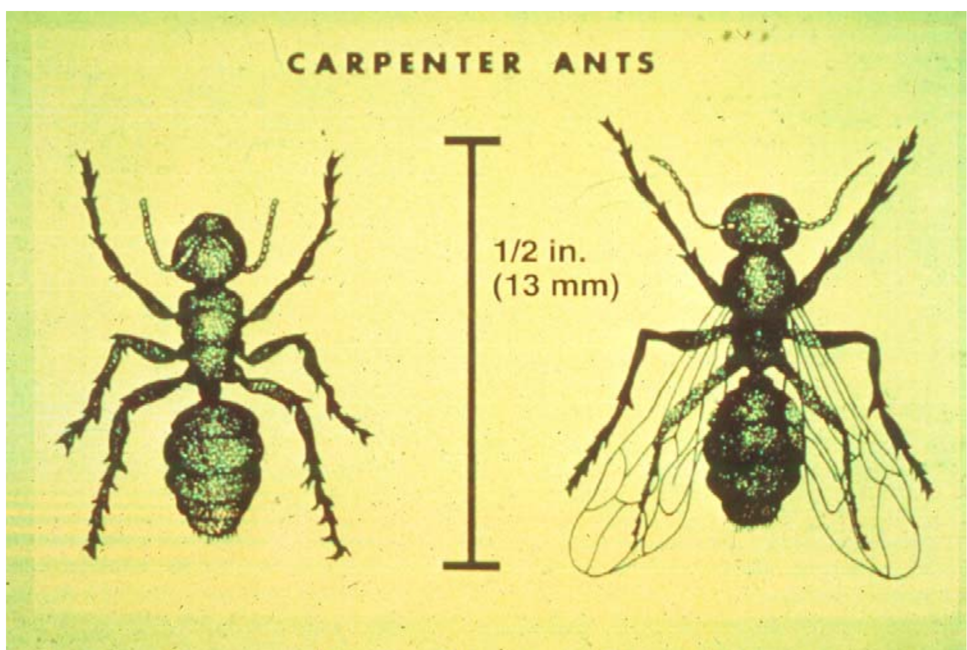


Figure 2.1.11 Carpenter Ants

Powder-post Beetles or Lyctus Beetles

Powder-post beetle larvae also hollow out the insides of timber members and leave the outer surface pocked with small holes. Often a powdery dust is dislodged from the holes. The inside may be completely excavated as the larvae of these beetles bore through the wood for food and shelter.

Caddisflies

The caddisfly is another insect that can damage timber piles. It is generally found in fresh water but can also be found in brackish water. Bacterial and fungal decay make the timber attractive to the caddisfly.

The caddisfly is an aquatic insect that is closely related to the moth and butterfly (see Figure 2.1.12). During the larva and pupa stage of their life cycle, they can dig small holes in the timber for protection. The larvae do not feed on the timber, but rather use it as a foundation for their silken shelters. This explains why caddisfly larvae have been known to exist on creosote treated timber.

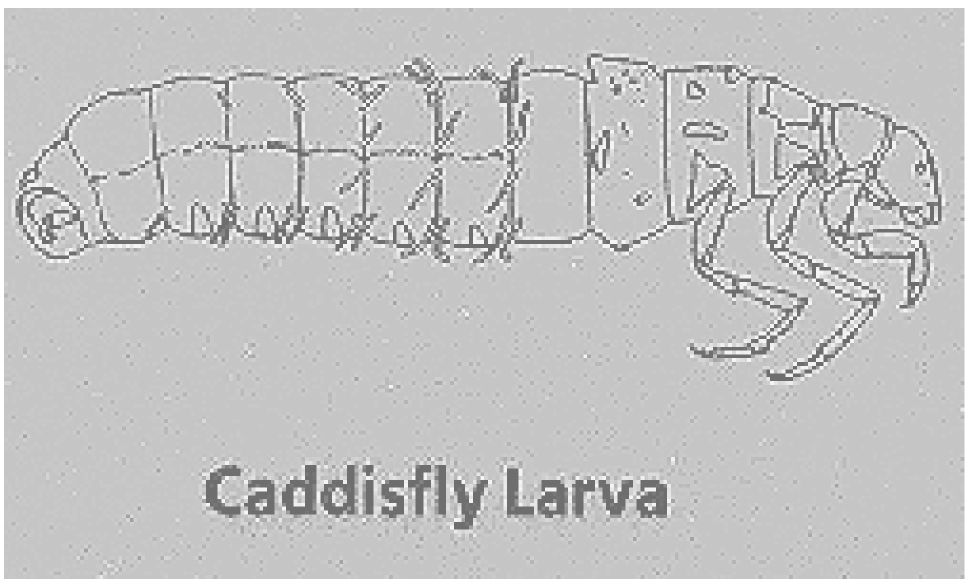


Figure 2.1.12 Caddisfly Larva

Marine Borers

Marine borers are found in sea water and brackish water only and cause severe damage to timber members in the area between high and low water, although damage may extend to the mud line (see Figure 2.1.13). They can be very destructive to wood and have been known to consume piles and framing in just a few months.

One type of marine borer is the mollusk borer, or shipworm (see Figure 2.1.14). The shipworm is one of the most serious enemies of marine timber installations. The most common species of shipworm is the teredo. This shipworm enters the timber in an early stage of life and remains there for the rest of its life. Teredos are gray and slimy and can typically reach a length of 380 mm (15 inches) and a diameter of 10 mm (3/8 inch). Some species of shipworm have been known to grow to a length of 1.8 m (6 feet) and up to 25 mm (1 inch) in diameter. The teredo maintains a small opening in the surface of the wood to obtain nourishment from the sea water.

Figure 2.1.13 Marine Borer Damage to Wood Piling

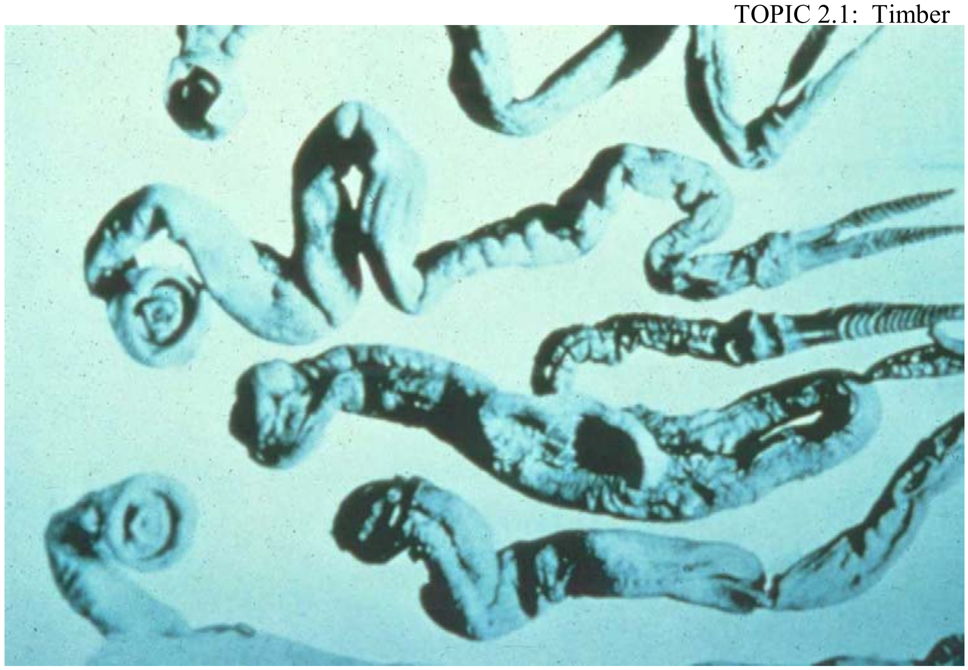


Figure 2.1.14 Shipworms (Mollusks)

Another type of marine borer is the crustacean borer. The most commonly encountered crustacean borer is the limnoria or wood louse (see Figure 2.1.15). It bores into the surface of the wood to a shallow depth. Wave action or floating debris breaks down the thin shell of timber outside the borers' burrows, causing the limnoria to burrow deeper. The continuous burrowing results in a progressive deterioration of the timber pile cross section, which will be noticeable by an hourglass shape developed between the tide levels. These borers are about 3 to 6 mm (1/8 to 1/4 inches) long and 2 to 3 mm (1/16 to 1/8 inches) wide.

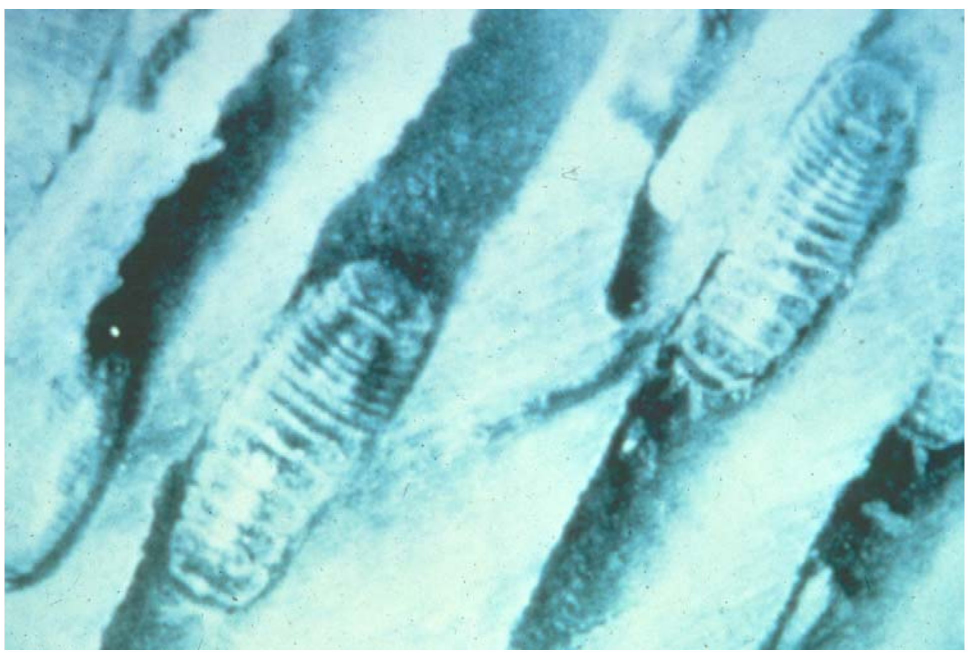


Figure 2.1.15 Limnoria Burrowing in Wood

Chemical Attack

Most petroleum based products and chemicals do not cause structural degradation to wood. However, animal waste can cause some damage, and strong alkalis will destroy wood fairly rapidly. Highway bridges are seldom exposed to these substances. Timber structures normally do not come in contact with damaging chemicals unless an accidental spill occurs.

Acids

Wood resists the effects of certain acids better than many materials and is often used for acid storage tanks. However, strong acids that have oxidizing properties, such as sulphuric and sulphureous acid, are able to slowly remove a timber structure's fiber by attacking the cellulose and hemi-cellulose. Acid damaged wood has weight and strength losses and looks as if it has been burned by fire.

Bases or Alkalis

Strong bases or alkalis attack and weaken the hemi-cellulose and lignin in the timber structure. Attack by strong bases leaves the wood a bleached white color. Mild alkalis do little harm to wood.

Other Types and Sources of Deterioration

Delaminations

Delaminations occur in glued-laminated members when the layers separate due to failure within the adhesive or at the bond between the adhesive and the laminate. They provide openings for decay to begin and may cause a reduction in strength (see Figure 2.1.16).

Loose connections

Loose connections may be due to shrinkage of the wood, crushing of the wood around the fastener, or from repetitive impact loading (working) of the connection. Loose connections can reduce the bridge's load-carrying capacity (see Figure 2.1.17).

Surface depressions

Surface depressions indicate internal collapse, which could be caused by decay.

Fire

Fire consumes wood at a rate of about 0.05 inches (1 mm) per minute during the first 30 minutes of exposure, and 0.021 inches (0.5 mm) per minute thereafter (see Figure 2.1.18). Large timbers build a protective coating of char (carbon) after the first 30 minutes of exposure. Small size timbers do not have enough volume to do this before they are, for all practical purposes, consumed. Preservative treatments are available to retard fire damage.

Figure 2.1.16 Delamination in a Laminated Timber Member

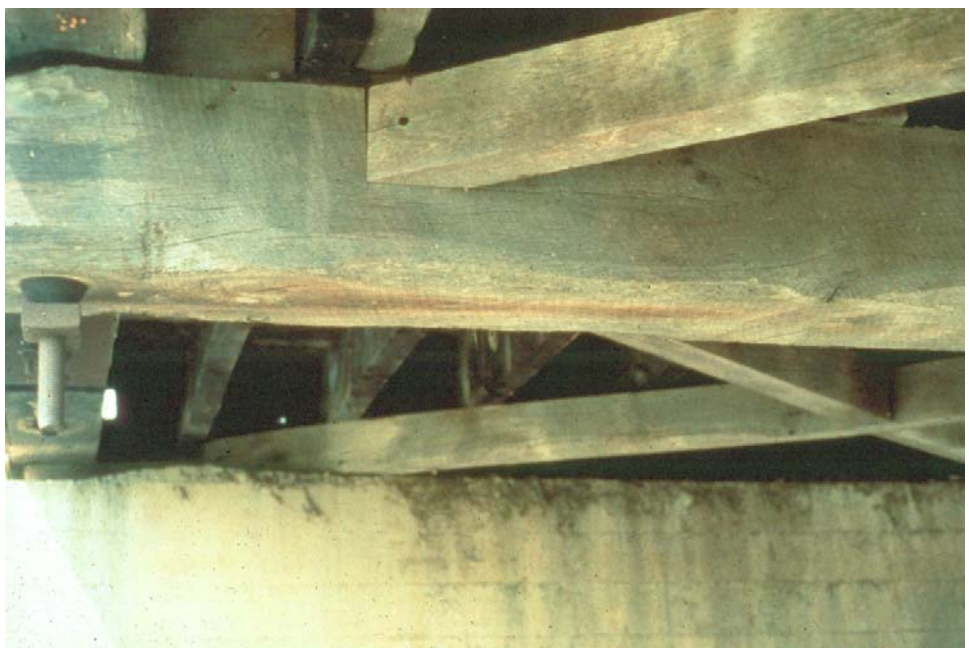


Figure 2.1.17 Hanger Connection on a Timber Floorbeam

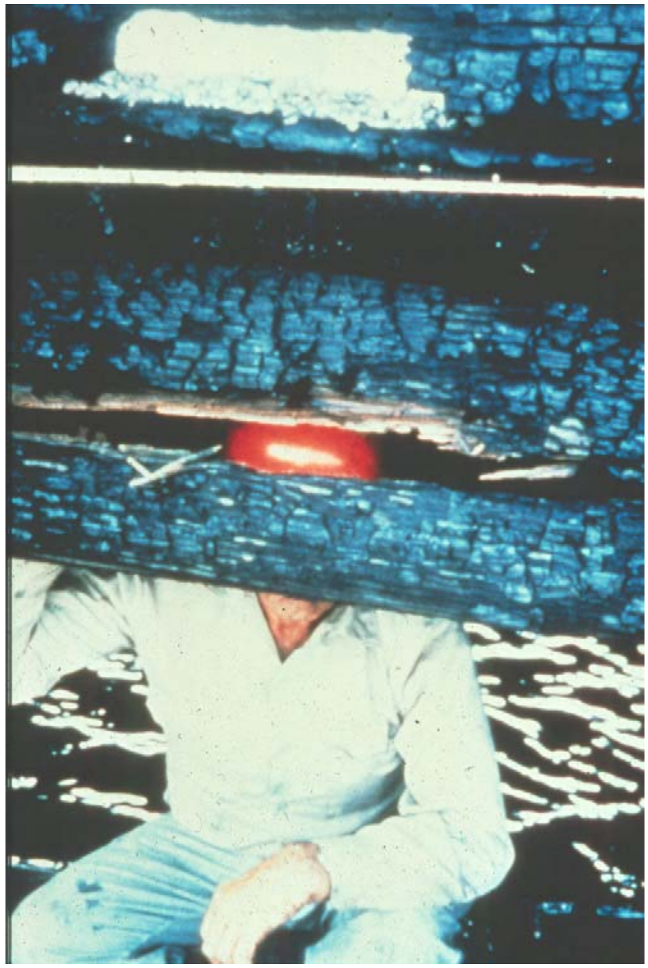


Figure 2.1.18 Fire Damaged Timber Member

Impact or Collisions

Severe damage can occur to truss members, railings, and columns when an errant vehicle strikes them (see Figure 2.1.19).

Figure 2.1.19 Impact/Collision Damage to a Timber Member

Abrasion or Mechanical Wear

Vehicular traffic is the main source of abrasion on timber decks (see Figure 2.1.20). Abrasion also occurs on timber piles that are subjected to tidal flows. Mechanical wear of timber members sometimes occurs due to movement of the fasteners against their holes when connections become loose.

Figure 2.1.20 Abrasion Damage on a Timber Deck

Overstress

Each timber member has a certain ultimate load capacity. If this load capacity is exceeded, the member will fail (see Figures 2.1.21 and 2.1.22).

Figure 2.1.21 Horizontal Shear Failure in Timber Member

Figure 2.1.22 Failed Timber Floor Beam

Weathering or Warping

Weathering is the affect of light, water, and heat. Weathering can change the equilibrium moisture content in the wood in a non-uniform fashion, thereby resulting in changes in the strength and dimensions of the wood. Uneven reduction in moisture content causes localized shrinkage, which can lead to warping, checking, splitting, or loosening of connectors (see Figure 2.1.23).

Figure 2.1.23 Weathering on Timber Deck

Protective Coating Failure

The following paint failures are common on wood:

- Cracking and peeling extend with the grain of the wood. They are caused by different shrink and swell rates of expansion and contraction between springwood (the lighter colored, wider spaces between the "rings" which grow in springtime) and denser summerwood (the darker, narrower rings which grow in hotter, drier summer).
- Decay fungi penetrate through cracks in the paint to cause wood to decay.
- Blistering is caused by paint applied over an improperly cleaned surface. Water, oil, or grease typically are responsible for blistering.
- Chalking is a degradation of the paint, usually by the ultraviolet rays of sunlight, leaving a powdery residue.
- Erosion is general thinning of the paint due to chalking, weathering, or abrasion.
- Mold fungi and stain fungi grow on the surface of paint, usually in warm, humid, shaded areas with low air flow. They appear as small green or black spots.

2.1.6 Protective Systems

Protective systems are a necessity when using timber for bridge construction. Proper preparation of the timber surface is required for the protective system to penetrate the wood surface and perform adequately. Untreated timber generally has a unit weight of about 640 to 800 kilograms per cubic meter (40 to 50 pounds per cubic foot (pcf)).

Types and Characteristics of Wood Protectants

Water Repellents

Water repellents prevent water absorption and maintain low moisture content in wood. This helps to prevent decay by molds and to slow the weathering process. Laminated wood (plywood) is particularly susceptible to moisture variations, which cause stress between plies due to swelling and shrinkage.

Preservatives

Wood preservatives prevent biological deterioration that can penetrate deep into timber. To be effective, the preservatives have to be applied to wood by vacuum-pressure treatment. This is done by placing the timber to be treated in a sealed chamber up to 2.4 m (8 feet) in diameter and 43 m (140 feet) long. The chamber is evacuated, drawing the air from the wood pores and cells. The treatment chemical is then fed into the chamber and pressure up to 1380 kPa (200 psi) is applied, forcing the chemical into the wood (see Figure 2.1.24). Preservatives are the best means to prevent decay but do not prevent weathering. A paint or water repellent coating is required for this. Treated timber generally has a unit weight of about 800 kilograms per cubic meter (50 pounds per cubic foot (pcf)).

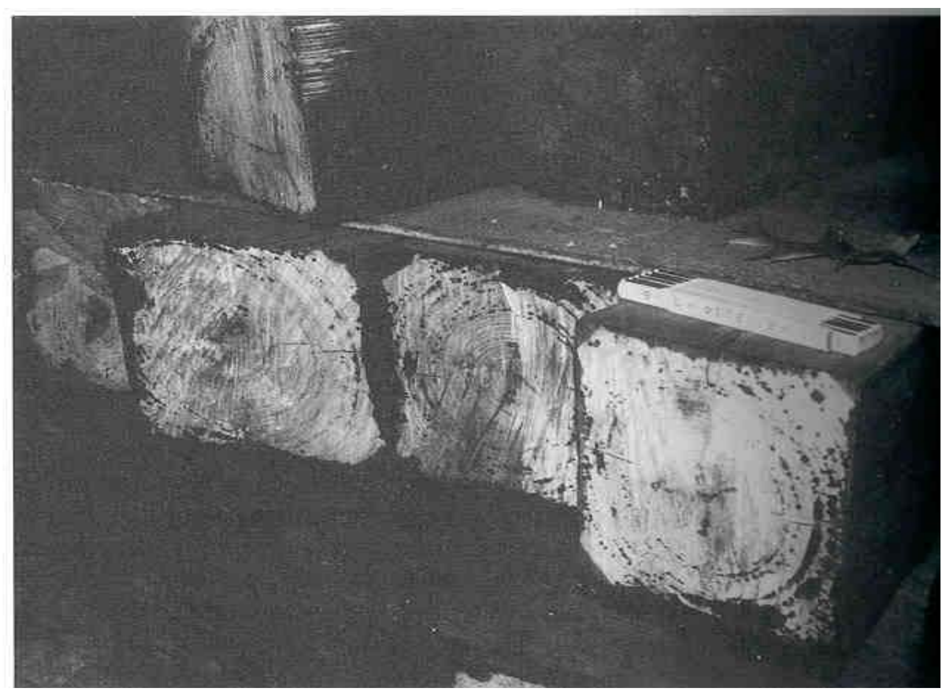


Figure 2.1.24 Bridge Timber Member Showing Penetration Depth of Preservative Treatment

Coal tar-creosote is a dark, oily protectant used in structural timber such as pilings and beams. Coal tar-creosote treated timber has a dark, oily appearance (see Figure 2.1.25). Unless it has weathered for several years, it cannot be painted, since paint adheres poorly to the oily surface, and the oils bleed through paint.

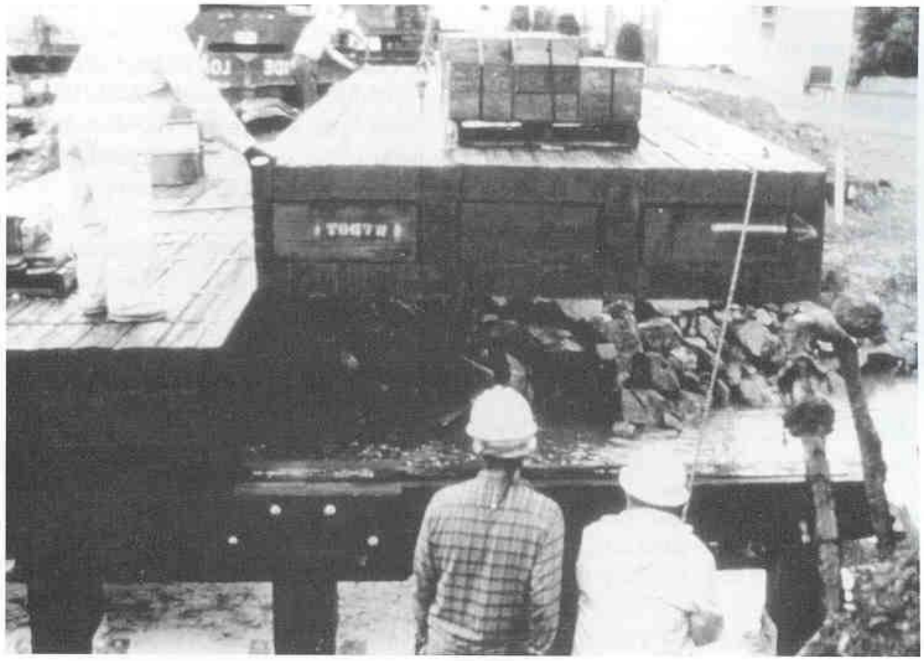


Figure 2.1.25 Coal-Tar Creosote Treated Timber Beams (Source: Barry Dickson, West Virginia University)

Pentachlorophenol (in a light oil solvent) is an organic solvent solution used as a decay inhibitor. It also leaves an oily surface, like creosote, but can be painted after all of the solvent has evaporated, usually in one or two years of normal service.

Chromated copper arsenate (CCA) is the most common waterborne salt decay inhibitor and is also applied by vacuum-pressure treatment. Timber treated with CCA has a green appearance. It is the only pressure-applied preservative that readily accepts painting. CCA also provides limited protection against the ultraviolet rays in sunlight.

Pole-fuming is used to kill decay fungi in timber pilings which are already in-service. The treatment chemical, injected through bore holes drilled into the piling, spreads along wood fibers for up to 2.7 m (9 feet) from the injection site. It stops existing decay and prevents further decay for up to nine years.

Fire Retardants

Fire retardants will not indefinitely prevent wood from burning but will retard the spread of fire and prolong the time to ignite wood. The two main classes of fire retardants are pressure impregnated fire retardant salts and intumescent coatings (paints). The intumescent paints expand upon intense heat exposure, forming a thick, puffy, charred coating which insulates the wood from the intense heat. Application of fire retardants may change some wood properties of glued-laminated timber.

Paint

Wood must be sufficiently dry to permit painting. A few months of seasoning will satisfactorily dry new wood. The wood surface must be free of dirt and debris prior to painting. Old, poorly adherent paint must be removed and the edges of intact paint feathered for a smooth finish. Mildew shows up as green or black spots on bare wood or paint. It is a fungus which typically grows in warm, humid, shaded areas with low air movement. Mildew must be removed with a solution of sodium hypochlorite (bleach) and water.

There are several common methods to prepare wood for painting:

- Hand tool cleaning is the simplest but slowest method. Sandpaper, scrapers, and wire brushes are used to clean small areas.
- Power tool cleaning utilizes powerized versions of the hand tools. They are faster than hand tools, but care must be exercised not to damage the wood substrate.
- Heat application with an electric heat gun softens old paint for easier removal to bare wood.
- Solvent-based and caustic chemical paint removers can efficiently clean large areas quickly. Some of the chemicals may, however, present serious fire or exposure hazards. Extreme caution must be exercised when working around chemical paint removers.
- Open nozzle abrasive blast cleaning and water blast cleaning remove old paint and foreign material, leaving bare wood. However, they can easily damage wood unless used carefully.

Paint protects wood from both moisture and weathering. By precluding moisture from wood, paint prevents decay. However, paint applied over unseasoned wood seals in moisture, accelerating, rather than retarding, decay. Oil-based paint and latex paint are both commonly used on wood bridges.

Oil-based paint provides the best shield from moisture. It is not, however, the most durable. It does not expand and contract as well as latex, and it is more prone to cracking. Oil/alkyd paints cure by air oxidation. These paints are low cost, with good durability, flexibility, and gloss retention. They are resistant to heat and solvents. Alkyd paints often contain lead pigments, known to cause numerous health hazards. The removal and disposal of lead paint is a regulated activity in all states.

Latex paint consists of a latex emulsion in water. Latex paint is often referred to as water-based paint. There are many types of latex paint, each formulated for a different application. They have excellent flexibility and color retention, with good adhesion, hardness, and resistance to chemicals.

2.1.7 Inspection Procedures for Timber

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

- Visual
- Physical
- Advanced inspection techniques

Visual Examination

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

The second type of visual inspection is called a "hands-on" inspection. This type of visual inspection requires the inspector to visually assess all defective timber surfaces at a distance no further than an arm's length. The timber surfaces are given close visual attention to quantify and qualify any defects.

For timber members, visual inspections reveal areas that need further investigation such as checks, splits, shakes, fungus decay, deflection, or loose fasteners.

Physical Examination

Once the defects are identified visually, physical procedures must be used to verify the extent of the defect. Most physical inspection procedures for timber members involve destructive methods. An inspection hammer, on the other hand, does not and can be used to tap on areas and determine the extent of internal decay. This is done by listening to the sound the hammer makes. If it sounds hollow, internal decay may be present. Some methods or areas of physical examination include:

Pick or Penetration Test

A pick or penetration test involves lifting a small sliver of wood with a pick or pocketknife and observing whether or not it splinters or breaks abruptly. Sound wood splinters, while decayed wood breaks abruptly (see Figure 2.1.27).

Figure 2.1.27 Inspector Probing Timber

Timber Boring and Drilling Locations

The following are common timber boring and drilling locations (see Figure 2.1.28):

- Deck planks - in the bottom, next to a beam.
- Beams - in sides near the deck and in the bottom over the bent cap.
- Cap - under the beams and over posts and piles.
- Post/pile - top under cap and bottom just above ground or water line.

An inspector may be required to take samples to determine the condition of the wood. When drilling or boring vertical faces, always drill at a slight upward angle so that any drainage will flow away from the plugged hole.

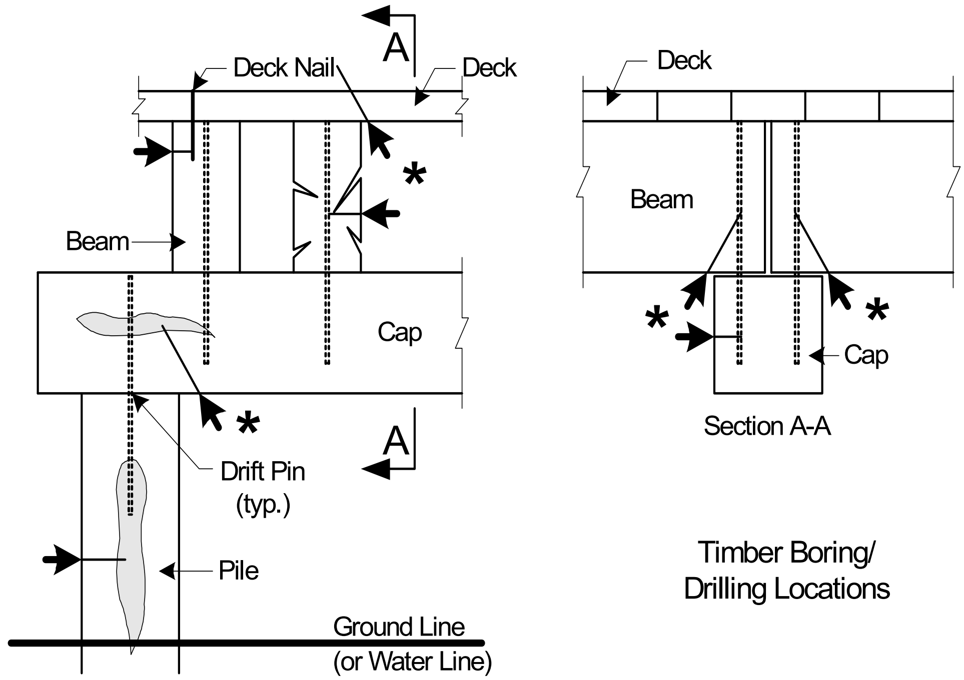


Figure 2.1.28 Timber Boring and Drilling Locations

Protective Coatings

When inspecting timber bridges, keep in mind the environment surrounding the bridge and how this can cause failures leading to rapid decay of the underlying wood members.

Paint Adhesion

Probe the paint with the point of a knife to test paint adhesion to wood. Attempt to lift the paint. Adhesion failure may occur between wood and paint or between layers of paint.

A more quantitative paint adhesion assessment is performed in accordance with American Society for Testing and Materials (ASTM) D-3359 "Measuring Adhesion by Tape Test". An "X" is cut through the paint to the wood surface. Adhesive test tape is applied over the "X" and removed in a continuous motion. The amount of paint (if any) removed is noted. Adhesion is rated on a scale of 0 to 5. Refer to ASTM D-3359 for the rating criteria.

Paint Dry Film Thickness

Paint dry film thickness is measured with a Tooke Gage (see Figure 2.1.26). With this instrument, a groove is cut at a known angle with the grain through the paint to expose the wood substrate. The thickness of each layer of paint is measured through a 50-power microscope built into the gage.

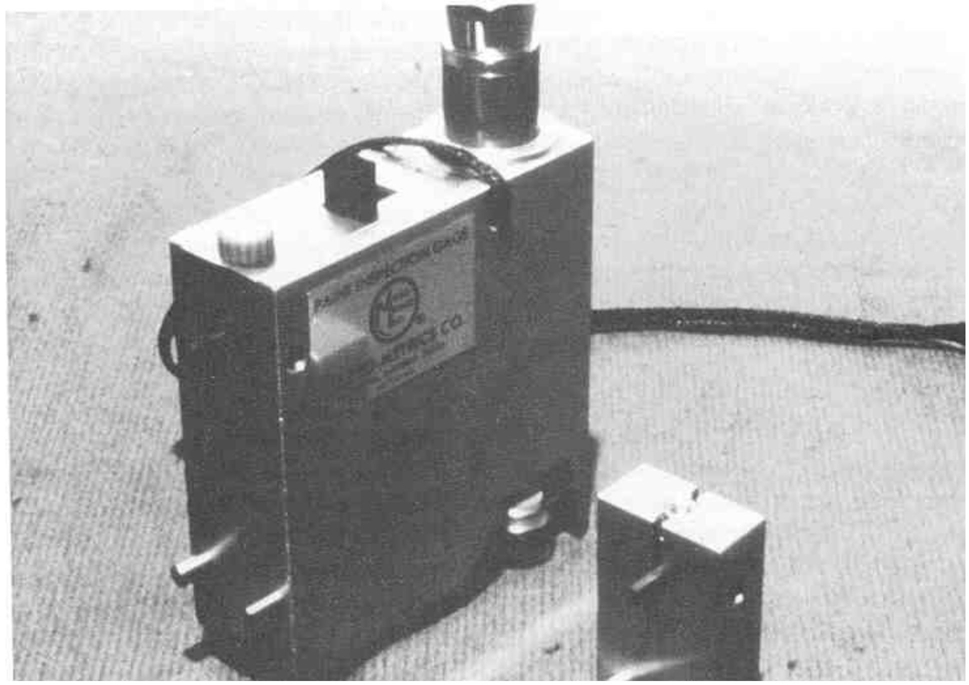


Figure 2.1.29 Tooke Gage Used to Measure Coating Dry Film Thickness

Repainting

If the coating is to be repainted, the type of paint in the existing topcoat must be known, since paints of different type may not adhere well to each other. Two methods to determine the type of existing paint are:

- Check historical records of previous painting
- Obtain paint samples from the bridge for laboratory analysis

Alternately, a test patch may be coated with new paint over intact existing paint. After the paint thoroughly dries in accordance with the manufacturer's specification, inspect the appearance and adhesion of the new paint.

Advanced Inspection Techniques

In addition, several advanced techniques are available for timber inspection. Nondestructive methods, described in Topic 13.1.2, include:

- Pol-Tek
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 13.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Shigometer

Detailed information about timber bridges can be found from the text, Timber Bridges, Design, Construction, Inspection and Maintenance, published by the USDA Forest Service. Latest information about timber bridge technology, including publications, can be obtained from the Forest Service's Timber Bridge Information Resource Center at (304) 285-1591 or at the website at http://www.fs.fed.us/. Information can also be obtained at the FHWA website, which is at TFHRC.dot.gov.

Table of Contents

Topic 2.2 Concrete

2.2.1 Introduction

A large percentage of the bridge structures in the nation's highway network are constructed of reinforced concrete or prestressed concrete. It is important that the bridge inspector understand the basic characteristics of concrete in order to efficiently inspect and evaluate a concrete bridge structure.

Concrete, commonly mislabeled as "cement", is a mixture of various components that, when mixed together in the proper proportions, chemically react to form a strong durable construction material ideally suited for certain bridge components. Cement is only one of the basic ingredients of concrete. It is the "glue" that binds the other components together. Concrete is made up of the following basic ingredients:

- Portland cement
- Water
- Air
- Aggregates
- Admixtures (reducers, plasticizers, retarders)

Portland Cement

The first ingredient, Portland Cement, is one of the most common types of cement, and it is made with the following raw materials:

- Limestone - provides lime
- Quartz or cement rock - provides silica
- Claystone - provides aluminum oxide
- Iron ore - provides iron oxide

The cement is produced by placing the above materials through a three process high temperature kiln system. During the three process kiln system, the temperature can range from 100° C to 1510° C (212° F to 2750° F). The first zone in the kiln process is known as the drying process. During this process, the materials are dehydrated due to the high temperature. The calcining zone is the next step and results in the production of lime and magnesia. The final step, called the burning zone or clinkering zone, produces clinkers or nodules of the sintered materials. Upon cooling, the clinkers are ground into a powder and finish the Portland cement production process.

Water

The second ingredient, water, can be almost any potable water. Impurities in water, such as dissolved chemicals, salt, sugar, or algae, produce a variety of undesirable effects on the quality of the concrete mix. Therefore, water with a noticeable taste or odor may be suspect.

Air

The third ingredient of concrete is air. Small evenly distributed amounts of entrained air provide:

- Increased durability against freeze/thaw effects
- Reduced cracking
- Improved workability
- Reduced water segregation

Air entrainment also reduces the weight of concrete slightly. Many tiny air bubbles introduced into the plastic concrete naturally create lighter weight concrete. The typical air entrainment additive is a vinsol resin. Air entrainment additives act like dishwashing liquids. When mixed with water, they create bubbles. These bubbles become part of the concrete mix, creating tiny air voids. Through extensive lab testing, it has been proven that when exposed to freeze/thaw conditions, the voids prevent excess pressure buildup in the concrete.

Aggregates

The fourth ingredient, aggregates, comprise approximately 75% of a typical concrete mix by volume. Some aggregate qualities which result in a strong and durable concrete are:

- Abrasion resistance
- Weather resistance
- Chemical stability
- Chunky compact shape
- Smooth, non-porous surface texture
- Cleanliness and even gradation

Normal weight concrete has a unit weight of about 2240 to 2400 kg/m^3 (140 to 150 pcf). Typical aggregate materials for normal weight concrete are sand, gravel, crushed stone, and air-cooled, blast-furnace slag.

Lightweight concrete normally has a unit weight of 1200 to 1840 kg/m^3 (75 to 115 pcf). The weight reduction comes from the aggregates and air entrainment. Lightweight aggregates differ depending on the location where the lightweight concrete is being produced. The common factor in lightweight aggregates is that they all have many tiny air voids in them that make them lightweight with a low specific gravity.

Admixtures

The fifth ingredient of most concrete mixes is one or more admixtures to change the consistency, setting time, or concrete strength. Pozzolans are a common type of admixture used to reduce permeability. There are natural pozzolans such as diatomite and pumicite, along with artificial pozzolans which include admixtures such as fly ash.

Admixtures can either be minerals or chemicals. The mineral admixtures include fly ash, silica fume, and ground granulated blast-furnace slag. Chemical admixtures can include water reducers, plasticizers, retarders, high range water reducers, and superplasticizers.

Fly ash is a by-product from the burning of ground or powdered coal. Fly ash was added to concrete mixes as early as the 1930's. This turned out to be a viable way to dispose of fly ash while positively affecting the concrete. The use of fly ash in concrete mixes improves concrete workability, reduces segregation, bleeding, heat evolution and permeability, inhibits alkali-aggregate reaction, and enhances sulfate resistance.

The use of fly ash in concrete mixes also has some drawbacks, however, such as increased set time and reduced rate of strength gain in colder temperatures. Admixture effects are also reduced when fly ash is used in concrete mixes. This means, for example, that a higher percentage of air entrainment admixture is needed for concrete mixes using fly ash.

Silica fume (microsilica) results from the reduction of high purity quartz with coal in electric furnaces while producing silicon and ferrosilicon alloys. It affects concrete by improving compressive strength, bond strength, and abrasion resistance. Microsilica also reduces permeability. Concrete with a low permeability minimizes steel reinforcement corrosion, which is of major concern in areas where deicing agents are used. These properties have contributed to the increased use of high performance concrete in recent bridge design and construction.

Some disadvantages that result from the use of silica fume include a higher water demand in the concrete mix, a larger amount of air entraining admixture, and a decrease in workability.

Ground granulated blast-furnace slag is created when molten iron blast furnace slag is quickly cooled with water. This admixture can be substituted for cement on a 1:1 basis. However, it is usually limited to 25% in areas where the concrete will be exposed to deicing salts and to 50% in areas that do not need to use deicing salts.

Water reducing admixtures and plasticizers are used to aid workability at lower water/cement ratios, improve concrete quality and strength using less cement content, and help in placing concrete in adverse conditions. These admixtures can be salts and modifications of hydroxylized carboxylic acids, or modifications of lignosulfonic acids, and polymeric materials. Some of the potentially negative effects that are encountered when using water reducers and plasticizers include loss of slump and excess setting time.

Retarding admixtures are used to slow down the hydration process while not changing the long-term mechanical properties. This type of admixture is needed when heat is a problem. Retarders slow down the setting time to reduce unwanted temperature and shrinkage cracks which result from a fast curing mix.

2.2.2 Properties of Concrete

It is necessary for the bridge inspector to understand the different physical and mechanical properties of concrete and how they relate to concrete bridges in service today.

Physical Properties

The major physical properties of concrete are:

- Thermal expansion - concrete expands as temperature increases and contracts as temperature decreases
- Porosity - because of entrapped air, the cement paste never completely

fills the spaces between the aggregate particles, permitting absorption of water and the passage of water under pressure

- Volume changes due to moisture - concrete expands with an increase in moisture and contracts with a decrease in moisture
- Fire resistance - quality concrete is highly resistant to the effects of heat; however, temperatures over 370°C (700°F) may cause damage
- Formability - concrete can be cast to any shape prior to curing

Mechanical Properties

The major mechanical properties of concrete are:

- Strength - Plain, unreinforced concrete has a 28-day compressive strength ranging from about 17 MPa (2500 psi) to about 41 MPa (6000 psi). Higher strength concrete, with compressive strengths ranging from 41 MPa (6000 psi) to about 76 MPa (11,000 psi), is also available and becoming more commonly used. However, its tensile strength is only about 10% of its compressive strength, its shear strength is about 12% to 13% of its compressive strength, and its flexural strength is about 14% of its compressive strength (see Table 2.2.1).

  Six principal factors that increase concrete strength are:

  - Increased cement content
  - Increased aggregate strength
  - Decreased water-to-cement ratio
  - Decreased entrapped air
  - Increased curing time (extent of hydration)
  - Use of pozzolanic admixtures and slag

- Elasticity - Within the range of normal use, concrete is able to deform a limited amount under load and still return to its original orientation when the load is removed (elastic deformation). Elasticity varies as the square root of compressive strength. See Topic P.1 for modulus of elasticity and how it affects elastic deformation.

- Creep - In addition to elastic deformation, concrete exhibits long-term, irreversible, continuing deformation under application of a sustained load. Creep (plastic deformation) ranges from 100% to 200% of initial elastic deformation, depending on time.

- Isotropy - Plain, unreinforced concrete has the same mechanical properties regardless of which direction it is loaded.

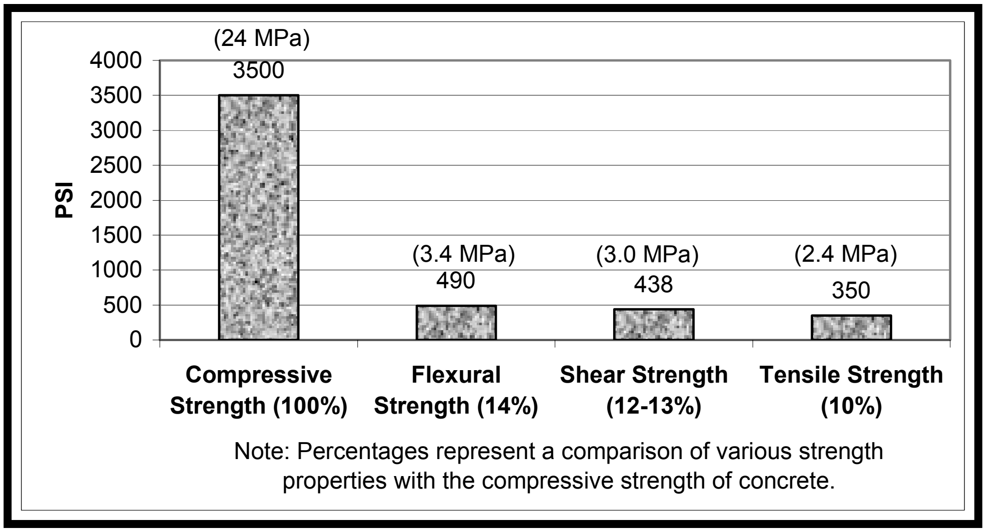


Table 2.2.1 Strength Properties of Concrete (24 Mpa) (3500 psi Concrete)

High Performance Concrete

High performance concrete (HPC) has been used for more than 20 years in the building industry. Under the FHWA's Strategic Highway Research Program (SHRP) Implementation Program, four types of high performance concrete mix designs were developed (see Table 2.2.2). High performance concrete is distinguished from regular concrete by its curing conditions and proportions of the ingredients in the mix design. The use of fly ash and high range water reducers play an important role in the design of HPC, as well as optimizing all components of the mix. Due to the increased strength and reduced permeability of HPC, bridge decks using HPC are expected to have double the life of conventional concrete bridge decks. The type and strength characteristics of concrete used to construct bridge components can be found in the bridge file under design specifications.

HPC Type	Minimum Strength Criteria	Water-Cementitious Ratio	Minimum Durability Factor
Very Early Strength (VES)	13.8 MPa (2,000 PSI)/ 6 hours	≤ 0.4	80%
High Early Strength (HES)	34.5 MPa (5,000 PSI)/ 24 hours	≤ 0.35	80%
Very High Strength (VHS)	69 MPa (10,000 PSI)/ 28 hours	≤ 0.35	80%
Fiber Reinforced	HES + (steel or poly)	≤ 0.35	80%
Additional information on the definition of HPC:			
- "HPC Defined for Highway Structures," Charles Goodspeed, Suneel Vanik			
Cook; *Concrete International*, February 1996, The American Concrete Ins			
- "Workshop Showcases High-Performance Concrete Bridges," *Focus* New			
May 1996.			

Table 2.2.2 FHWA's SHRP Implemented HPC Mix Designs

2.2.3 Reinforced Concrete

Concrete is commonly used in bridge applications due to its compressive strength properties. However, in order to supplement the limited tensile strength of concrete, tensile steel reinforcement is used (see Figure 2.2.1).

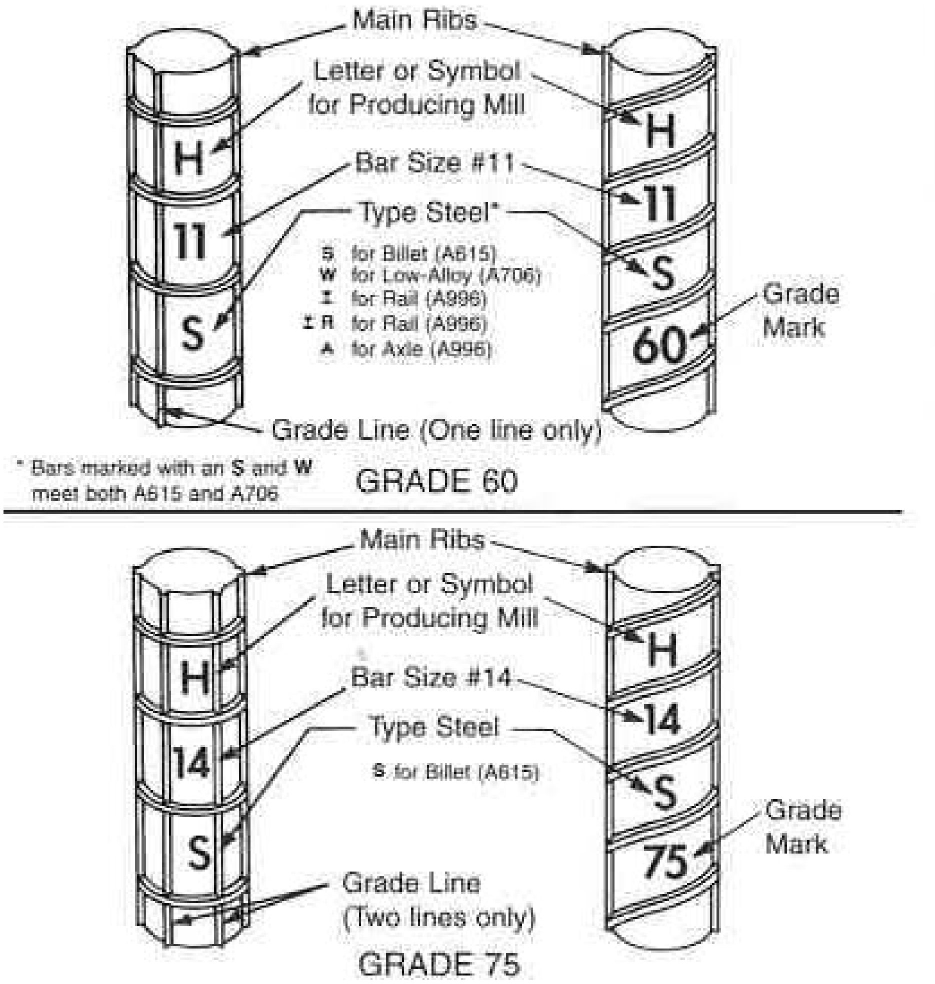
Figure 2.2.1 Concrete Member with Tensile Steel Reinforcement Showing

Steel reinforcement has a tensile yield strength of 276 MPa (40 ksi) or 414 MPa (60 ksi) and therefore has approximately 100 times the tensile strength of commonly used concrete. Therefore, in reinforced concrete members, the concrete resists the compressive forces and the steel reinforcement primarily resists the tensile forces. The type of steel reinforcement used in reinforced concrete is "mild steel", which is a term used for low carbon steels. The steel reinforcement is located close to the tension face of a structural member to maximize its efficiency.

Shear reinforcement is also needed to resist diagonal tension (refer to Topic P.1). Shear cracks start at the bottom of concrete members near the support and propagate upward and away from the support at approximately a 45° angle. Vertical or diagonal shear reinforcement is provided in this area to intercept the cracks and to stop the cracks from opening wider.

Reinforcing bars are also placed uniformly around the perimeter of a member to resist stresses resulting from temperature changes and volumetric changes of concrete. This steel is referred to as temperature and shrinkage steel.

Steel reinforcing bars can be "plain" or smooth surfaced, or they can be "deformed" with a raised gripping pattern protruding from the surface of the bar (see Figure 2.2.2). The gripping pattern improves bond with the surrounding concrete. Modern reinforced concrete bridges are generally constructed with "deformed" reinforcing steel.



INCH-POUND BAR SIZE	DIAMETER (in.)	METRIC BAR SIZE	DIAMETER (mm)
#3	0.375	#10	9.5
#4	0.500	#13	12.7
#5	0.625	#16	15.9
#6	0.750	#19	19.1
#7	0.875	#22	22.2
#8	1.000	#25	25.4
#9	1.128	#29	28.7
#10	1.270	#32	32.3
#11	1.410	#36	35.8
#14	1.693	#43	43.0
#18	2.257	#57	57.3

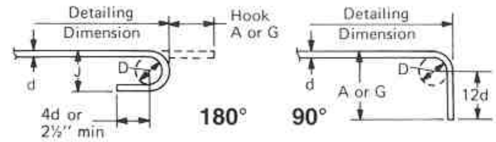
Figure 2.2.2 Standard Deformed Reinforcing Bars (Source: Concrete Reinforcing Steel Institute)

STANDARD HOOK DETAILS

in accordance with ACI 318-99

All grades of steel (min yield strengths)
D = Finished inside bend diameter
d = Bar diameter

D = 6d for #3 through #8
D = 8d for #9, #10 and #11
D = 10d for #14 and #18



RECOMMENDED END HOOKS

BAR SIZE	D	180° HOOKS		90° HOOKS
		A or G	J	A or G
#3	2¼"	5"	3"	6"
#4	3"	6"	4"	8"
#5	3¾"	7"	5"	10"
#6	4½"	8"	6"	1'-0"
#7	5¼"	10"	7"	1'-2"
#8	6"	11"	8"	1'-4"
#9	9½"	1'-3"	11¾"	1'-7"
#10	10¾"	1'-5"	1'-1¼"	1'-10"
#11	12"	1'-7"	1'-2¾"	2'-0"
#14	18¼"	2'-3"	1'-9¾"	2'-7"
#18	24"	3'-0"	2'-4½"	3'-5"

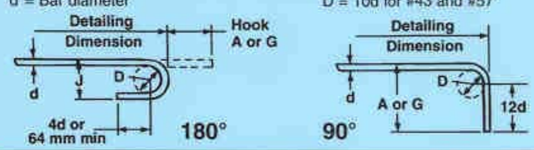
STEEL TYPE	BAR SIZE RANGE	GRADE	MINIMUM YIELD, KSI	MINIMUM TENSILE, KSI
Billet A615	#3 - #6	40	40	70
	#3 - #18	60	60	90
	#6 - #18	75	75	100
Low-Alloy A706	#3 - #18	60	60	80
Rail & Axle A996	#3 - #8	40	40	70
	#3 - #8	50	50	80
	#3 - #8	60	60	90

STANDARD METRIC HOOK DETAILS

in accordance with ACI 318M-99

All grades of steel (min yield strengths)
D = Finished inside bend diameter
d = Bar diameter

D = 6d for #10 through #25
D = 8d for #29, #32 and #36
D = 10d for #43 and #57



RECOMMENDED END HOOKS

BAR SIZE	D	180° HOOKS		90° HOOKS
		A or G	J	A or G
#10	60	125	80	150
#13	80	150	105	200
#16	95	175	130	250
#19	115	200	155	300
#22	135	250	180	375
#25	155	275	205	425
#29	240	375	300	475
#32	275	425	335	550
#36	305	475	375	600
#43	465	675	550	775
#57	610	925	725	1050

NOTE: All dimensions are in millimeters (mm).

STEEL TYPE	BAR SIZE RANGE	GRADE	MINIMUM YIELD, MPa	MINIMUM TENSILE, MPa
Billet A615M	#10 - #19	300	300	500
	#10 - #57	420	420	620
	#19 - #57	520	520	690
Low-Alloy A706M	#10 - #57	420	420	550
Rail & Axle A996	#10 - #25	300	300	500
	#10 - #25	350	350	550
	#10 - #25	420	420	620

Figure 2.2.2 Standard Deformed Reinforcing Bars (Source: Concrete Reinforcing Steel Institute) (Continued)

In US units, reinforcing bars up to 1” nominal diameter are identified by numbers that correspond to their nominal diameter in eighths of an inch. For example, a #4 bar has a 1/2 inch nominal diameter (or 4 times 1/8 inch). For the remaining bar sizes (#9, #10, #11, #14, and #18), the area is equivalent to the old 1”, 1 1/8”, 1 1/4”, 1 1/2”, and 2” square bars, respectively.

Reinforcing bars can also be used to increase the compressive strength of a concrete member. When reinforcing bars are properly cast into a concrete member, the steel and concrete acting together provide a strong, durable construction material.

Reinforcing bars can be protected or unprotected from corrosion. Unprotected reinforcement is referred to as “black” steel because only mill scale is present on the surface. Unprotected reinforcement is primarily used in concrete footings or other concrete members that are underground or not exposed to moisture.

The deformed epoxy coated bar is the most common type of protected reinforcing bar used. It is commonly specified when a concrete member may be exposed to an adverse environment. The epoxy provides a protective coating against corrosion agents such as de-icing chemicals and brackish water, and is inexpensive compared to other protective coatings. Another type of protected reinforcing bar is the galvanized bar. Unprotected bars are given a zinc coating, which slows down or stops the corrosion process. See Topic 2.2.9 for a detailed description of reinforcement protective coatings.

In the near future, tensile reinforcement made of Fiber Reinforced Polymer (FRP) composites may become common. Currently the FHWA and other government agencies and universities are performing research to better understand the properties and appropriate methods for use of this material in new construction. FRP is lighter weight than traditional steel reinforcement, can be designed with a wide range of mechanical properties including tensile, flexural, impact and compressive strengths and provides a viable alternative in areas where deicing salts are used due to the fact that the deck does not deteriorate due to steel reinforcement corrosion. Current holdbacks to widespread use of FRP reinforcement are the relative cost when compared to steel, the limited amount of experience contractors have building with it, and not having much performance data.

2.2.4 Prestressed Concrete

Another type of concrete used in bridge applications is prestressed concrete, which uses high tensile strength steel strands as reinforcement. To reduce the tensile forces in a concrete member, internal compressive forces are induced through prestressing steel tendons or strands. When loads are applied to the member, any tensile forces developed are counterbalanced by the internal compressive forces induced by the prestressing steel. By prestressing the concrete in this manner, the final tensile forces are typically within the tensile strength limits of plain concrete. Therefore, properly designed prestressed concrete members do not develop flexure cracks under service loads (see Figure 2.2.3).

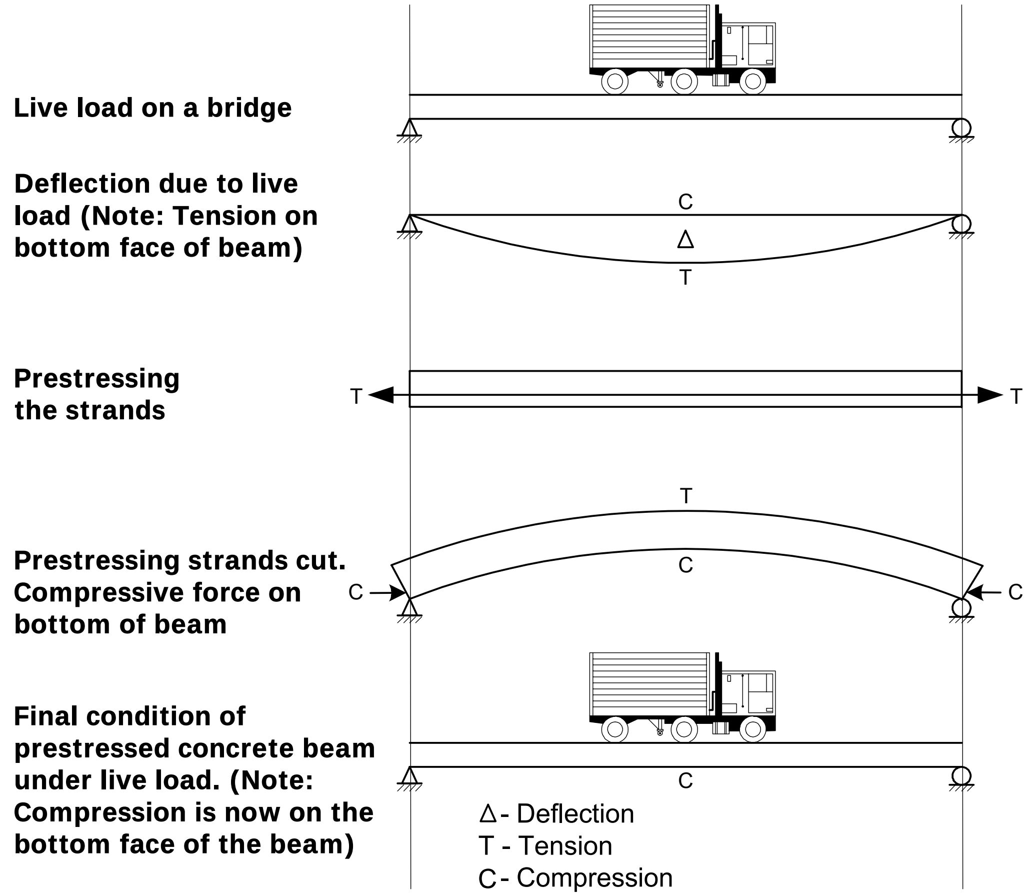


Figure 2.2.3 Prestressed Concrete Beam

There are three methods of prestressing concrete:

- Pretensioning - during fabrication of the member, prestressing steel is placed and tensioned prior to casting and curing of the concrete (see Figure 2.2.4)
- Post-tensioning - during fabrication of the member, ducts are cast-in-place so that after curing, the prestressing steel can be passed through the ducts and tensioned (see Figure 2.2.5)
- Combination method - this is used for long members for which the required prestressing force cannot safely be applied using pretensioning only

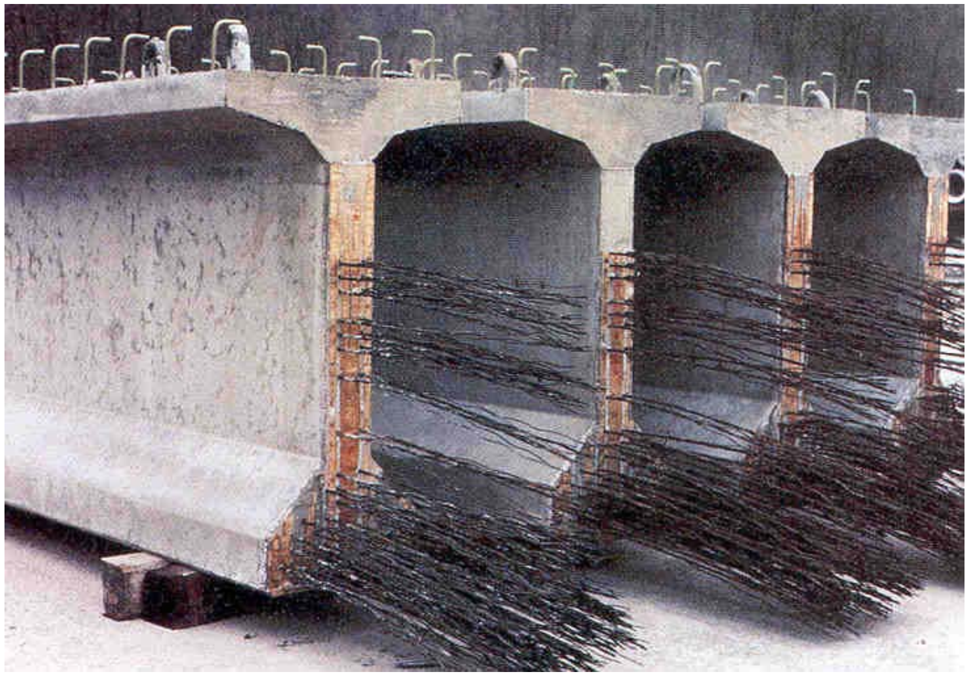


Figure 2.2.4 Pretensioned Concrete I-beams

Figure 2.2.5 Post-tensioned Concrete Box Girder

Steel for prestressing, which is named high tensile strength steel, comes in three basic forms:

- Wires (ASTM A421) - single wires or parallel wire cables; the parallel wire cables are commonly used in post-tensioning operations; the most popular wire size is 6 mm (1/4 inch) diameter and the most common grade of steel is the 1860 Mpa (270 ksi) grade.

- Strands (ASTM A416) - fabricated by twisting wires together; the seven wire strand is the most common type of prestressing steel used in the United States, and the 1860 Mpa (270 ksi) grade is most commonly used today

- Bars (ASTM A322 and A29) - high tensile strength bars typically have a minimum ultimate stress of 1000 Mpa (145 ksi); the bars have full length deformations that also serve as threads to receive couplers and anchorage hardware

Epoxy coated prestressing strand is a newer alternative to help minimize the amount of corrosion that occurs to otherwise unprotected strands. The epoxy is applied to the ordinary seven wire low relaxation prestressing strand through a process called "fusion bonding". Once the epoxy is applied, the strand has very little bond capacity and an aluminum oxide grit has to be applied to aid in the bonding. From recent testing by the FHWA, the epoxy coated strands have a tendency to slip when advanced curing temperatures are 63°C (145°F) and above. This slip occurs because the epoxy material begins to melt at these temperatures. Because the epoxy coating has a tendency to melt, this type of alternative is not used unless protection of the prestressing strand is critical.

In pretensioned members, transfer of tendon tensile stress occurs through bonding, which is the secure interaction of the prestressing steel with the surrounding concrete. This is accomplished by casting the concrete in direct contact with the prestressed steel.

In post-tensioned members, transfer of tendon tensile stress is accomplished by mechanical end anchorages and locking devices. If bonding is also desired, special ducts are used which are pressure injected with grout after the tendons are tensioned and locked off.

For purposes of crack control in end sections of pretensioned members, the prestressing steel is sometimes debonded. This is accomplished by providing a protective cover on the steel, preventing it from contacting the concrete.

For post-tensioned members, when bonding is not desirable, grouting of tendon ducts is not performed and corrosion protection in the form of galvanizing, greasing, or some other means must be provided.

In prestressed concrete beams, shear strength is enhanced by the local compressive stress present. However, mild shear reinforcement is still required. Similar to reinforced concrete, prestressed concrete also requires mild steel temperature and shrinkage reinforcement.

2.2.5 Types of Concrete Deterioration

In order to properly inspect a concrete bridge, the inspector must be able to recognize the various types of defects or deterioration associated with concrete. The inspector must also understand the causes of the defects or deterioration and how to examine them. There are many common defects or deterioration that occur on reinforced concrete bridges:

- Cracking (flexure, shear, freeze-thaw cycles)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Efflorescence
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage

Structural Cracks

A crack is a linear fracture in concrete. It may extend partially or completely through the member. There are two basic types of cracks: structural and non-structural cracks. Structural cracks are caused by dead load and live load stresses. Cracking is considered normal for mildly reinforced concrete (e.g., in cast-in-place tee-beams) as long as the cracks are small and there are no rust stains or other signs of deterioration present. Larger structural cracks indicate potentially serious problems, because they are directly related to the structural capacity of the member. When cracks can be observed opening and closing under load, they are referred to as "working" cracks. See Table 2.2.3 for crack width guidelines. There are two types of structural cracks: flexure and shear (see Figure 2.2.6).

Flexure Cracks

Flexure cracks are caused by tensile forces and therefore develop in the tension zones. Tension zones occur either on the bottom or the top of a member, depending on the span configuration. Tension zones can also occur in substructure components. Tension cracks terminate when they approach the neutral axis of the member. If a beam is a simple span structure (refer to Topic P.1.9), flexure cracks can often be found at the mid-span at the bottom of the member where bending or flexure stress is greatest (see Figure 2.2.7). If the beams are continuous span structures (refer to Topic P.1.9), flexure cracks occur at the top of members at or near their supports.

Shear Cracks

Shear cracks are caused by diagonal tensile forces that typically occur in the web of a member near the supports where shear stress is the greatest. Normally, these cracks initiate near the bearing area, beginning at the bottom of the member, and extending diagonally upward toward the center of the member (see Figure 2.2.8). Shear cracks also occur in abutment backwalls, stems and footings, pier caps, columns, and footings.

Although structural cracks are typically caused by dead load and live load forces, they can also be caused by overstresses in members resulting from unexpected secondary forces. Restricted thermal expansion or contraction such as caused by frozen bearings, or forces due to the expansion of an approach slab or failure of a backwall can induce significant forces which result in cracks (see Figure 2.2.9).

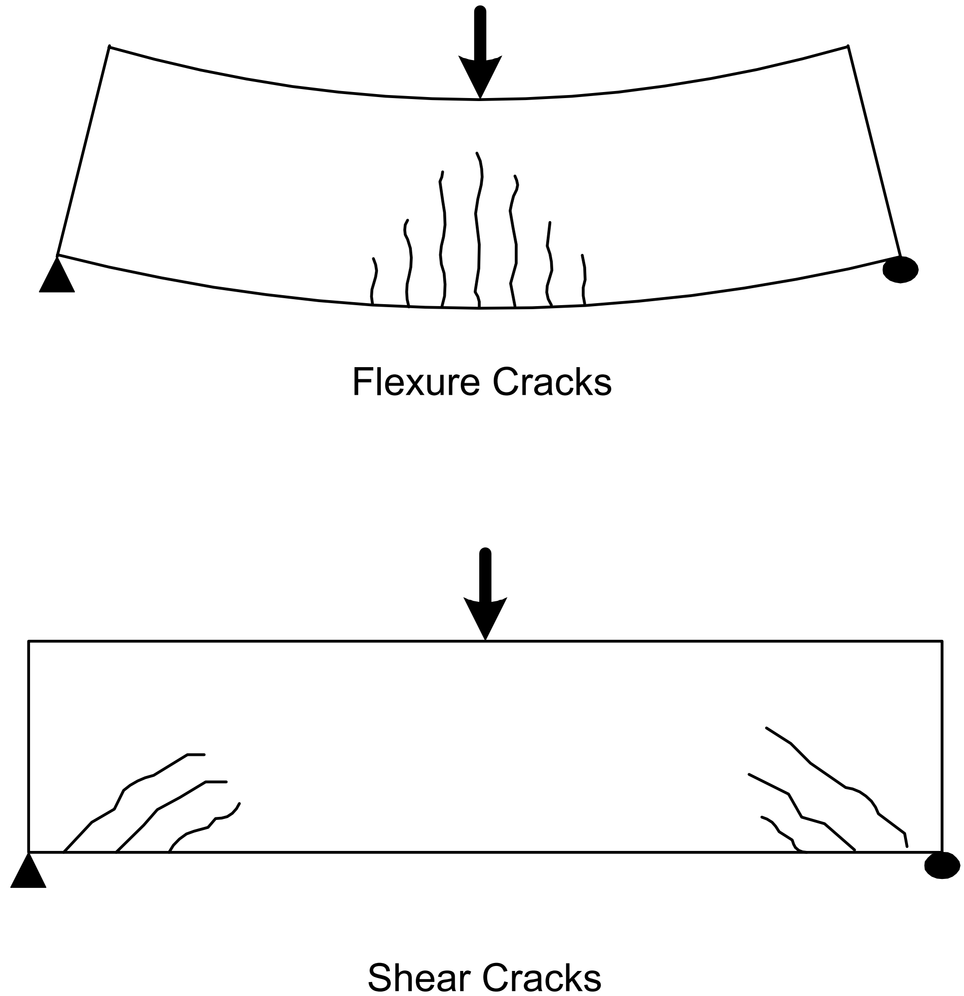


Figure 2.2.6 Structural Cracks

Figure 2.2.7 Flexural Crack on a Tee Beam

Figure 2.2.8 Shear Crack on a Slab Beam

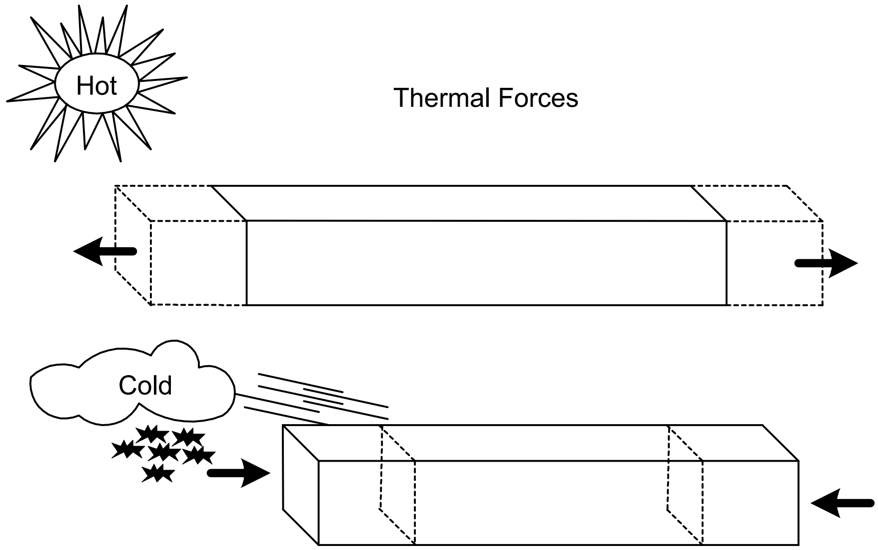


Figure 2.2.9 Thermal Forces

Crack Size

Crack size is very important in assessing the condition of an in-service bridge. Cracks may extend partially or completely through the concrete member. On reinforced concrete, cracking will usually be large enough to be seen with the naked eye. A crack comparitor card can be used to measure and differentiate cracks (see Figure 2.2.10 and Table 2.2.3).

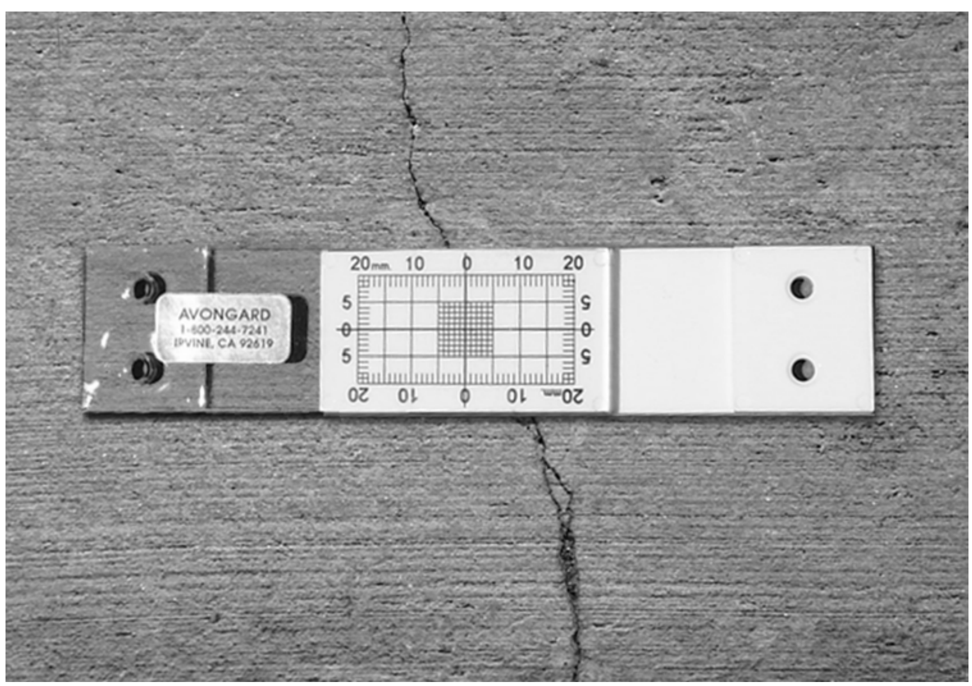


Figure 2.2.10 Crack Comparitor Card and Crack Gauge

CRACK WIDTH GUIDELINES				
	REINFORCED CONCRETE		PRESTRESSED CONCRETE	
	English	Metric	English	Metric
HAIRLINE (HL)	< 1/16"	< 1.6mm	< 0.004"	< 0.1mm
	(0.0625)			
NARROW (N)	1/16" to 1/8"	1.6 to 3.2mm	0.004 to 0.009"	0.1 to 0.23mm
	0.0625" – 0.125"			
MEDIUM (M)	1/8" to 3/16"	3.2 to 4.8mm	0.010 to 0.030"	0.25 to 0.76mm
	0.125" – 0.1875"			
WIDE (W)	>3/16"	> 4.8mm	> 0.030"	> 0.76mm
	>0.1875"			

Table 2.2.3 Crack Width Guidelines

Cracks can be classified as hairline, narrow, medium, or wide. Hairline cracks are small and cannot be measured with normal equipment, such as a six-foot rule. Medium and wide cracks are cracks that can be measured by simple means or a crack comparitor card. On conventionally reinforced structures, hairline cracks are usually insignificant. All other crack widths may be significant and should be monitored and recorded in the inspection notes.

On prestressed structures, all cracks are significant and an optical crack gauge is the proper instrument needed to measure and differentiate cracks.

When reporting cracks, the length, width, location, and orientation (horizontal, vertical, or diagonal) should be noted. Both large and small cracks in main members, especially in prestressed members, should be carefully recorded. The presence of rust stains or efflorescence or evidence of differential movement on either side of the crack should be indicated.

Nonstructural Cracks

Nonstructural cracks result from internal stresses due to dimensional changes. Nonstructural cracks are divided into three categories:

- Temperature cracks (see Figure 2.2.11)
- Shrinkage cracks (see Figure 2.2.12)
- Mass concrete cracks

Though these cracks are nonstructural and relatively small in size, they provide openings for water and contaminants, which can lead to serious problems. Temperature cracks are caused by the thermal expansion and contraction of the concrete. Concrete expands or contracts as its temperature rises or falls. If the concrete is prevented from contracting, due to friction or because it is being held in place, it will crack under tension. Inoperative bearing devices and clogged expansion joints can also cause this to occur. Shrinkage cracks are due to the shrinkage of concrete caused by the curing process. Volume reduction due to curing is also referred to as plastic shrinkage. Plastic shrinkage cracks occur while the concrete is still plastic and are usually short, irregular shapes and do not extend the full depth into the member. Mass concrete cracks occur due to thermal gradients (differences between interior and exterior) in massive sections immediately after placement and for a period of time thereafter. Temperature, shrinkage, and mass concrete cracks typically do not significantly affect the structural strength of a concrete member.

Figure 2.2.11 Temperature Cracks

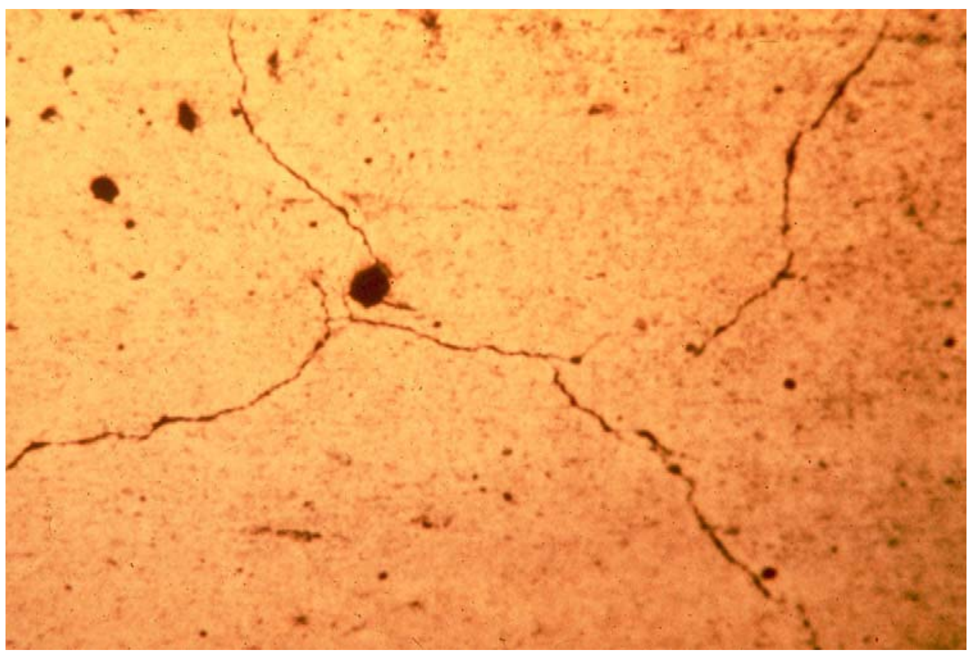


Figure 2.2.12 Shrinkage Cracks

In concrete bridge decks, temperature and shrinkage cracks can occur in both the transverse and longitudinal directions. In retaining walls and abutments, these cracks are usually vertical, and in concrete beams, these cracks occur vertically or transversely on the member. However, since temperature and shrinkage stresses exist in all directions, the cracks could have other orientations.

Inspectors must exercise care in distinguishing between nonstructural cracks and structural cracks. However, regardless of the crack type, water seeps in and causes the reinforcement to corrode. The corroded reinforcement expands and exerts pressure on the concrete. This pressure can cause delaminations and spalls.

Crack Orientation

In addition to classifying cracks as either structural or nonstructural and recording their lengths and widths, inspectors must also describe the orientation of the cracks. The orientation of the crack with respect to the loads and supporting members is an important feature that must be recorded accurately to ensure the proper evaluation of the crack. The orientation of cracks may generally be described by one of the following five categories:

- Transverse cracks – These are fairly straight cracks that are roughly perpendicular to the centerline of the bridge or a bridge member (see Figure 2.2.13).

- Longitudinal cracks - These are fairly straight cracks that run parallel to the centerline of the bridge or a bridge member (see Figure 2.2.14).

- Diagonal cracks - These cracks are skewed (at an angle) to the centerline of the bridge or a bridge member, either vertically or horizontally.

- Pattern or map cracking - These are inter-connected cracks that form networks of varying size. They vary in width from barely visible, fine cracks to cracks with a well defined opening. Map cracking resembles the lines on a road map (see Figure 2.2.15).

- Random cracks - These are meandering, irregular cracks. They have no particular form and do not logically fall into any of the types described above.

Figure 2.2.13 Transverse Cracks

Figure 2.2.14 Longitudinal Cracks

Figure 2.2.15 Pattern or Map Cracks

Scaling

Scaling is the gradual and continuing loss of surface mortar and aggregate over an area due to the chemical breakdown of the cement bond. Scaling is accelerated when the member is exposed to a harsh environment. Scaling is classified in the following four categories:

- Light or minor scale - loss of surface mortar up to 6 mm (¼ inch) deep, with surface exposure of coarse aggregates (see Figure 2.2.16)
- Medium or moderate scale - loss of surface mortar from 6 to 13 mm (¼ inch to ½ inch) deep, with mortar loss between the coarse aggregates (see Figure 2.2.17)
- Heavy scale - loss of surface mortar from 13 to 25 mm (½ inch to 1 inch) deep; coarse aggregates are clearly exposed (see Figure 2.2.18)
- Severe scale - loss of coarse aggregate particles, as well as surface mortar and the mortar surrounding the aggregates; depth of the loss exceeds 25 mm (1 inch); reinforcing steel is usually exposed (see Figure 2.2.19)

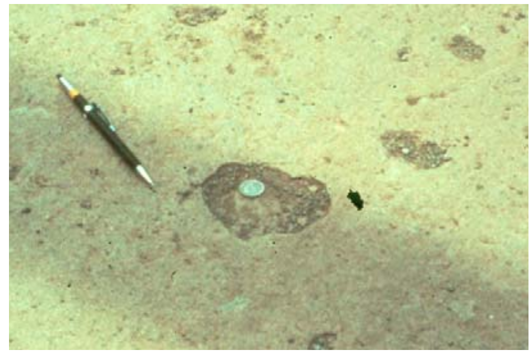


Figure 2.2.16 Light or Minor Scale

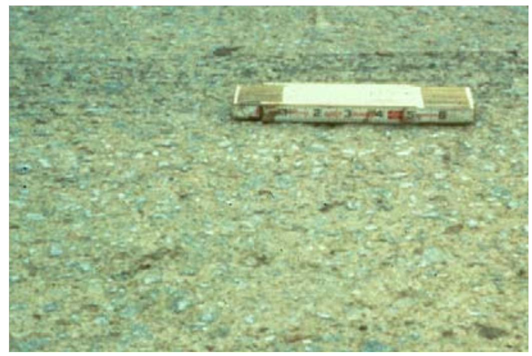


Figure 2.2.17 Medium or Moderate Scale

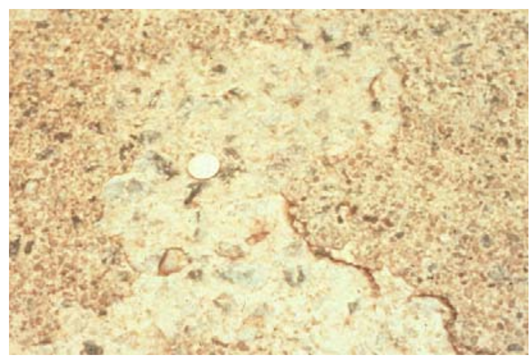


Figure 2.2.18 Heavy Scale

Figure 2.2.19 Severe Scale

When reporting scaling, the inspector should note the location of the defect, the size of the affected area, and the scaling classification. For severe scale, the depth of penetration of the defect should also be recorded.

Delamination

Delamination occurs when layers of concrete separate at or near the level of the top or outermost layer of reinforcing steel. The major cause of delamination is expansion of corroding reinforcing steel. This is commonly caused by intrusion of chlorides or salt. Another cause of delamination is severe overstress in a member. Delaminated areas give off a hollow "clacking" sound when tapped with a hammer. When a delaminated area completely separates from the member, the resulting depression is called a spall.

When reporting delamination, the inspector should note the location and the size of the defect.

Spalling

A spall is a depression in the concrete (see Figure 2.2.20). Spalls result from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface. Spalls can be caused by corroding reinforcement, friction from thermal movement, and overstress. Reinforcing steel is often exposed in a spall, and the common shallow pothole in a concrete deck is considered a spall. Spalls are classified as follows:

- Small spalls - not more than 25 mm (1 inch) deep or approximately 150 mm (6 inches) in diameter
- Large spalls - more than 25 mm (1 inch) deep or greater than 150 mm (6 inches) in diameter

Figure 2.2.20 Spalling on a Concrete Deck

When concrete is overstressed, it gives or fractures. Over time, the fracture opens wider from debris, freeze/thaw cycles, or more overstress. This cycle continues until a spall is formed. Spalls caused from overstress are very serious and should be brought to the attention of the Chief Bridge Engineer. Most spalls are caused from corroding reinforcement, but if the spall is located at or near a high moment region, overstress may be the cause. Examples that might indicate a spall was caused by overstress include:

- A spall that is at or near flexure cracks in the lower portion of a beam at mid-span
- A spall that is at or near flexure cracks in the top of a continuous member over a support

Similarly, when concrete is overstressed in compression, it is common for the surface to spall.

When reporting spalls, the inspector should note the location of the defect, the size of the area, and the depth of the defect.

Chloride Contamination

Chloride contamination in concrete is the presence of recrystalized soluble salts. Concrete is exposed to chlorides in the form of deicing salts, acid rain, and in some cases, contaminated water used in the concrete mix. It causes accelerated reinforcement corrosion that leads to cracking of the concrete.

Efflorescence

The presence of cracks permits moisture absorption and increased flow within the concrete that is evidenced by dirty-white surface deposits called efflorescence. Efflorescence is a combination of calcium carbonate leached out of the cement paste and other recrystalized carbonate and chloride compounds (see Figure 2.2.21). In order to estimate the percent of concrete contaminated by chloride, nondestructive testing is required (refer to Topic 13.2.2).

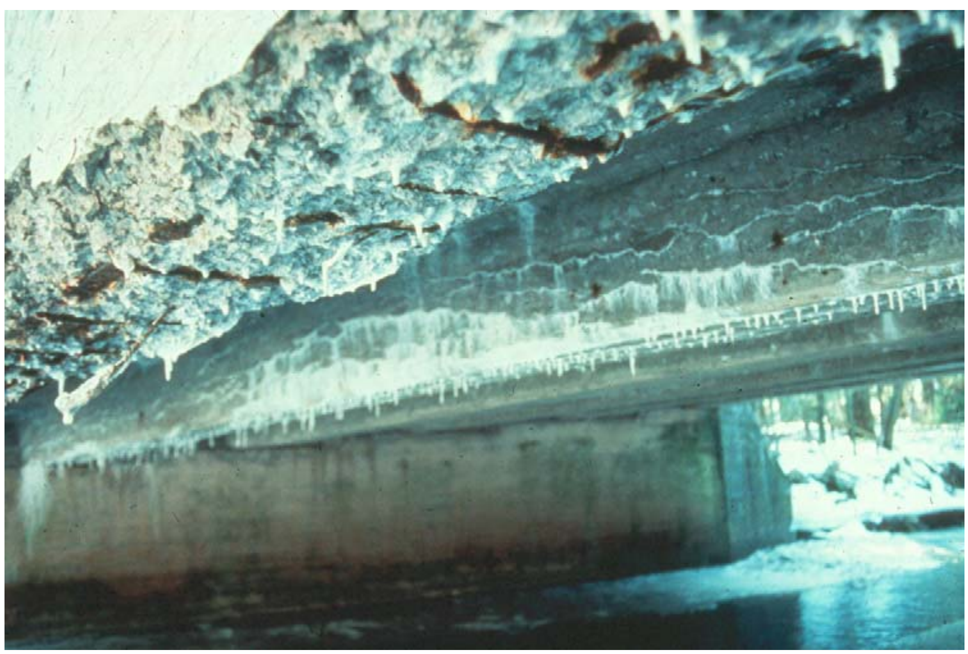


Figure 2.2.21 Efflorescence

Ettringite Formation

Ettringite formation is an internal defect that occurs in concrete from the reaction of sulfates, calcium aluminates, and water. From this reaction, ettringite, which is a crystalline mineral, expands up to eight times in volume compared to the volume of the tricalcium nitrates (C_3A). Ettringite formation is initially formed when water is added to the cement but prior to the concrete's initial set. The initial formation does not harm the concrete. A secondary or delayed ettringite formation occurs after the concrete has hardened. This formation creates very high forces in hardened concrete and is the cause of the deterioration. The only way to identify ettringite formation as a cause of premature concrete deterioration is through advanced inspection techniques such as petrographic analysis. Recent studies have shown that ettringite formation is linked to alkali-silica reaction (ASR), but further research is still needed.

Honeycombs

Honeycombs or construction voids are hollow spaces or voids that may be present within the concrete. Honeycombs are construction defects caused by improper vibration during concrete placement, resulting in the segregation of the coarse aggregates from the fine aggregates and cement paste. In some cases, honeycombs are the result of insufficient vibration, where the entire concrete mix does not physically reach the form surface (see Figure 2.2.22).

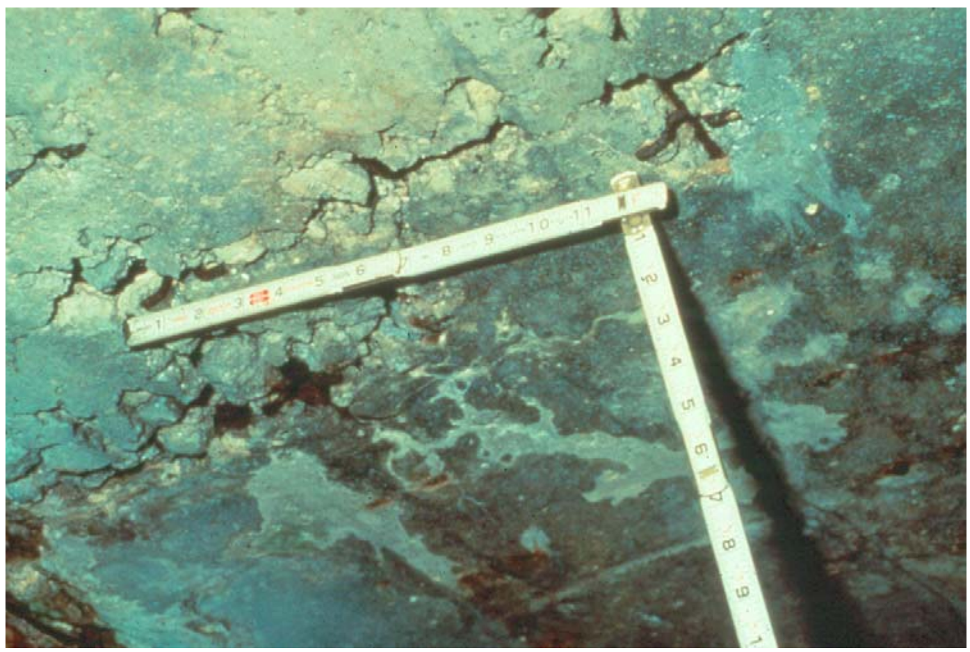


Figure 2.2.22 Honeycomb

Pop-outs

Pop-outs are conical fragments that break out of the surface of the concrete, leaving small holes. Generally, a shattered aggregate particle will be found at the bottom of the hole, with a part of the fragment still adhering to the small end of the pop-out cone. Pop-outs are caused by aggregates which expand with absorption of moisture. Other causes of pop-outs include use of reactive aggregates and high alkali cement.

Wear

Wear is the gradual removal of surface mortar due to friction and occurs to concrete surfaces, like a bridge deck, when exposed to traffic. Advanced wear exhibits polished aggregate, which is potentially a safety hazard when the deck is wet. The scraping action of snowplows and street sweepers also wears the deck surface and damages curbs, parapets, and pier faces.

Collision Damage

Trucks, derailed railroad cars, or marine traffic may strike and damage concrete bridge components (see Figures 2.2.23 and 2.2.24). The damage is generally in the form of cracking or spalling, with exposed reinforcement. Prestressed beams are particularly sensitive to collision damage, as exposed tendons undergo stress corrosion and fail prematurely.

Figure 2.2.23 Concrete Column Collision Damage

Figure 2.2.24 Collision Damage to Prestressed Concrete I-Beam

Abrasion

Abrasion damage is the result of external forces acting on the surface of the concrete member and is similar to wear. Erosive action of silt-laden water running over a concrete surface and ice flow in rivers and streams can cause considerable abrasion damage to concrete piers and pilings. In addition, concrete surfaces in surf zones may be damaged by the abrasive action of sand and silt in the water. Abrasion damage can be accelerated by freeze-thaw cycles. This will usually occur near the water line on concrete piers. The use of the term "scour" to indicate "abrasion" is incorrect. The term scour is used to describe the loss of streambed material from around the base of a pier or abutment due to stream flow or tidal action.

Overload Damage

Overload damage or serious structural cracking occurs when concrete members are sufficiently overstressed. Concrete decks, beams, and girders are all subject to damage from such overload conditions. Note any excessive vibration or deflection that may occur under traffic, which can indicate overstress. Other visual signs that can indicate overstress include excessive sagging, spalling, and/or cracking at the mid-span of simple span structures and at the supports of continuous span structures. Permanent deformation is another visual sign of overstress damage in a member. If overload damage is detected or suspected, the Chief Bridge Engineer should be notified immediately.

2.2.6 Reinforcing Steel Corrosion

Due to the chemistry of the concrete mix, reinforcing steel embedded in concrete is normally protected from corrosion. In the high alkaline environment of the concrete, a tightly adhering film forms on the steel that protects it from corrosion. However, this protection is eliminated by the intrusion of chlorides, which enables water and oxygen to attack the reinforcing steel, forming iron oxide (i.e., rust). Chloride ions are introduced into the concrete by marine spray, industrial brine, or deicing agents. These chloride ions can reach the reinforcing steel by diffusing through the concrete or by penetrating cracks in the concrete. An inspector may see rebar rust stains on the outer concrete surfaces before a spall occurs. The corrosion product (rust) can occupy up to 10 times the volume of the corroded steel that it replaces. This expansive action creates internal pressures up to 20.7 MPa (3000 psi) that will cause the concrete to yield, resulting in wider cracks, delaminations, and spalls (see Figure 2.2.25).

Figure 2.2.25 Corroded Reinforcing Bar

2.2.7 Prestressed Concrete Deterioration

Prestressed concrete members deteriorate in a similar fashion to ordinary concrete members. However, the effects on Prestressed concrete member performance are usually more detrimental. Significant defects include:

- Structural cracks
- Exposed prestressing tendons
- Corrosion of tendons in the bond zone
- Loss of camber due to concrete creep
- Loss of camber due to lost prestress forces

Structural cracks indicate an overload condition has occurred. These cracks expose the tendons to the environment, which can lead to corrosion.

Exposed steel tendons via cracks or collision damage corrode at an accelerated rate due to the high tensile stresses carried and can fail prior to any measurable section loss due to environmentally induced cracking (EIC).

Environmentally induced cracking in steel prestressing strands can occur when the steel prestressing strands are subject to high tensile stresses in a corrosive environment. Rust stains may be present. The strands, which are normally ductile, undergo a brittle failure due to the combination of the corrosive environment along with the tensile stresses.

There are two types of environmentally induced cracking. The first is called stress corrosion cracking (SCC). This type of cracking grows at a slow rate and has a branched cracking pattern. The corrosion of prestressing steel along with the tensile stress in the steel causes a cracking pattern perpendicular to the stress direction.

The second type is called hydrogen-induced cracking (HIC) and occurs due to hydrogen diffusing into the prestressing steel. Once in the steel, hydrogen gas is formed. The hydrogen gas applies an internal pressure to the prestressing steel. This internal pressure, in conjunction with the tensile stress due to prestressing, has the ability to create very brittle, non-branching, fast growing cracks in the prestressing steel strands. The specific type of environmentally induced cracking can only be positively identified after failure through the use of advanced inspection techniques.

When deteriorated concrete cover allows corrosion of the tendons in the bond zone (the end thirds of the beam), loss of development occurs which reduces prestress force. This can sometimes be evidenced by reduced positive camber and ultimately structural cracking. Prestress force can also be reduced through a beam-shortening phenomenon called creep, which relaxes the steel tendons. Loss of prestress force is followed by structural cracking at normal loads due to reduced live load capacity.

2.2.8 Other Causes of Concrete Deterioration

Temperature Changes

Freezing and thawing are common causes of concrete deterioration (see Figure 2.2.26). Porous concrete absorbs water, and when this water freezes, high expansive pressures are created due to the larger volume created by ice formation. These pressures often produce cracking and light spalling. Freeze/thaw damage should not be confused with scaling.

Figure 2.2.26 Freeze-Thaw Damage on a River Pier

Chemical Attack

Aside from accelerated rebar corrosion, the use of salt or chemical deicing agents contributes to weathering through recrystallization. This is quite similar to the effects of freezing and thawing.

Sulfate compounds in soil and water are also a problem. Sodium, magnesium, and calcium sulfates react with compounds in cement paste and cause rapid deterioration of the concrete.

Alkali-silica reaction (ASR) is a reaction between the alkalis in cement with the silica molecules of various aggregates. When the reaction takes place, a gel-like substance is formed. Once exposed to moisture, the gel expands and causes cracking in the concrete.

Moisture Absorption

All concrete is porous and will absorb water to some degree. As water is absorbed, the concrete will swell. If restrained, the material will burst or the concrete will crack. This type of deterioration is limited to concrete members that are continuously submerged in water.

Differential Foundation Movement

Foundation movement can also cause serious cracking in concrete structures. Differential settlement induces stresses in the supported superstructure and can lead to concrete deterioration.

Design and Construction Deficiencies

Some conditions or improper construction methods that can cause concrete to deteriorate are:

- Insufficient reinforcement bar cover - Insufficient concrete cover over rebars may lead to early corrosion of the steel reinforcement.

- Weep holes and scuppers - Improper placement or inadequate sizing of scuppers and weep holes can cause an accumulation of water with its damaging effects.

- Leaking deck joints

- Improper curing - A primary cause of concrete deterioration (loss of strength).

- Soft spots - Soft spots in the subgrade of an approach slab will cause the slab to settle and crack.

- Premature form removal - If the formwork is removed between the time the concrete begins to harden and the specified time for formwork removal, cracks will probably occur.

- Improper vibration - If the concrete is not properly vibrated, internal settling of the concrete mix can cause surface cracking above the reinforcing bars as the mix settles around the bars. Excessive vibration may cause segregation (separation of water, aggregate, and cement) of the concrete mix.

- Impurities - The inclusion of clay or soft shale particles in the concrete mix will cause small holes to appear in the surface of the concrete as these particles dissolve. These holes are known as mudballs.

- Internal voids - If reinforcing bars are too closely spaced, voids, which collect water, can occur under the reinforcing mat if the mix is not properly vibrated.

Fire Damage

- Extreme heat will damage concrete. High temperatures (above 370°C (700°F)) will cause a weakening in the cement paste and lead to cracking.

2.2.9 Protective Systems

Types and Characteristics of Concrete Coatings

Coatings form a protective barrier film on the surface of concrete to preclude entry of water and chlorides into the porous concrete. The practice of coating the concrete surface varies with each agency. Two primary concrete coatings are paint and water repellent membranes.

Paint

Paint is applied in one or two layers. The first layer fills the voids in a rough concrete surface. The second layer forms a protective film over the first. On smooth concrete surfaces, only one layer may be necessary.

Several classes of paint are coated on concrete:

- Oil-based paint
- Latex paint
- Epoxy paint

- Urethanes

Oil-based Paint

Oil-based paint is declining in use but is still found on some older concrete structures. Oil paint is subject to saponification failure in wet areas. Saponification is a chemical attack on the coating caused by the inherent alkalinity of the concrete. The moisture may be from humidity in the atmosphere, rain runoff, or ground water entering the porous concrete from below. Saponification does not occur over dry concrete (or occurs at a greatly reduced rate).

Latex Paint

Latex paint consists of a resin emulsion. Latexes can contain a variety of synthetic polymer binding agents. Latex paint resists attack by the alkaline concrete. Acrylic or vinyl latexes provide better overall performance, in that they are more resistant to alkaline attack than oil-based paint. Latex paints, however, are susceptible to efflorescence. Efflorescing is a process in which water-soluble salts pass outward through concrete and are deposited at the concrete/paint interface. This can cause loss of coating adhesion. If the paint is also permeable to water, the salts are deposited on the paint surface as the water evaporates.

Acrylics do not chalk as rapidly as other latexes and have good resistance to ultraviolet rays in sunlight. Polyvinyl acetate latexes are the most sensitive to attack by alkalis.

Epoxy Paint

Epoxy paint uses a cross-linking polymer binder, in which the epoxy resin in the paint undergoes a chemical reaction as the paint cures, forming a tough, cross-linked paint layer. Epoxies have excellent resistance to chemicals, water, and atmospheric moisture. Most epoxies are sensitive to the concrete's moisture content during painting. Polyamide-cured and water-base epoxy systems, however, have substantially overcome the moisture intolerance problem. For other epoxy systems, the concrete moisture should be measured prior to painting.

Urethanes

Urethanes are usually applied over an epoxy primer. They provide excellent adhesion, hardness, flexibility, and resistance to sunlight, water, harmful chemicals, and abrasion. They are, however, sensitive to temperature and humidity during application. The urethanes used on concrete require moisture to cure. In high humidity, the paint cures too quickly, leaving a bubbly appearance.

Many states now apply moisture-cured urethane anti-graffiti coatings on accessible concrete structures (see Figure 2.2.27). These are smooth, clear coatings applied without a primer coat. Spray paint and indelible marker ink adhere poorly to the smooth urethane, permitting easier cleaning than if they were applied to porous concrete.

Figure 2.2.27 Anti-Graffiti Coating on Lower Area of Bridge Piers

Water Repellent Membranes

Water repellent membranes (sealers) applied to concrete bridge decks, piers, abutments, columns, barriers, or aprons form a tight barrier to water and chlorides. The membrane penetrates up to 10 mm (3/8 inch) into the concrete to give strong adhesion. Membranes have good resistance to abrasion from weathering and traffic. Methyl methacrylate, silane, and silicone are three common water repellent coatings.

Surface Preparation

Concrete, as with any other surface, must be properly cleaned prior to coating. The surface may also require roughening to improve coating adhesion, as the forms used to mold concrete leave a surface that is too smooth for good coating adhesion. In addition, the oils applied to wooden forms to facilitate removal may impede coating adhesion.

Blast Cleaning

Blast cleaning with dry abrasives, high pressure water (up to 380 Mpa (55,000 psi)), or a water/abrasive mix is used to remove dirt, old paint, grease, and deteriorated concrete. It is also the best method to roughen the surface.

Open nozzle blast cleaning uses compressed air free of oil and moisture to propel the abrasive at speeds up to 645 kilometers per hour (400 miles per hour). Centrifugal wheel blast cleaning uses a rotating wheel to propel abrasive. The most common abrasive is sand, although many others, such as steel shot and grit, silica, aluminum oxide, and silicon carbide, are also available. Unlike dry abrasive blast cleaning, high pressure water can penetrate deep into concrete. The concrete must be allowed to dry thoroughly before a coating is applied.

Acid Etching

Acid etching is an efficient method of cleaning concrete. Hydrochloric acid (also called muriatic acid) reacts with the alkaline concrete surface, allowing surface contaminants to easily wash away. It leaves a roughened surface profile for good coating adhesion. All acid must be removed prior to coating application.

Types and Characteristics of Reinforcement Coatings

Because unprotected steel reinforcement corrodes and has adverse effects on concrete, some type of protective coating should typically be used on all steel reinforcement placed in concrete structures to ensure minimal steel corrosion. Steel reinforcement can be protected by the following methods:

- Epoxy coating
- Galvanizing
- Cathodic protection

Epoxy Coating

Epoxy coating is resistant to chemicals, water, and atmospheric moisture. Epoxies utilize an epoxy polymer binder, which forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation, while curing entails a chemical reaction between the coating components. Epoxy coatings have excellent atmospheric exposure characteristics, as well as resistance to chemicals and water. They are often used as the intermediate coat in a three layer paint system. There are also two- and three-layer systems, which use only epoxies. One disadvantage of epoxies is that they chalk when exposed to sunlight. This chalking must be removed prior to topcoating with another layer of epoxy or another material. If not removed, the chalking will compromise subsequent adhesion.

Galvanizing

Another method of protecting steel reinforcement is by galvanizing the steel. This also slows down the corrosion process and lengthens the life of the reinforced concrete. This occurs by coating the bare steel reinforcement with zinc. The two unlike metals form an electrical current between them and one metal virtually stops its corrosion process while the other's accelerates due to the electrical current. In this situation, the steel stops corroding, while the zinc has accelerated corrosion.

Cathodic Protection

Steel reinforcement corrosion can also be slowed down by cathodic protection. Corrosion of steel reinforcing bars in concrete occurs by an electrical process in a moist environment at the steel surface. During corrosion, a voltage difference (less than 1 volt) develops between rebars or between different areas on the same rebar. Electrons from the iron in the rebar are repelled by the negative anode area of the rebar and attracted to the positive cathode area. This electron flow constitutes an electrical current, which is necessary for the corrosion process.

Corrosion occurs only at the anode, where the electrons from the iron are given up.

By cathodic protection, this electrical current is reversed, which slows or stops corrosion. By the impressed current method, an electrical DC rectifier supplies electrical current from local electrical power lines to a separate anode embedded in the concrete. The anode is usually a wire mesh embedded just under the concrete surface. Another type of anode consists of an electrically conductive coating applied to the concrete surface. The wires from the rectifier are embedded in the coating at regular intervals.

When the impressed current enters the mesh or coating anode, the voltage on the rebars is reversed, turning the entire rebar network into a giant cathode. Since natural corrosion occurs only at the anode, the rebars are protected.

The natural corrosion process is allowed to proceed with electrons leaving the iron atoms in the anode. With impressed current cathodic protection, however, the electrons are supplied from an external source, the DC rectifier. Thus, the artificial anode mesh or coating is also spared from corrosion.

During the bridge inspection, check that all visible electrical connections and wiring from the rectifier to the concrete structure are intact.

2.2.10 Inspection Procedures for Concrete and Protective Coatings

There are three basic procedures used to inspect prestressed and reinforced concrete members. Depending on the type of inspection, the inspector may be required to use only one individual procedure or all procedures. They include:

- Visual
- Physical
- Advanced inspection techniques

Visual Examination

There are two types of visual inspections that may be required of an inspector. The first, called a cursory inspection, involves reviewing the previous inspection report and visually examining the members from beneath the bridge. All concrete surfaces should receive a thorough visual assessment to identify obvious defects during a cursory inspection.

The second type of visual inspection is called a “hands-on” inspection. This type of visual inspection requires the inspector to visually assess all defective concrete surfaces at a distance no further than an arm’s length. The concrete surfaces are given close visual attention to quantify and qualify any defects.

Physical Examination

Areas of concrete or rebar deterioration identified visually should also be examined physically using an inspection hammer. This hands-on effort verifies the extent of the defect and its severity.

High stress areas should be sounded for defects using an inspection hammer. Hammer sounding is commonly used to detect areas of delamination and unsound concrete. For large horizontal surfaces such as bridge decks, a chain drag may be used. A chain drag is made of several sections of chain attached to a handle. The inspector drags this across a deck and makes note of the resonating sounds. 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 (see Figure 2.2.28).

The location and width of cracks found during the visual inspection and sounding procedures should be given special attention. For typical reinforced concrete members, a crack comparitor card can be used to measure the width of cracks. This type of crack width measuring device is a transparent card about the size of an identification card. The card has black lines on it that represent crack widths. The line on the card that best matches the width of the crack lets the inspector know the measured width of the crack. For prestressed members, crack widths are usually narrower in width. For this reason, a crack gauge, which is a more accurate crack width-measuring device, should be used. For crack width guidelines, see Table 2.2.3.

Figure 2.2.28: Inspector Using a Chain Drag

Advanced Inspection Techniques

If the extent of the concrete defect cannot be determined by the visual and/or physical inspection procedures described above, advanced inspection techniques should be used. Nondestructive methods, described in Topic 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Destructive methods for protective coatings include:

- Probing
- Paint dry film thickness (Tooke Gauge)

Physical Examination of Protective Coatings

Areas to Inspect

While inspecting protective coatings, pay close attention to the following areas:

- Areas open to direct weathering by wind, rain, hail, or seawater spray.
- Roadway splash zones along curbs, parapets, and expansion dams. These areas are subject to impact abrasion by debris from passing vehicles.
- Inaccessible or hard-to-reach areas where coatings may be missing or improperly applied.
- All concrete joints.
- Areas that retain moisture or salt. Horizontal surfaces of concrete beams and piers are common examples. Also inspect areas where drainage systems deposit salt and water, such as beneath catch basins, scuppers, downspouts, and bearing areas.
- Impact areas on bridge decks and parapets where snowplows or vehicle accidents damage coatings.

Coating Failures

The following failures are characteristic of paint on concrete:

- Lack of adhesion/peeling can be caused by poor adhesion of the primer layer to the concrete or by poor bonding between coating layers. Waterborne salts depositing under a water-impermeable coating (efflorescence) will also cause a coating to peel.
- Chalking is a powdery residue left on paint as ultraviolet light degrades the paint.
- Erosion is a gradual wearing away of a coating. It is caused by abrasion from wind-blown sand, soil and debris, rain, hail, or debris propelled by motor vehicles.
- Checking is composed of short, irregular breaks in the top layer of paint, exposing the undercoat.
- Cracking is similar to checking, but with cracking, the breaks extend completely through all layers of paint to the concrete substrate.
- Microorganism failure occurs as bacteria and fungi feed on paint containing biodegradable components. The damp nature of concrete makes it susceptible to this type of paint failure.
- Saponification results from a chemical reaction between concrete, which is alkaline, and oil-based paint. It destroys the paint, leaving a soft residue.
- Wrinkling is a rough, crinkled paint surface due to excessive paint thickness or high temperature during painting. It is caused by the surface of the paint film at the air interface solidifying before solvents have had a chance to escape from the interior of the paint film.

Table of Contents

Section 2
Bridge Materials

Steel Description	Steel Designation: American Society for Testing and Materials (ASTM)	Steel Designation: American Association of State Highway and Transportation Officials (AASHTO)	Years in Use
Structural Carbon Steel	A7	M94	1900-1967
Structural Nickel Steel	A8	M96	1912-1962
Structural Steel	A36 (A709 Grade 36)	M183 (M270 Grade 36)	1960-Present (1974-Present)
Structural Silicon Steel	A94	M95	1925-1965
Structural Steel	A140		1932-1933
Structural Rivet Steel	A141	M97	1932-1966
High-Strength Structural Rivet Steel	A195	M98	1936-1966
High-Strength Low-Alloy Structural Steel	A242	M161	1941-Present
Low and Intermediate Tensile Strength Carbon Steel Plates	A283		1946-Present
Low and Intermediate Tensile Strength Carbon-Silicon Steel Plates	A284		1946-Present
Steel Sheet Piling	A328	M202	1950-Present
Structural Steel for Welding	A373	M165	1954-1965
High-Strength Structural Steel	A440	M187	1959-1979
High-Strength Low-Alloy Structural Manganese Vanadium Steel	A441	M188	1954-1989
High-Yield-Strength, Quenched and Tempered Alloy Steel Plate (Suitable for Welding)	A514 (A709 Grade 100/100W)	M244 (M270 Grade 100/100W)	1964-Present (1974-Present)
Hi Strength Low-Alloy Columbium-Vanadium Steel of Structural Quality	A572 (A709 Grade 50)	M223 (M270 Grade 50)	1966-Present (1974-Present)
Hi-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 inches Thick	A588 (A709 Grade 50W)	M222 (M270 Grade 50W)	1968-Present (1974-Present)
High-Strength Low-Alloy Steel H-Piles and Sheet Piling	A690		1974-Present
Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi Minimum Yield Strength to 4 inches Thick	A852 (A709 Grade 70W)	M313 (M270 Grade 70W)	1985-Present (1985-Present)

Summary of Steel Designations (Primary Source: Beer and Johnston, *Mechanics of Materials*, New York: McGraw-Hill, 1981)

Topic 2.3 Steel

2.3.1 Introduction

Steel is a widely used construction material for bridges due to its strength, relative ductility, and reliability. It is found in a variety of members on a large number of bridges. Therefore, the bridge inspector should be familiar with the various properties and types of steel.

2.3.2 Common Methods of Steel Member Fabrication

Rolled Beams

Rolled beams are manufactured in structural rolling mills. The flanges and web are one piece of steel. Rolled beams in the past were generally available no deeper than 914 mm (36”) in depth but are now available from some mills as deep as 1120 mm (44”).

Rolled beams are generally compact’ sections which satisfy flange to web thickness ratios to prevent buckling.

Rolled beams generally will have bearing stiffeners but no intermediate stiffeners since they are compact.

Plate Girders

These are larger members than can be provided for in a rolling mill. In other words, larger than 914 mm (36”) or 1120 mm (44”).

These are built-up shapes which are composed of any combination of plates, bars, and rolled shapes. The term “built-up” describes the way the final shape is made.

Older fabricated multi-girders were constructed of riveted built-up members. Today’s fabricated multi-girders are constructed from welded members.

2.3.3 Common Steel Shapes Used in Bridge Construction

Steel as a bridge construction material is available as wire, cable, plates, bars, rolled shapes, and built-up shapes. Typical areas of application for the various types of steel shapes are listed below:

- Wires are typically used as prestressing strands or tendons in beams and girders (See Figure 2.2.4).
- Cable-stay and steel suspension bridges are primarily supported by steel cables (see Figure 2.3.1).
- Steel plates have a wide variety of uses. They are primarily used to construct built-up shapes (see Figure 2.3.2).
- Steel bars are generally placed in concrete to provide tensile reinforcement in the form of deformed round bars. Steel bars can also be used as tension bracing members or with steel plates to construct grid systems (see Figure

2.3.3).

- Rolled shapes are used as structural beams and columns and are made by placing a block of steel through a series of rollers that transform the steel into the desired shape. These steel shapes are either hot rolled or cold rolled. The typical rolled shape is an "I" shape. The "I" shape comes in many sizes and weights (see Figure 2.3.4). They can also be fabricated with a straight or tapered flange thickness. Other rolled shapes are channel or "C" shapes, angles, and "T" shapes.

- Built-up shapes are also used as structural beams and columns but are composed of any combination of plates, bars, and rolled shapes. The term "built-up" describes the way the final shape is made. Built-up shapes are used when an individual rolled shape cannot carry the required load or when a unique shape is desired. Built-up shapes are riveted, bolted, or welded together. Common built-up shapes include I-girders, box girders, and truss members (see Figure 2.3.5).

Figure 2.3.1 Steel Cables

Figure 2.3.2 Steel Plates

Figure 2.3.3 Steel Bars

Figure 2.3.4 Rolled Shapes

Figure 2.3.5 Built-up Shapes

2.3.4
Properties of Steel

Physical Properties

Many of the nation's largest bridges are constructed primarily of steel. When compared with iron, steel has greater strength characteristics, it is more elastic, and it can withstand the effects of impact and vibration better.

Although iron has some carbon chemically dissolved in it, when the carbon content is greater than 0.l%, the material is classified as steel. Steel has a unit weight of about 7850 kg/m^3 (490 pcf).

ASTM and AASHTO define the required properties for various steel types. ASTM classifies each type with an "A" designation, while AASHTO uses an “M” designation.

Low carbon steel, steel with carbon content less than approximately 0.3%, defines some of the most common steel types:

- A7 steel - the most widely used bridge steel up to about 1967; obsolete due to poor weldability characteristics
- A373 steel - similar to A7 steel but has improved weldability characteristics due to controlled carbon content
- A36 steel - the latest of the low carbon steels (first used in 1960); it features good weldability and improved strength

Structural nickel steel (A8) was used widely prior to the 1960's in bridge construction, but welding problems occurred due to relatively high carbon content.

Structural silicon steel (A94) was used extensively in riveted or bolted bridge structures prior to the development of low alloy steels in the 1950's. This steel also has poor weldability characteristics due to high carbon content.

Quenched and tempered alloy steel plate (A514) was developed primarily for use in welded bridge members.

High strength, low alloy steel is used where weight reduction is required, where increased durability is important, and where atmospheric corrosion resistance is desired; examples include:

- A441 steel - manganese vanadium steel
- A572 steel - columbium-vanadium steel (replaced by A441 in 1989)
- A588 steel - a "weathering steel," was developed to be left unpainted, which develops a protective oxide coating upon exposure to the atmosphere under proper design and service conditions (refer to Topic 2.3.5 for a further description of weathering steel)

These steels are also copper bearing, which provides increased resistance to atmospheric corrosion and a slight increase in strength.

Some of the steel types listed above were used widely in the past but are no longer being manufactured. A new ASTM designation (A709) was developed in 1974. This designation covers carbon and high-strength low-alloy steel structural shapes, plates, and bars, and quenched and tempered alloy steel for structural plates intended for use in bridges. Six grades are available in four yield strength levels (36, 50, 70, and 100). The steel grade is equivalent to the yield strength in units of kips per square inch (ksi). Grades 36, 50, 50W, 70W, and 100/100W are also included in ASTM Specifications A36, A572, A588, A852, and A514, respectively. Grades 50W, 70W, and 100W have enhanced atmospheric corrosion resistance and are labeled with a "W" for weathering steel.

In 1996, a new steel type, High Performance Steel (HPS), was introduced to bridge construction. This type of steel was designed to improve weldability, toughness, and atmospheric corrosion resistance. Prior to the new steel designs, a set of "goal properties" was implemented and then testing took place to meet the goals. The first grade of HPS was HPS-70W, which was produced by Thermo-Mechanical-Controlled Processing (TMCP). The HPS-70W has improved Charpy V-Notch impact properties compared to 70W. Currently the HPS grades available are HPS-50W, HPS-70W, and HPS-100W.

In addition to the ASTM steel designations, the American Association of State Highway and Transportation Officials (AASHTO) also publishes its own steel designation (M270). For each ASTM steel designation, there is generally a corresponding AASHTO steel designation. For a summary of the various ASTM and AASHTO steel designations, refer to the table at the beginning of Topic 2.3.

Mechanical Properties

Some of the mechanical properties of steel include:

- Strength - steel is isotropic and possesses great compressive and tensile strength, which varies widely with type of steel
- Elasticity - the modulus of elasticity is nearly independent of steel type and is commonly assigned as 200,000 MPa (29,000,000 psi)
- Ductility - both the low carbon and low alloy steels normally used in bridge construction are quite ductile; however, brittleness may occur because of heat treatment, welding, or metal fatigue
- Fire resistance - steel is subject to a loss of strength when exposed to high temperatures such as those resulting from fire (see Topic 2.3.4 – for specific temperature information)
- Corrosion resistance - unprotected carbon steel corrodes (i.e., rusts) readily; however, steel can be protected
- Weldability - steel is weldable, but it is necessary to select a suitable welding procedure based on the chemistry of the steel
- Fatigue - fatigue problems in steel members and connections can occur in bridges due to numerous live load stress cycles combined with poor weld or connection details

2.3.5 Types and Causes of Steel Deterioration

Corrosion

To properly inspect a steel bridge, the inspector must be able to recognize the various types of steel defects and deterioration. The inspector must also understand the causes of the defects and how to examine them. The most recognizable type of steel deterioration is corrosion (see Figure 2.3.6). Bridge inspectors should be familiar with corrosion since it can lead to a substantial reduction in member capacity. Corrosion is the primary cause of section loss in steel members and is most commonly caused by the wet-dry cycles of exposed steel. When deicing chemicals are present, the effect of moisture is accelerated.

Figure 2.3.6 Steel Corrosion and Complete Section Loss on a Beam Web

Some of the common types of corrosion include:

- Environmental corrosion - primarily affects metal in contact with soil or water and is caused by formation of a corrosion cell due to deicing salt concentrations, moisture content, oxygen content, and accumulated foreign matter such as roadway debris and bird droppings
- Stray current corrosion - caused by electric railways, railway signal systems, cathodic protection systems for pipelines or foundation pilings, DC industrial generators, DC welding equipment, central power stations, and large substations
- Bacteriological corrosion - organisms found in swamps, bogs, heavy clay, stagnant waters, and contaminated waters can contribute to corrosion of metals
- Stress corrosion - occurs when tensile forces expose an increased portion of the metal at the grain boundaries, leading to corrosion and ultimately fracture
- Fretting corrosion - takes place on closely fitted parts which are under vibration, such as machinery and metal fittings, and can be identified by pitting and a red deposit of iron oxide at the interface

Fatigue Cracking

Another type of deterioration is fatigue cracking (see Figure 2.3.7). Fatigue failure occurs at a stress level below the yield stress and is due to repeated loading. Fatigue cracking has occurred in several types of bridge structures around the nation. This type of cracking can lead to sudden and catastrophic failure on certain bridge types. Therefore, the bridge inspector should know where to look and how to recognize early stages of fatigue crack development.

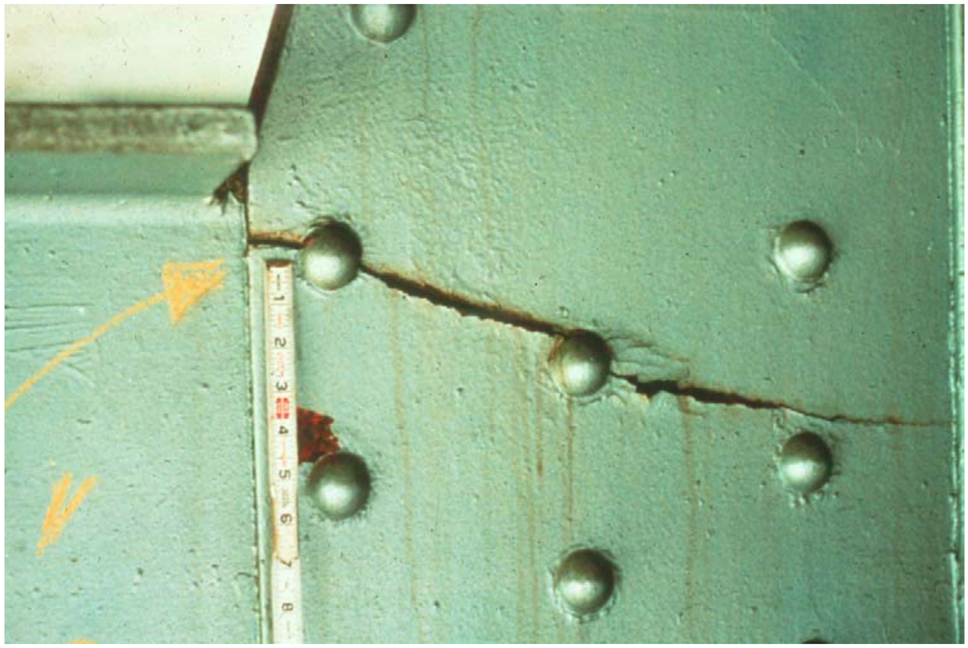


Figure 2.3.7 Fatigue Crack at Coped Top Flange of Riveted Connection

Some factors leading to the development of fatigue cracks are:

- Frequency of truck traffic
- Age or load history of the bridge
- Magnitude of stress range
- Type of detail
- Quality of the fabricated detail
- Material fracture toughness (base metal and weld metal)
- Weld quality
- Ambient temperature

There are two basic types of bending in bridge members: in-plane and out-of-plane. When in-plane bending occurs, the cross section of the member resists the load according to the design and undergoes nominal elastic deformation. Out-of-plane bending implies that the cross section of the member is loaded in a plane other than that for which it was designed and undergoes significant elastic deformation or distortion. More correctly, out-of-plane bending should be referred to as out-of-plane distortion. Out-of-plane distortion is common in beam webs where transverse members connect and can lead to fatigue cracking (see Figure 2.3.8).

There is a distinction between fatigue that is caused from in-plane (as designed) bending and out-of-plane distortion.

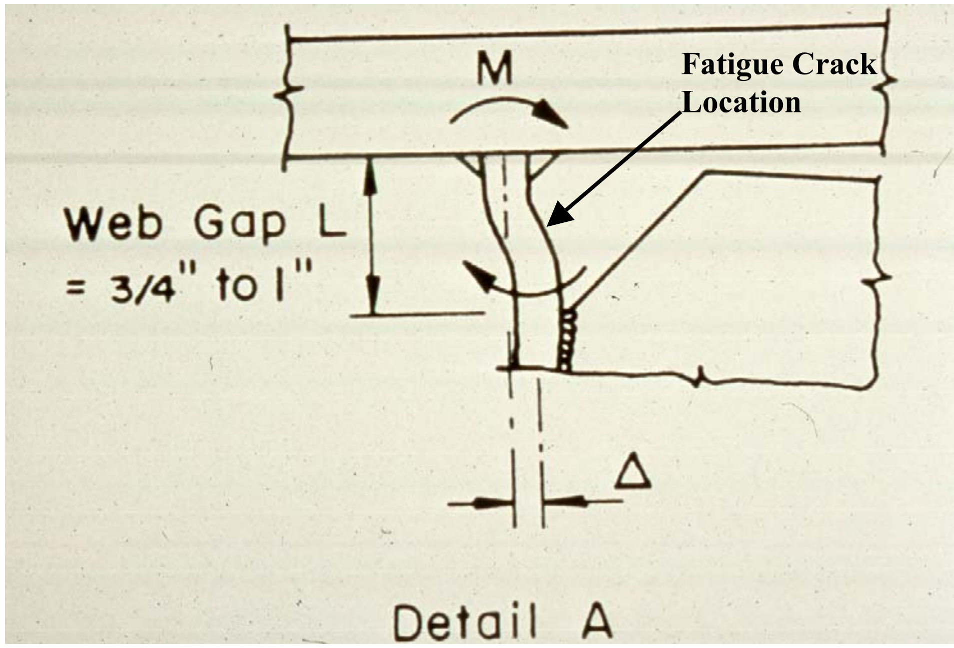


Figure 2.3.8 Out-of-plane Distortion

Additional information about fatigue and fracture in steel bridges is presented in Topic 8.1.

Overloads

Overloads are loads that exceed member or structure design loads. Steel is elastic (i.e., it returns to the original shape when a load is removed) up to a certain point, known as the yield point (see Topic P.1). When yield occurs, steel will bend or elongate and remain bent or elongated after the load has been removed. This type of permanent deformation of material beyond the elastic range is called plastic deformation. Plastic deformations due to overload conditions may be encountered in both tension and compression members.

The symptoms of plastic deformation in tension members are:

- Elongation
- Decrease in cross section, commonly called "necking down"

The symptoms of plastic deformation in compression members are:

- Buckling in the form of a single bow
- Buckling in the form of a double bow or "S" type, usually occurring where the section under compression is pinned or braced at the center point

An overload can lead to plastic deformation, as well as complete failure of the member and structure. This occurs when a tension member breaks or when a compression member exhibits buckling distortion at the point of failure.

Collision Damage

Components and structural members of a bridge that is adjacent to a roadway or waterway are susceptible to impact damage. Indications of impact damage include dislocated and distorted members (see Figure 2.3.9).

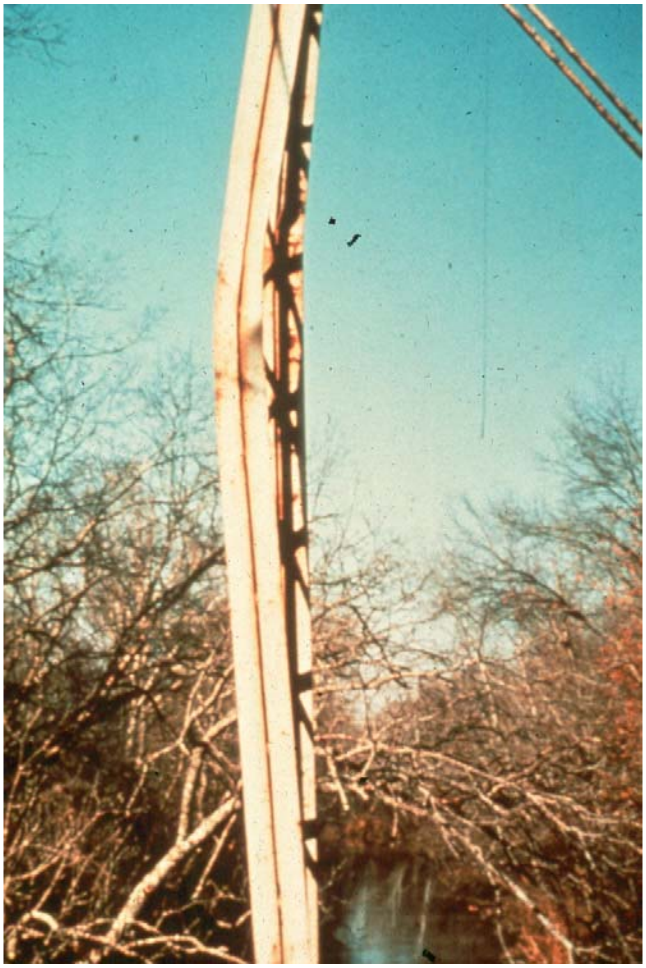


Figure 2.3.9 Collision Damage on a Steel Bridge

Heat Damage

Steel members will undergo serious deformation upon exposure to extreme heat (see Figure 2.3.10). In addition to sagging, or elongation of the metal, intense heat often causes members to buckle and twist; rivets and bolts may fail at connection points. Buckling could be expected where the member is under compression, particularly in thin sections such as the web of a girder.

Figure 2.3.10 Heat Damage

Temperatures affecting steel strength are as follows:

- 204–260°C (400°-500°F) - starts to affect strength
- 482–538°C (900°-1000°F) - major loss of strength

Paint Failures

The following paint failures are common on steel:

- Chalking, erosion, checking, cracking, and wrinkling (see Figure 2.3.11), as described in ASTM D-3359.
- Blisters are caused by painting over oil, grease, water, salt, or by solvent retention. Corrosion can occur under blisters.
- Undercutting occurs when surface rust advances under paint. It commonly occurs along scratches that expose the steel or along sharp edges (see Figure 2.3.12). The corrosion undermines intact paint, causing it to blister and peel.
- Pinpoint rusting can occur at pinholes in the paint, which are tiny, deep holes in the paint, exposing the steel (see Figure 2.3.13). It can also be caused by thin paint coverage. In this case, the "peaks" of the roughened steel surface protrude through the paint and corrode.
- Microorganism failure is caused by bacteria or fungi attacking biodegradable coatings. Oil/alkyds are the most often affected.
- Alligatoring can be considered a widely spaced checking failure, caused by internal stresses set up within the surface of a coating during drying (see Figure 2.3.14). The stresses cause the surface of the coating to shrink more rapidly to a much greater extent than the body of the coating. This causes large surface checks that do not reach the steel substrate.

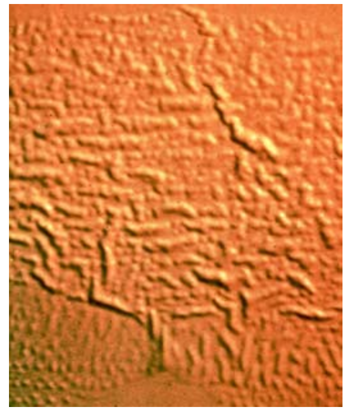


Figure 2.3.11 Paint Wrinkling

Figure 2.3.12 Rust Undercutting at Scratched Area

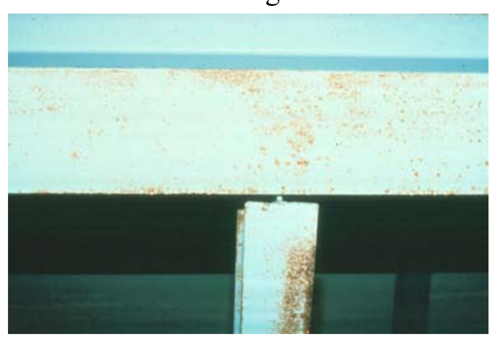


Figure 2.3.13 Pinpoint Rusting

Figure 2.3.14 Paint Peeling from Steel Bridge Members

- Mudcracking can be considered a widely spaced cracking failure, where the breaks in the coating extend to the steel substrate, allowing rapid corrosion (see Figure 2.3.15). Mudcracking is often a phenomenon of inorganic zinc-rich primers, which are applied as a very thick layer or are applied on a hot surface. Rapid curing causes the shrinkage, which yields the alligatoring, and ultimately, mudcracks.
- Bleeding occurs when soluble colored pigment from an undercoat penetrates the topcoat, causing discoloration.

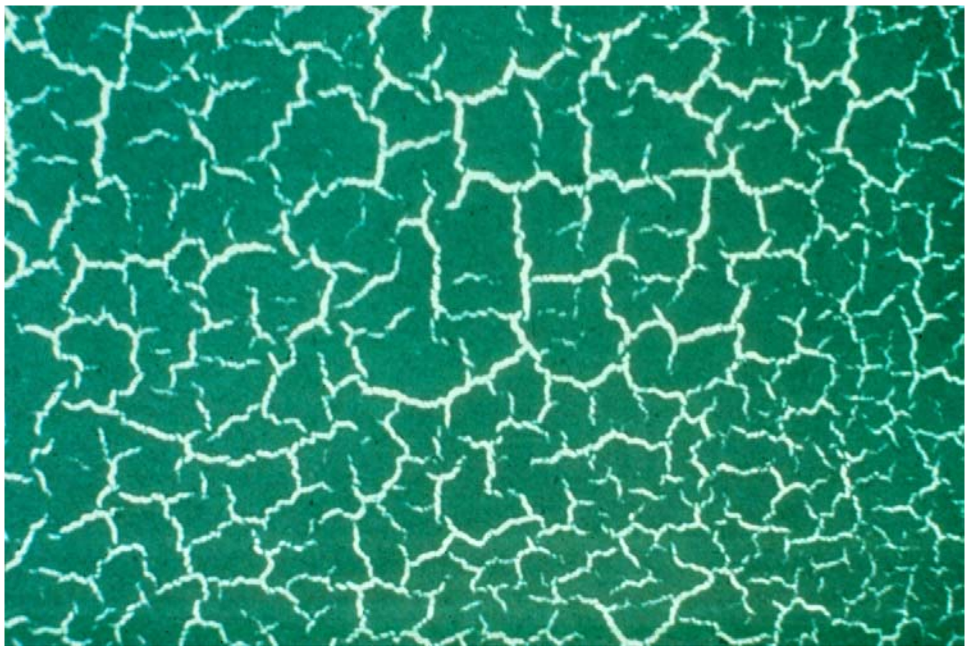


Figure 2.3.15 Mudcracking Paint

2.3.6 Factors that Influence Fatigue Life

Main factors influencing the development of fatigue cracks are:

1. **Magnitude of Stress Range**
   Trucks (not cars) produce stress ranges that may lead to fatigue cracks.

2. **Number of Cycles (Frequency)**
   This is dependent on:
   - Frequency of truck traffic
   - Age or load history of the bridge

3. **Fatigue Prone Details**
   This depends on:
   - The type of detail
   - Quality of the fabricated detail
   - Weld quality

Other factors influencing the development of fatigue cracks are:

Material Fracture Toughness
Toughness of the:
- Base metal
- Weld metal

Toughness is based on the chemical composition of the steel.

Ambient Temperature
- Colder – more likely to crack

Fatigue and fracture will be discussed in further detail in Topic 8.1 and the "Fracture Critical Inspection Techniques for Steel Bridges" NHI Course number 130078.

2.3.7 Protective Systems

Protective systems, when applied properly, provide protection needed against rust or corrosion**.**

Corrosion of Steel

Painting and weathering are the primary means used to protect structural steel from rust and corrosion. To understand how paint or formation of protective rust prevents corrosion and how to inspect paint coatings, it is first necessary to understand the corrosion process.

Corrosion can be defined as a wearing away of metal by a chemical or electrochemical oxidizing process. Corrosion in metals is a form of oxidation caused by a flow of electricity from one part of the surface of one piece of metal to another part of the same piece. The result is the conversion of metallic iron to iron oxide. Once the corrosion process takes place, the steel member has a loss of section which results in a loss of structural capacity. Both conduction and soluble oxygen are necessary for the corrosion process to occur.

A conductive solution (water) or electrolyte must be present in order for current to flow. Corrosion occurs very slowly in distilled water, but much faster in salty water, because the presence of salt (notably sodium chloride) improves the ability of water to conduct electricity and contributes to the corrosion process. In the absence of chlorides, steel (iron) corrodes slowly in the presence of water. Water is both the medium in which corrosion normally occurs and provides the corrosion reaction. In addition, oxygen accelerates the corrosion process. Corrosion stops or proceeds at a reduced rate when access to water and oxygen is eliminated or limited. Water and oxygen are therefore essential for the corrosion process. For example, corrosion of steel does not occur in moisture-free air and is negligible when the relative humidity of the air is below 30% at normal or lower temperatures. The presence of chlorides in the water will accelerate corrosion by increasing the conductivity of the water.

To have corrosion take place in steel, then, one must have:

- Oxygen
- An electrolyte to conduct current
- An area or region on a metallic surface with a negative charge (cathode)

- An area or region on the metallic surface with a positive charge (anode)

Exposure of steel to the atmosphere provides a plentiful supply of oxygen. The presence of oxygen can limit corrosion by the formation of corrosion product films that coat the surface and prevent water and oxygen from reaching the uncorroded steel. The presence of contaminants such as chlorides accelerates the corrosion rate on steel surfaces by disrupting the protective oxide film.

Galvanic Action

The term "galvanic action" is generally restricted to the changes in normal corrosion behavior that result from the current generated when one metal is in contact with a different one. The two metals are in a corrosive solution when one metal may become an anode when it contacts a dissimilar metal. In such a "galvanic couple," the corrosion of one of the metals (e.g., zinc) will be accelerated, and the corrosion of the other (e.g., steel) will be reduced or possibly stopped. Galvanized coatings on highway guardrails and zinc-rich paint on structural steel are examples of galvanic protection using such a sacrificial (zinc) anode.

Types and Characteristics of Steel Coatings

Surface Preparation for Painting

The steel surface must be properly prepared prior to paint application. All foreign material must be removed. The following steps must be taken when preparing the surface for painting:

- Dirt and dust particles or spent abrasive from blast cleaning interfere with paint adhesion to the steel substrate and prevent application of a smooth, uniform film of paint. Debris embedded in the paint can also wick moisture and corrosive elements through the film to the substrate.
- Rust cannot be penetrated by most paints. Rust can become poorly adherent. In such cases, disbonding of the rust carries away the paint layers, permitting accelerated corrosion.
- Flash rust is a light layer of rust, which forms on the cleaned steel soon after exposure to the air, particularly in moist or humid environments. This layer may not be thoroughly wetted and may impede adhesion.
- Salts trapped in the paint film can cause blistering and disbonding.
- Oil and grease prevent good paint adhesion and must be completely removed. Welding smoke and inspection markings leave an oily residue that must also be completely removed.
- Dead paint that is loose, cracking, or flaking will eventually lift from the surface, carrying any new paint with it.
- Mill scale is a layer of iron oxide on the surface of steel. It forms when the steel is heated at high temperatures in a furnace. Mill scale has a bluish, somewhat shiny appearance, which may be difficult to see on partially blastcleaned steel. It must be completely removed when using most coating materials, as it may disbond upon expansion and contraction, carrying the paint with it.
- Weld spatter may also dislodge, leaving a bare exposed steel surface.

The surface should also be roughened to promote paint adhesion, as paint will not adhere well to a smooth surface.

Methods of Surface Preparation

The Society of Protective Coatings publishes a set of standards and specifications describing the following methods of surface preparation:

- Solvent cleaning
- Hand tool cleaning
- Power tool cleaning
- Abrasive blast cleaning
- Water blast cleaning

Solvent cleaning removes oil and grease. It is usually used in conjunction with or prior to the mechanical preparation methods. Common solvents include petroleum and coal tar solvents, turpentine, mineral spirits, alkaline cleaners, and emulsion cleaners, which contain oil soaps mixed with kerosene or mineral spirits.

Hand tool cleaning is used for removing loosely adhering paint, rust, or mill scale. It will not remove tightly adhering mill scale, or dirt and oils in crevices. Due to its slow speed, hand tool cleaning is used mostly for small area spot cleaning. Common hand tools include scrapers, wire brushes, chipping hammers, knives, chisels, and abrasive pads.

Power tool cleaning is effective on both plane and contoured steel surfaces. Power tool cleaning devices remove loose paint, rust, and scale. Power tools do not leave the residue common with blast cleaning. Also, power tools are used on small areas and where the abrasive could damage sensitive surroundings.

Abrasive blast cleaning is the preferred surface preparation method for coatings, which require a high degree of cleanliness and a uniformly roughened surface profile. Blast cleaning is a high production method, which can remove mill scale. A water collar is sometimes used with abrasive blast cleaning to prevent abrasive rust and paint particles from becoming airborne.

Water blast cleaning (hydroblasting) may be high or low pressure, hot or cold, with or without detergent, depending upon the type of cleaning desired. Water does not etch a steel surface and may not remove tight paint, rust, or mill scale. Abrasives may be injected into the water stream to remove tightly adhering material for faster clearing or to produce a roughened surface profile. Sand is the most common abrasive. The process can remove all old paint, rust, and mill scale. It yields a degree of cleanliness equivalent to open nozzle abrasive blast cleaning. Due to flash rusting caused by the high pressure water, water blast cleaned areas must be either cleaned by dry abrasive blast cleaning or a corrosion inhibiting chemical must be added to the high pressure water to prevent flash rusting.

Paint

Once the steel surface is properly prepared, the appropriate type and application of paint must be chosen based on the paint characteristics.

Paint is by far the most common coating used to protect steel bridges. Paint is composed of four basic compounds: pigments, vehicle (also called binder), solvents (also called thinners), and additives (such as thickeners and mildewoides). The pigments contribute such properties as inhibition of corrosion of the metal surface (e.g., zinc, zinc oxide, red lead, and zinc chromate), reinforcement of the dry paint film, stabilization against deterioration by sunlight, color, and hardness. Pigments are generally powders before being mixed into paint. The vehicle also remains in the dry-cured paint layer. It binds the pigment particles together and provides adhesion to the steel substrate and to other paint layers. Thus, the strength of the binder contributes to the useful life of the coating. Paint can be classified as inorganic or organic, depending on the vehicle. Inorganic paint uses a water soluble silicate binder which reacts with water during paint curing. Most types of paint contain one of a variety of available organic binders. The organic binders cure (harden) by one or more of the following mechanisms:

- Evaporation of solvents
- Reaction with oxygen in the air
- Polymerization through the action of heat or a catalyst
- Combination of reactive components in the binder

Solvents, which are liquids (such as water and mineral spirits), are included in paint to transport the pigment-binder combination to the substrate, to lower paint viscosity for easier application, to help the coating penetrate the surface, and to wet the substrate. Since the solvent is volatile, it eventually evaporates from the dry paint film. Additives are special purpose ingredients that give the product extra performance features. For example, mildewoides reduce mildew problems, and thickeners lengthen the drying time for application in hot weather.

Paint used on steel bridges acts as a physical barrier to moisture, oxygen, and chlorides, all of which promote corrosion. While water and oxygen are important to corrosion, chlorides from deicing road salts or seawater spray accelerate the corrosion process significantly.

Paint Layers

Paint on steel is usually applied in up to three layers, or coats:

- Primer coat
- Intermediate coat
- Topcoat

The primer coat is in direct contact with the steel substrate. It is formulated to have good wetting and bonding properties and may or may not contain passivating (corrosion-inhibiting) pigments.

The intermediate coat must strongly adhere to the primer. It provides increased thickness of the total coating system, abrasion and impact resistance, and a barrier to chemical attack.

The topcoat (also called the finish coat) is typically a tough, resilient layer, providing a seal to environmental attack, water, impact, and abrasion. It is also formulated for an aesthetic appearance.

Types of Paint

A wide variety of paints are applied to steel bridges. All of them except some zinc-rich primers use an organic binder.

Oil/alkyd Paint

Oil/alkyd paints use an oil such as linseed oil and an alkyd resin as the binding agent. Alkyd resin is synthetically produced by reacting a drying oil acid with an alcohol. Alkyd paints are low cost, with good durability, flexibility, and gloss retention. They are also tough, with moderate heat and solvent resistance. They should not be used in water immersion service or in alkaline environments.

A disadvantage is their offensive odor. They are also slow drying, difficult to clean up, and have poor exterior exposure. Alkyd paints often contain lead pigments, which are known to cause numerous health problems. The removal and disposal of lead-based paints is a regulated activity in all states.

Vinyl Paint

Vinyl paints are based on various vinyl polymer binding agents dissolved in a strong solvent. These paints cure by solvent evaporation. Vinyls have excellent chemical, water, salt, acid, and alkali resistance, good gloss retention, and are applicable at low temperatures. Conversely, their disadvantages include poor heat and solvent resistance, and poor adhesion. Vinyls are usually not used with other types of paint in a paint system. Vinyl coatings can be formulated to serve as primer, intermediate, and topcoat in paint systems.

Epoxies

Epoxies utilize an epoxy polymer binder, which forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation, while curing entails a chemical reaction between the coating components. Epoxy coatings have excellent atmospheric exposure characteristics, as well as resistance to chemicals and water. They are often used as the intermediate coat in a three-layer paint system. There are also two- and three-layer systems, which use only epoxies. One disadvantage of epoxies is that they chalk when exposed to sunlight. This chalking must be removed prior to topcoating with another layer of epoxy or another material. If not removed, the chalking will compromise subsequent adhesion.

Epoxy Mastics

Epoxy mastics are heavy, high solid content epoxy paints, often formulated with flaking aluminum pigment. The mastics are useful in applications where a heavy paint layer is required in one application. They can be formulated with wetting and penetrating agents, which permit application on minimally prepared steel surfaces.

Urethanes

Urethanes are commonly used as the topcoat layer. They provide excellent sunlight resistance, hardness, flexibility (i.e., resistance to cracking), gloss retention, and resistance to water, harmful chemicals, and abrasion. All-urethane systems are also available which utilize urethane paints as primer, intermediate, and topcoat.

Zinc-rich Primers

Zinc-rich primers contain finely divided zinc powder (75% to 95%) and either an organic or inorganic binder. They protect the steel substrate by galvanic action, wherein the metallic zinc corrodes in preference to the steel. The materials have excellent adhesion and resist rust undercutting when applied over a properly prepared surface. The zinc-rich primers must be well mixed prior to application, or some coated areas will be deficient in zinc, lowering the substrate protection.

Latex Paint

Latex paint consists of a resin emulsion. The term covers a wide range of materials, each formulated for a different application. Latex on steel has excellent flexibility (allowing it to expand and contract with the steel as the temperature changes) and color retention, with good adhesion, hardness, and resistance to chemicals. Latex paint has low odor, faster drying time, and easier clean up.

The disadvantages of latex paint include sacrificed durability, and it must be applied at temperatures over 10°C (50°F).

It is important to document the existing paint system on a bridge. The paint type may be shown on the bridge drawings or specifications. Some agencies list the paint type and application date on the bridge. Once the existing paint is determined, a compatible paint for any required rehabilitation can be chosen.

Protection of Suspension Cables

Suspension cables of steel suspension bridges are particularly difficult to protect from corrosion. One method is to wrap the cables with a neoprene elastomeric cable wrap system or with a glass-fabric-reinforced plastic shell. In some cases, the elastomeric cable wrap has retained water and accelerated corrosion. Another method is to pour or inject paints into the spaces between the cable strands. Commonly, inhibitive pigments, such as zinc oxide, in an oil medium are used. Red lead pigment was commonly used in the past. Lead constitutes a significant health hazard, and care must be exercised care when inspecting cables. Do not inhale or ingest old paint. The paint on the exterior surface of a suspension cable dries, but the paint on the interior, surrounding individual strands, stays in the liquid, uncured state for years. The exterior of the cable is often topcoated with a different paint, such as an aluminum pigmented oil-based paint. Another option to protect suspension cables is to wrap tightly with small diameter wires. This allows the cable to “breath” while still providing a protective cover.

A newer technique used to resist the corrosion process of suspension cables is forced air dehumidification. On larger structures (such as the Kobe Bridge in Japan and the Ben Franklin Bridge in Pennsylvania), dry air is passed through the cables, which does not allow the steel to be exposed to moisture. For this protection system to work, the relative humidity of the forced air should be less than approximately 60%.

Weathering Steel

In the proper environments, weathering steel does not require painting but produces its own protective coating. When exposed to the atmosphere, weathering steel develops a protective oxide film, which seals and protects the steel from further corrosion. This oxide film is actually an intended layer of surface rust, which protects the member from further corrosion and loss of material thickness.

Weathering steel was first used for bridges in 1964 in Michigan. Since then, thousands of bridges have been constructed of weathering steel in the United States. The early successes of weathering steel in bridges led to the use of this steel in locations where the steel could not attain a protective oxide layer and where corrosion progressed beyond the intended layer of surface rust. Therefore, it is important for the inspector to distinguish between the protective layer of rust and advanced corrosion that can lead to section loss. It is also important to note that fatigue cracks can initiate in rust pitted areas of weathering steel.

The frequency of surface wetting and drying cycles determines the oxide film’s texture and protective nature. The wetting cycle includes the accumulation of moisture from rainfall, dew, humidity, and fog, in addition to the spray of water from traffic. The drying cycle involves drying by sun and wind. Alternate cycles of wetting and drying are essential to the formation of the protective oxide coating. The protective film will not form if weathering steels remain wet for long periods of time.

Uses of Weathering Steel

Weathering steels may be unsuitable in the following environments:

- Areas with frequent high rainfall, high humidity, or persistent fog
- Marine coastal areas where the salt-laden air may deposit salt on the steel, which leads to moisture retention and corrosion
- Industrial areas where chemical fumes may drift directly onto the steel and cause corrosion
- Areas subject to “acid rain” which has a sulfuric acid component

The location and geometrics of a bridge also influence performance of weathering steel. Locations where weathering steel may be unsuitable include:

- Tunnel-like situations which permit concentrated salt-laden road sprays,

caused by high-speed traffic passing under the bridge, to accumulate on the superstructure
- Low level water crossings where insufficient clearance over bodies of water exists so that spray and condensation of water vapor result in prolonged periods of wetness

2.3.8 Inspection Procedures for Steel

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

- Visual
- Physical
- Advanced inspection techniques

Visual Examination

Steel Members

Fatigue

Steel members should be inspected for corrosion, section loss, buckling, and cracking.

Some common inspection locations and signs of fatigue distress include:

- Bent or damaged members - determine the type of damage (e.g., collision, overload, or fire), measure the variance from proper alignment, and check for cracks, tears, and gouges near the damaged location
- Corrosion, which could reduce structural capacity through a decrease in member section and make the member less resistant to both repetitive and static stress conditions; since rust continually flakes off of a member, the severity of corrosion cannot always be determined by the amount of rust; therefore, corroded members must be examined by physical as well as visual means (see Figure 2.3.16)
- Fatigue cracks - fatigue cracks are common at certain locations on a bridge, and certain inspection procedures should be followed when fatigue cracks are observed (see Figure 2.3.17 and Topic 8.1 for additional information about fatigue cracks)
- Other stress-related cracks - determine the length, size, and location of the crack
- Points on the structure where a discontinuity or restraint is introduced
- Loose members which could force the member or other members to carry unequal or excessive stress
- Damaged members, regardless of damage magnitude, which are misaligned, bent, or torn
- Welded details
- Repairs that show indiscriminate welding or cutting procedures
- Areas of excessive vibrations or twisting

Inspection procedures for in-plane fatigue cracks:
- Report the fatigue crack immediately

- Determine the visual ends of the crack
- Examine other identical details on the bridge for cracks
- Examine other details for breaks in the paint and the formation of oxide (rust)
- If a suspect area is located, a more detailed examination, such as blast cleaning and using dye penetrant or ultrasonic testing, is required

Figure 2.3.16 Corrosion of Steel

Figure 2.3.17 Fatigue Crack

Protective Coatings

Knowing where to inspect is just as important as knowing how to inspect.

Areas to Inspect

Rust typically starts in a few characteristic places, then spreads to larger areas.

Examine sharp edges and square corners of structural members (see Figure 2.3.18). Paint is generally thinner at sharp edges and corners than at rounded edges and corners or flat surfaces. Rusting starts at sharp edges, then undercuts intact paint as it spreads away from the edge. Inside square corners often receive an extra thick layer of paint due to double or triple passes made over them. Extra thick layers are prone to cracking, exposing the steel. It is difficult to completely remove dirt and spent blast cleaning abrasive from inside corners. Painting over this foreign material results in early peeling and corrosion.

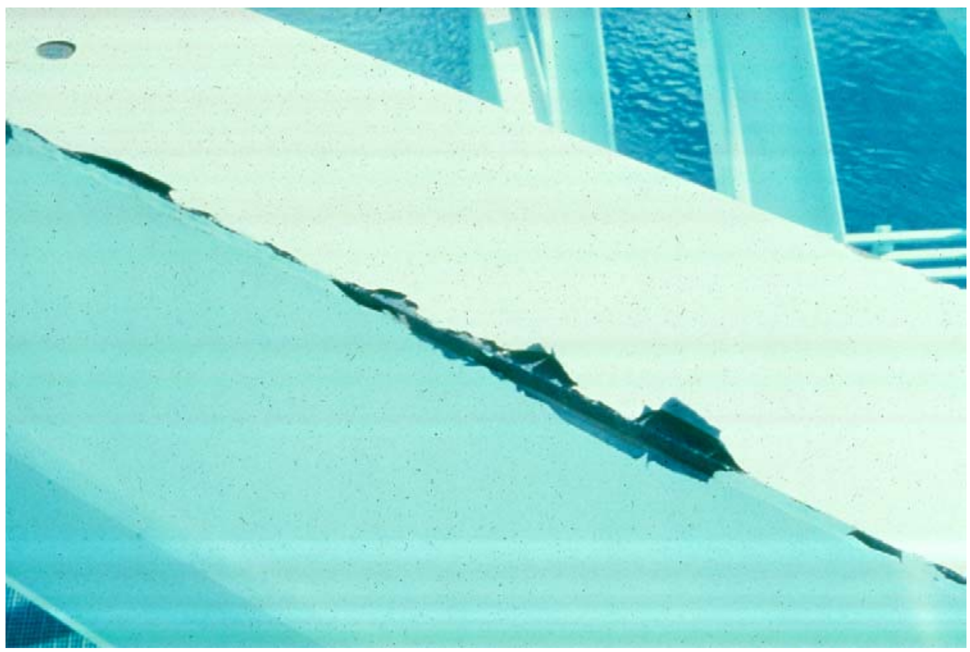


Figure 2.3.18 Edge Failure on Painted Steel Beam

Examine all areas that retain moisture and salt. Check under scuppers and beneath downspouts. Check horizontal surfaces under the edge of bridge decks and under expansion dams, where roadway deicing salt runoff collects (see Figure 2.3.19). Examine the bottom inside flange of girders.

Inspect inaccessible or hard-to-reach areas that may have been missed during painting. A flashlight and inspection mirror may be needed here. Examine the inside surfaces of lattice girders and beams. Examine the top surface of girder upper flanges under the bridge deck, if possible.

Inspect around bolts, rivets, and pins (see Figure 2.3.20). Rust detected around the heads may indicate corrosion along the entire length of the bolt, rivet, or pin, causing reduced structural integrity.

Examine roadway splash zones, where debris and corrosive deicing salt-laden water are directly deposited on painted members by passing traffic (see Figure 2.3.21). On through-truss bridges, this includes some bracing members above the roadway.

Examine areas exposed to wind and rain, seawater spray, and other adverse weather conditions.

Figure 2.3.19 Water and Salt Runoff Under Expansion Dam Deck Opening

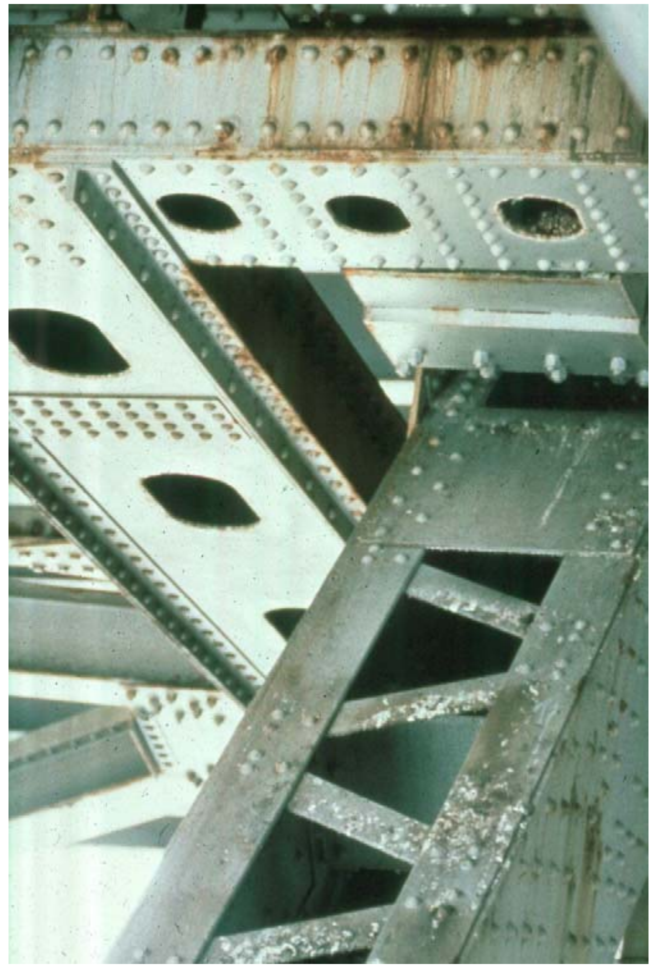


Figure 2.3.20   Corroding Rivet Head

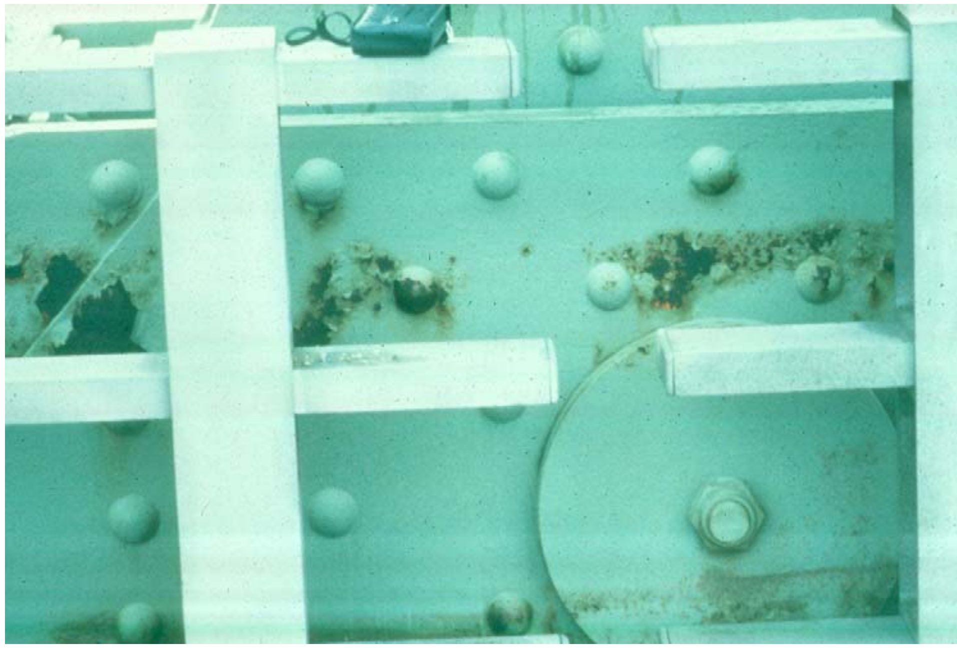


Figure 2.3.21 Roadway Splash Zone Damage (Note Aluminum Bridge Railing in Foreground)

Weathering Steel

It is particularly important for weathering steel to be inspected in the following locations:

- Where water ponds or the steel remains damp for long periods of time due to rain, condensation, leaky joints, or traffic spray
- Where debris is likely to accumulate
- Where the steel is exposed to salts and atmospheric pollutants
- Near defective joints or drainage devices

Color

The color of the surface of weathering steel is an indicator of the protective oxide film (see Figure 2.3.22). The color changes as the oxide film matures to a fully protective coating. Figures 2.3.23 through 2.3.26 correlate the color of the weathering steel and the degree of protection.

Figure 2.3.22 Color of Oxide Film is Critical in the Inspection of Weathering Steel; Dark Black Color in an Indication of Non-protective Oxide

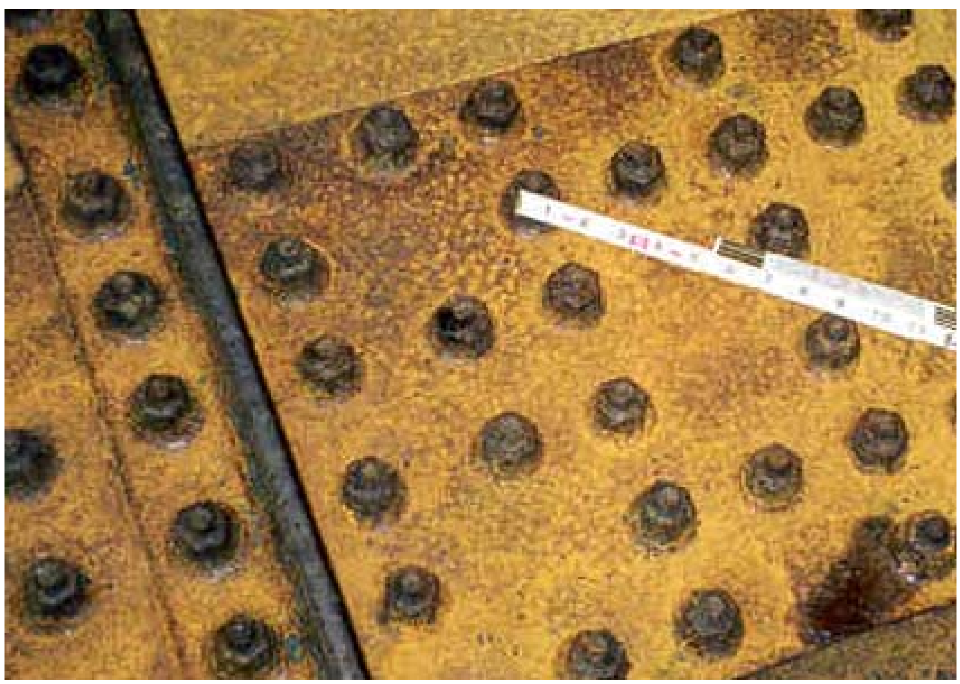


Figure 2.3.23 Yellow Orange – Early Stage of Exposure or Active Corrosion

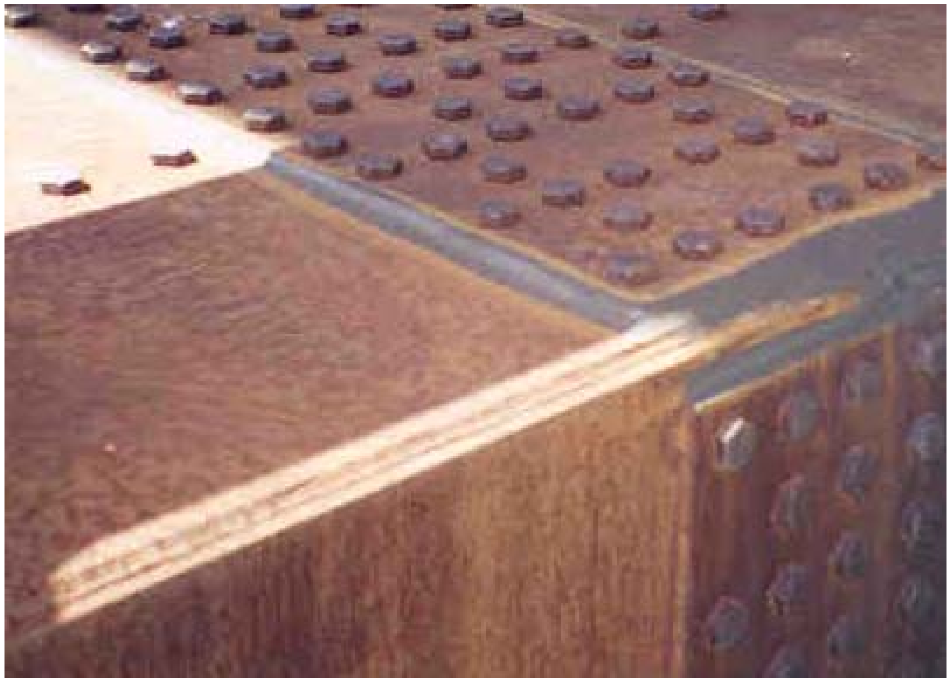


Figure 2.3.24 Light Brown – Early Stage of Exposure

Figure 2.3.25 Chocolate Brown to Purple Brown - Boldly Exposed and Good Degree of Protection

Figure 2.3.26 Black – Non-protective Oxide

An area of steel, which is a different color than the surrounding steel indicates a potential problem. The discolored area should be investigated to determine the cause of the discoloration. Color photographs are an ideal way to record the color of the weathering steel over time. A color coupon should be included in each photograph to enable comparison.

Texture

The texture of the oxide film also indicates the degree of protection of the film. An inspection of the surface by tapping with a hammer and vigorously brushing the surface with a wire brush determines the adhesion of the oxide film to the steel substrate. Surfaces, which have granules, flakes, or laminar sheets are examples of non-adhesion. Table 2.3.1 presents a correlation between the texture of the weathering steel and the degree of protection.

Appearance	Degree of Protection
Tightly adhered, capable of withstanding hammering or vigorous wire brushing	Protective oxide
Dusty	Early stages of exposure; should change after few years
Granular	Possible indication of problem, depending on length of exposure and location of member
Small flakes, 6 mm (1/4 inch) in diameter	Initial indication of non-protective oxide
Large flakes, 13 mm (1/2 inch) in diameter or greater	Non-protective oxide
Laminar sheets or nodules	Non-protective oxide, severe conditions

Table 2.3.1 Correlation Between Weathering Steel Texture and Condition

Physical Examination

Steel Members

Once the defects are identified visually, physical procedures must be used to verify the extent of the defect. For steel members, the main physical inspection procedures involve an inspection hammer and wire brush. Corrosion results in loss of member material. This partial loss of cross section due to corrosion is known as section loss. Section loss may be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or a D-meter, should be used to measure the remaining section of steel. The inspector must remove all corrosion products (rust scale) prior to making measurements.

The inspector should measure the bridge members to verify that the sizes recorded in the plans or inspection report are accurate. If incorrect member sizes are used, then any load rating analysis for safe load capacity of the bridge is worthless.

Protective Coatings

The degree of coating corrosion must be assessed during the inspection. Coating corrosion is measured differently than structural corrosion. There are a variety of proprietary procedures which use a set of photographic standards to evaluate and categorize the degree and extent of coating corrosion on composite spans, cross frames, exterior fascias, and bearings. A simple method entails evaluation of painted surfaces in accordance with SSPC-Vis 2. Vis 2 is a pictorial standard for evaluating the degree of rusting on painted steel surfaces.

Mill Scale

Incomplete removal of mill scale can provide a starting point for corrosion. When mill scale cracks, it allows moisture and oxygen to reach the steel substrate. Mill scale accelerates corrosion of the substrate because of its electrochemical properties. To check for mill scale corrosion during a paint inspection, use a knife to remove a small patch of paint in random spots. Inspect the exposed surface for mill scale, either intact or rusted. Probe with a knife or other sharp object at weld spatter to check for rusting. Re-coat areas where paint is removed.

Invisible microscopic chloride deposits from deicing salt or seawater spray may permeate a corroding steel surface. Painting over a partially cleaned chloride-contaminated surface simply seals in the contaminant. Salt deposits draw moisture through the paint by osmosis, and corrosion will continue.

Paint Adhesion

Paint can undergo adhesion failure between paint layers or between the primer and steel. Some bridge painting contracts specify a minimum acceptable paint adhesion strength for new paint. Over time, however, adhesion strength may degrade as the paint weathers and is affected by sunlight, or as rusting occurs under the paint.

The simplest test of adhesion is to probe under paint with the point of a knife. A more quantitative evaluation is performed by a tape test, as described in Topic 2.1.

Paint Dry Film Thickness

There are a variety of instruments to measure the dry film thickness of paint applied to steel. Accuracy ranges from 10% +/- to 15% +/-, and they fall into three classes:

- Magnetic pull-off
- Fixed probe
- Destructive test

The magnetic pull-off dry film thickness gages use the attractive force between a magnet and the steel substrate to determine the paint thickness. The thicker the paint, the lower the magnetic force. These instruments must be calibrated prior to and during use with plastic shims of known thickness, or with ferrous plates coated with a non-ferrous layer. Such shims are produced by the National Bureau of Standards (NBS).

The fixed probe gages also use a magnet. Measurement of paint thickness is done by an electrical measurement of the interaction of the probe's magnetic field with the steel rather than by the force to move the magnet. They are normally calibrated with plastic shims. Neither the magnetic pull-off nor fixed probe gages can be used closer than one inch to edges, as this will distort the reading. SSPC-PA2 "Measurement of Dry Paint Thickness With Magnetic Gages" provides a detailed description of how to calibrate and take measurements using magnetic gages.

A destructive method for measuring dry film thickness uses the Tooke Gage described in Topic 2.1. An advantage of this method is that it can be used at any location, including close to edges. While the magnetic gages measure the combined thickness of all paint layers, the Tooke Gage measures each layer individually. Limitations of the destructive test are that only coatings up to 50 mils thick can be measured and multiple layers of the same color cannot be distinguished.

Repainting

If the coating is to be repainted, the type of in-place paint must be known, since different type paints may not adhere to each other. Methods described in Topic 2.1 can be used to determine the type of in-service paint.

Weathering Steel

Weathering steel with any of the following degree of protection should be inspected:

- Laminar texture of steel surface, such as slab rust or thin and fragile sheets of rust
- Granular and flaky rust texture of steel surface
- A very coarse texture
- Large granular (3 mm (1/8 inch) in diameter) texture
- Flakes (13 mm (1/2 inch) in diameter)
- Surface rubs off by hand or wire brush revealing a black substrate
- Surface is typically covered with deep pits

If such conditions are discovered, the following steps should be taken to determine the adequacy of the oxide film:

- Scrape the surface of the steel to the bare metal
- Check to determine the extent of pitting
- Measure the metal section loss with calipers or an ultrasonic thickness gauge

It is important to set a benchmark at the point where the metal thickness measurement is taken so that any metal loss may be monitored with future measurements. Benchmarks are important since steel rolled sections and steel plates often vary within acceptable tolerances in thickness from the nominal thickness values.

Data obtained from the inspection should include visual observations of the steel (e.g., color, texture, and flaking), physical measurements with a thickness gauge, and observation of environmental conditions.

Advanced Inspection Techniques

In addition, several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Acoustic emissions testing
- Computer programs
- Computer tomography
- Corrosion sensors

- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 13.3.3, include:

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

2.3.9 Other Bridge Materials

Cast Iron

Iron is an elemental metal smelted from iron ore. Cast iron is the most widely used cast metal. However, it is easily fractured by shocks and has low tensile strength due to a large percentage of free carbon and slag. Consequently, it is basically a poor bridge construction material and is not used in new bridge construction today. It may, however, be found in compression members of old bridges.

Cast iron is gray in color due to the presence of tiny flake-like particles of graphite (carbon) on the surface. It has a unit weight of about 7210 kg/m^3 (450 pcf).

Properties of Cast Iron

Some of the mechanical properties of cast iron include:

- Strength - tensile strength varies from 172 MPa (25,000 psi) to 345 MPa (50,000 psi), while compressive strength varies from 448 MPa (65,000 psi) to 1,035 MPa (150,000 psi)
- Elasticity - cast iron has an elastic modulus of 89,635 MPa (13,000,000 psi) to 206,850 MPa (30,000,000 psi): elasticity increases with a decrease in carbon content
- Workability - cast iron possesses good machinability, and casting is relatively easy and inexpensive
- Weldability - cast iron can not be effectively welded due to its high free carbon content
- Corrosion resistance - cast iron is generally more corrosion resistant than the other ferrous metals
- Brittleness - cast iron is very brittle and prone to fatigue-related failure when subjected to bending or tension stresses

Types of Cast Iron Deterioration

The primary forms of deterioration in cast iron are similar to those in steel (refer to Topic 2.3.4).

Wrought Iron

When iron is mechanically worked or rolled into a specific shape, it is classified as wrought iron. This process results in slag inclusions that are embedded between the microscopic grains of iron. It also results in a fibrous material with properties in the worked direction similar to steel. Wrought iron is no longer made in the United States. However, wrought iron tension members still exist on some older bridges, and it was well-suited for use in the early suspension bridges.

Properties of Wrought Iron

Some of the mechanical properties of wrought iron include:

- Strength - wrought iron is anisotropic (i.e., its strength varies with the orientation of its grain) due to the presence of slag inclusions; compressive strength is about 241 MPa (35,000 psi), while tensile strength varies between 248 MPa (36,000 psi) and 345 MPa (50,000 psi)
- Elasticity - modulus of elasticity ranges from 165,000 MPa (24,000,000 psi) to 200,000 MPa (29,000,000 psi), nearly as high as steel
- Impact resistance - wrought iron is tough and is noted for impact and shock resistance
- Workability - wrought iron possesses good machinability
- Weldability - wrought iron can be welded, but care should be exercised when welding the metal of an existing bridge
- Corrosion resistance - the fibrous nature of wrought iron produces a tight rust which is less likely to progress to flaking and scaling than is rust on carbon steel
- Ductility - wrought iron is generally ductile; reworking the wrought iron causes a finer and more thread-like distribution of the slag, thereby increasing ductility

Types of Wrought Iron Deterioration

The primary forms of deterioration in wrought iron are similar to those in steel (refer to Topic 2.3.4).

Aluminum

Aluminum is widely used for signs, light standards, railings, and sign structures. Aluminum is seldom used as a primary material in the construction of vehicular bridges.

Properties of Aluminum

The properties of aluminum are generally similar to those of steel (refer to Topic 2.3.3). However, a few notable differences exist:

- Weight - aluminum alloy has a unit weight of about 2800 kg/m^3 (175 pcf)
- Strength - aluminum is not as strong as steel, but alloying can increase its strength to that of steel

- Corrosion resistance - aluminum is highly resistant to atmospheric corrosion
- Workability - aluminum is easily fabricated, but welding of aluminum requires special procedures
- Durability - aluminum is durable
- Expense - aluminum is more expensive than steel

Types of Aluminum Deterioration

The primary forms of deterioration in aluminum are:

- Fatigue cracking - the combination of high stresses and vibration caused by wind produces fatigue
- Pitting - aluminum can pit slightly, but this condition rarely becomes serious

This page intentionally left blank

Table of Contents

Section 3

Fundamentals of Bridge Inspection

Section 3
Fundamentals of Bridge Inspection

Topic 3.1 Duties of the Bridge Inspection Team

3.1.1 Introduction

Bridge inspection is playing an increasingly important role in providing a safe infrastructure for our nation. As our nation's bridges continue to age and deteriorate, an accurate and thorough assessment of each bridge's condition is critical in maintaining a dependable highway system.

There are five basic types of inspection:

- Initial (inventory)
- Routine (periodic)
- Damage
- In-depth
- Special (interim)

These are discussed in Article 3.2 of the AASHTO *Manual for Condition Evaluation of Bridges*. Although this topic is organized for "in-depth" inspections, it applies to all inspection types. However, the amount of time and effort required for performing each duty will vary with the type of inspection performed.

This section presents the duties of the bridge inspection team. It also describes how the inspection team can prepare for the inspection and some of the major inspection procedures. For some duties, the inspection program manager may be involved.

3.1.2 Duties of the Bridge Inspection Team

There are five basic duties of the bridge inspection team:

- Planning the inspection
- Preparing for the inspection
- Performing the inspection
- Preparing the report
- Identifying items for repairs and maintenance

The duties of the inspector are simply the tasks that must be performed in order to fulfill the responsibilities that come with the job.

3.1.3 Planning the Inspection

In order to make the inspection orderly and systematic, the lead inspector should make plans in advance. Planning the inspection is necessary for an efficient, cost-effective effort which will also result in a thorough and complete inspection.

Basic activities include:

- Determination of the type of inspection
- Selection of the inspection team
- Evaluation of required activities (e.g., nondestructive testing and underwater inspection)
- Development of an inspection sequence
- Establishment of a schedule

3.1.4 Preparing for the Inspection

Preparation measures needed prior to the inspection include organizing the proper tools and equipment, reviewing the bridge structure files, and locating plans for the structure. The success of the on-site field inspection is largely dependent on the effort spent in preparing for the inspection. The major preparation activities include:

- Reviewing the bridge structure file
- Identifying the components and elements
- Developing an inspection sequence
- Preparing and organizing notes, forms, and sketches
- Arranging for traffic control
- Making arrangements for required methods of access
- Reviewing safety precautions
- Organizing tools and equipment
- Arranging for subcontract special activities
- Accounting for other special considerations

Review Bridge Structure File

The first step in preparing for a bridge inspection is to review the many available sources of information about the bridge, such as:

- "As-built" bridge plans
- Previous inspection reports
- Maintenance and repair records
- Rehabilitation/Retrofit plans
- Geotechnical data
- Hydrologic data
- Roadway plans
- Utility plans
- Right-of-way plans

Bridge Plans

The bridge plans contain information about the bridge type, the number of spans, the use of simple or continuous spans, and the materials of construction (see Figure 3.1.1). They also contain information about the presence of composite action between the deck and girders, the use of framing action at the substructure members, and the kind of connection details used. The year of construction and the design loading are also usually contained in the bridge plans.

Figure 3.1.1 Inspectors Reviewing Bridge Plans

Previous Inspection Reports

Previous inspection reports provide valuable information about the history of the bridge, documenting its condition in previous years. This information can be used to determine which components and elements of the bridge warrant special attention. It also allows the inspector to compare the current levels of deterioration with those noted during previous inspections to help determine the rate of deterioration.

Maintenance and Repair Records

Maintenance and repair records allow the inspector to report all subsequent repairs during the inspection phase, noting the types, extent, and dates of the repairs.

Rehabilitation Plans

Rehabilitation plans show modifications and replacements performed on the structure. Just as with the design plans, "As-Built (or record) drawings are preferable.

Geotechnical Data

Geotechnical data provides information about the foundation material below the structure. Sand, silt, or clay is more susceptible to settlement and scour problems than is rock. Therefore, structures founded on these materials should generally be given more attention with respect to foundation and scour issues than those founded on rock.

Hydrologic Data

Hydrologic data provides information about the shape and location of the channel, the presence of protection devices, flood frequencies, and water elevations for various flood intervals. This information is needed for scour evaluation, expected flood flows, and water velocity.

Roadway Plans

Roadway plans may not provide some information if the structure plans are not available.

Additional Data

Utility plans can be used to determine the types and numbers of utility attachments, and right-of-way plans can be used to determine the limits of the right-of-way, which can be a factor in determining access requirements.

Identify Components and Elements

Another important activity in preparing for the inspection is to establish the structure orientation, as well as a system for identifying the various components and elements of the bridge (see Figure 3.1.2). If drawings or previous inspection reports are available, the identification system used during the inspection should be the same as that used in these sources, with the exception of truss numbering as discussed below.

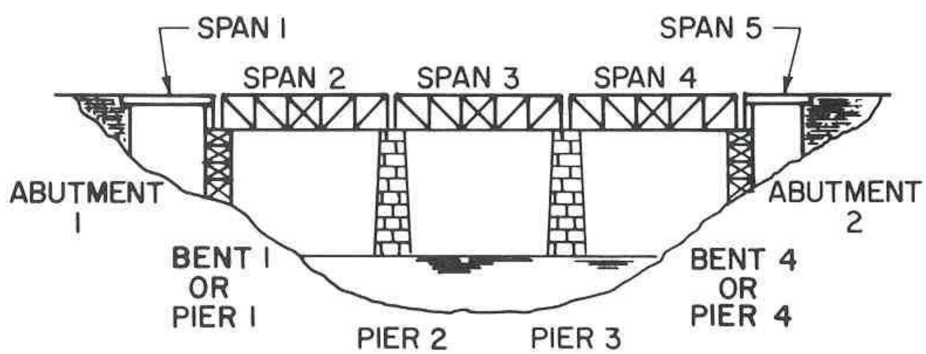


Figure 3.1.2 Sample Bridge Numbering Sequence

If no previous records are available, then the inspector should establish an identification system. The numbering system presented in this section is one possible system, but some states may use a different numbering system.

The route direction can be determined based on mile markers or stationing, and this direction should be used to identify the beginning and the end of the bridge.

Deck Element Numbering System

The deck element numbering system should include the deck sections (between construction joints), expansion joints, railing, parapets, and light standards. These elements should be numbered consecutively, from the beginning to the end of the bridge.

Superstructure Element Numbering System

The superstructure element numbering system should include the spans, the beams, and, in the case of a truss, the panel points. The spans should be numbered consecutively, with Span 1 located at the beginning of the bridge. Multiple beams should be numbered consecutively from left to right facing in the route direction. Similar to spans, floorbeams should be numbered consecutively from the beginning of the bridge, but the first floorbeam should be labeled as Floorbeam 0. This will coordinate the floorbeam and the bay numbers such that a given floorbeam number will be located at the end of its corresponding bay.

For trusses, the panel numbers should be numbered similarly to the floorbeams, beginning with Panel Point 0. Label both the upstream and downstream trusses. Points in the same vertical line have the same number. If there is no lower panel point in a particular vertical line, the numbers of the lower chord will skip a number (see Figure 3.1.3). Some design plans number to midspan on the truss and then number backwards to zero using prime numbers. However, this numbering system is not recommended for field inspection use since the prime designations in the field notes may be obscured by dirt.

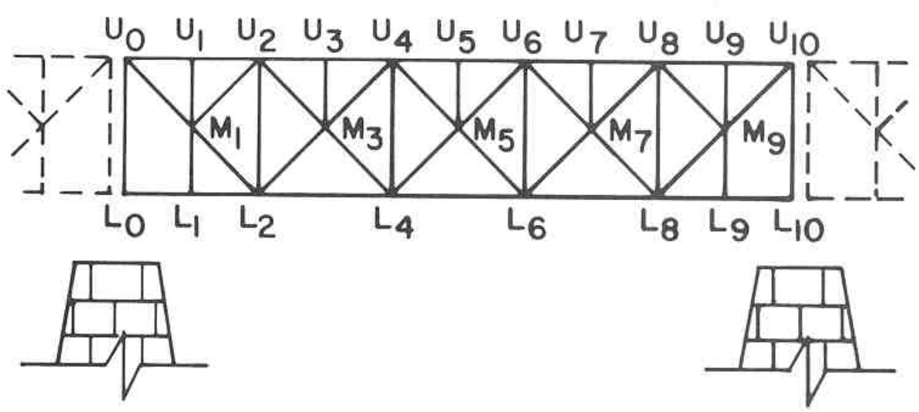


Figure 3.1.3 Sample Truss Numbering Scheme

Substructure Element Numbering System

The substructure element numbering system should include the abutments and the piers. Abutment 1 is located at the beginning of the bridge, and Abutment 2 is located at the end. The piers should be numbered consecutively, with Pier 1 located closest to the beginning of the bridge (see Figure 3.1.2). Alternatively, all the substructure units may be numbered consecutively without noting abutments or piers.

Develop Inspection Sequence

An inspection normally begins with the deck and superstructure elements and proceeds to the substructure. However, there are many factors that must be considered when planning a sequence of inspection for a bridge, including:

- Type of bridge
- Condition of the bridge components

- Overall condition
- Inspection agency requirements
- Size and complexity of the bridge
- Traffic conditions
- Special procedures

A sample inspection sequence for a bridge of average length and complexity is presented in Table 3.1.1. While developing an inspection sequence is important, it is of value only if following it ensures a complete and thorough inspection of the bridge.

1) Roadway Elements

- Approach roadways
- Traffic safety features
- General alignment
- Approach alignment
- Deflections
- Settlement

2) Deck Elements

- Bridge deck surface
- Expansion joints
- Sidewalks and railings
- Drainage
- Signing
- Electrical-lighting
- Barriers, gates, and other traffic control devices
- Bridge deck soffit

3) Superstructure Elements

- Bearings
- Main supporting members
- Secondary members and bracings
- Utilities
- Anchorages

4) Substructure Elements

- Abutments
- Skewbacks (arches)
- Slope protection
- Piers
- Footings
- Piles
- Curtain walls

5) Channel and Waterway Elements

- Channel profile and alignment
- Channel streambed
- Channel embankment
- Channel embankment protection
- Fenders
- Dolphins
- Hydraulic opening
- Water depth scales
- Navigational lights and aids

Table 3.1.1 Sample Inspection Item List

Prepare and Organize Notes

Preparing notes, forms, and sketches prior to the on-site inspection eliminates unnecessary work in the field. Copies of the agency's standard inspection form should be obtained for use in recordkeeping and as a checklist to ensure that the condition of all elements is noted.

Photocopy sketches from previous inspection reports so that defects previously documented can simply be updated. Preparing extra copies provides a contingency for sheets that may be lost or damaged in the field.

If previous sketches are not available, then pre-made, generic sketches may be used for repetitive features or members. Possible applications of this timesaving procedure include deck sections, floor systems, bracing members, abutments, piers, and retaining walls. Numbered, pre-made sketches and forms can also provide a quality control check on work completed.

Traffic Control

Bridge inspection, like construction and maintenance activities on bridges, often presents motorists with unexpected and unusual situations (see Figure 3.1.4). Most state agencies have adopted the federal *Manual on Uniform Traffic Control Devices for Streets and Highways* (*MUTCD*). Some state and local jurisdictions, however, issue their own manuals. When working in an area exposed to traffic, the bridge inspector should check and follow the governing standards. These standards will prescribe the minimum procedures for a number of typical applications and the proper use of standard traffic control devices, such as cones, signs, and flashing arrow boards.

Figure 3.1.4 Traffic Control Operation

Principles and procedures, which enhance the safety of motorists and bridge inspectors in work areas, include the following:

- Traffic safety should be a high priority element on every bridge inspection project where the inspectors' activities are exposed to traffic or likely to affect normal traffic movements.
- Traffic should be routed through work areas with geometrics and traffic control devices comparable to those employed for other highway situations.
- Traffic movement should be inhibited as little as practicable.
- Approaching motorists should be guided in a clear and positive manner throughout the bridge inspection site.
- On long duration inspections, routine inspection of traffic control devices should be performed.
- All persons responsible for the performance of traffic control operations should be adequately trained.

In addition, schedules may have to be adjusted to accommodate traffic control needs. For example, the number of lanes that can be closed at one time may require conducting the inspection operation with less than optimum efficiency. While it might be most efficient to inspect a floor system from left to right, traffic control may dictate working full length, a few beams at a time.

Special Considerations

Time Requirements

The inspection report or the bridge record file should state the amount of time required for the inspection. The inspection time requirements should be broken down into office preparation, travel time, field time, and report preparation. The overall condition of the bridge will play a major role in determining how long an inspection will take. Previous inspection reports provide an indication of the bridge's overall condition. It generally takes more time to inspect and document a deteriorated element (e.g., measuring, sketching, and photographing) than it does to simply observe and document that an element is in good condition.

Peak Travel Times

In populated areas, an inspection requiring traffic restrictions may be limited to certain hours of the day, such as 10:00 AM to 2:00 PM. Some days may be banned for inspection work altogether. Actual inspection time may be less than a 40-hour work week in these situations, and schedules should be adjusted accordingly.

Set-up Time

Set-up time must be considered both before and during the inspection. For example, rigging efforts may require several days before the inspectors arrive on the site. Also, other equipment, such as compressors and cleaning equipment, may require daily set-up time. Adequate time should be provided in the schedule for set-up and take-down time requirements. The time to install and remove traffic control devices must also be considered.

Access

Access requirements must also be considered when preparing for an inspection. Bridge members may be very similar to each other, but they may require different amounts of time to gain access to them. For example, it may take longer to maneuver a lift device to gain access to a floor system near utility lines than for one that is free of obstructions. On some structures, access hatches may need to be opened to gain access to a portion of the bridge.

Weather

Adverse weather conditions may not halt an inspection entirely, but may play a significant role in the inspection process. During adverse weather conditions, climbing should generally be avoided. There must be an increased awareness of safety hazards, and keeping notes dry can be difficult. During seasons of poor weather, a less aggressive schedule should be adopted than during the good weather months.

Safety Precautions

While completing the inspection in a timely and efficient manner, the importance of taking safety precautions cannot be overlooked. The inspection team must follow the general guidelines for safe inspections. Confined space entry procedures must be in accordance with OSHA and the owners' requirements. For climbing inspections, the three basic requirements for safe climbing must be followed. For additional information about safety precautions, refer to Topic 3.2.6.

Permits

When inspecting a bridge owned by or crossing a railroad, an access permit generally must be obtained before proceeding with the field inspection. A permit must also be obtained when inspecting bridges passing over navigable waterways.

Tools

To perform a complete and accurate inspection, the proper tools and equipment must be used. Bridge location and type are two main factors in determining required tools and equipment. Refer to Topic 3.4 for a complete list of inspection tools and equipment.

Subcontract Special Activities

Consideration must be given to time requirements when special activities must be scheduled. These activities may include one or more of the following:

- Maintenance and protection of traffic (M.P.T.)
- Access, including rigging, inspection vehicle(s), or a combination there of
- Coordination with various railroads, including obtaining the services of railroad flagmen

3.1.5 Performing the Inspection

This duty is the on-site work of accessing and examining bridge components and waterway, if present.

Inspection procedures as presented in the NBIS should always be followed.

Basic activities include:

- Visual examination of bridge components
- Physical examination of bridge components
- Evaluation of bridge components
- Examination and evaluation of the waterway beneath the structure, if any, and approach roadway geometry

General Inspection Procedures

Duties associated with the inspection include maintaining the proper structure orientation and member numbering system, developing an inspection sequence, and following proper inspection procedures.

The procedures used to inspect a bridge depend largely on the bridge type, the materials used, and the general condition of the bridge. Therefore, the inspector must be familiar with the basic inspection procedures for a wide variety of bridges. A first step in the inspection procedure is to establish the orientation of the site and of the bridge. The orientation should include the compass directions, the direction of waterway flow, and the direction of the inventory route. Also record inspection team, air temperature, weather conditions, and time.

After the site orientation has been established, the inspector is ready to begin the on-site inspection. The inspector must be careful and attentive to the work at hand, and no portion of the bridge should be overlooked. Those portions that are most critical to the structural integrity of the bridge should be given special attention. (Refer to Topic 8.1 for a description of fracture critical members in steel bridges.)

The prudence used during the inspection must be combined with thorough and complete recordkeeping. Observations should be careful and attentive, and every defect should be recorded. A very careful inspection is worth no more than the records kept during that inspection.

Numbers or letters should be crayoned or painted on the bridge to identify and code components and elements of the structure. The purpose of these marks is to keep track of the inspector's location and to guard against overlooking any portion of the structure.

The inspector should note the general approach roadway alignment, and sight along the railing and edge of the deck or girder to detect any misalignment or settlement.

Decks

The inspector should check the approach pavement for unevenness, settlement, or roughness. Also check the condition of the shoulders, slopes, drainage, and approach guardrail.

The deck and any sidewalks should be examined for various defects, noting size, type, extent, and location of each defect. The location should be referenced using the centerline or curb line, the span number, and the distance from a specific pier or joint.

Examine the expansion joints for sufficient clearance and for adequate seal. Record the width of the joint opening at both curb lines, noting the air temperature and the general weather conditions at the time of the inspection.

Finally, check that safety features, signs, and lighting are present, and note their condition.

Superstructures

The superstructure must be inspected thoroughly, since the failure of a main supporting member could result in the collapse of the bridge. The most common forms of main supporting members are:

- Beams and girders
- Floorbeams and stringers
- Trusses
- Catenary and suspender cables
- Eyebar chains
- Arch ribs
- Frames
- Pins and hanger plates

Bearings

The bearings must also be inspected thoroughly, since they provide the critical link between the superstructure and the substructure. Record the difference between the rocker tilt and a fixed reference line, noting the direction of tilt, the air temperature, and the general weather conditions at the time of the inspection.

Substructures

The substructure, which supports the superstructure, is made up of abutments, piers, and bents. If "as-built" plans are available, the dimensions of the substructure units should be compared with those presented on the plans. Since the primary method of bridge inspection is visual, all dirt, leaves, animal waste, and debris should be removed to allow close observation and evaluation. Substructure units should be checked for settlement by sighting along the superstructure and noting any tilting of vertical faces. In conjunction with the scour inspection of the waterway, the substructure units should be checked for undermining, noting both its extent and location.

Waterways

Waterways are dynamic in nature, with their volume of flow and their path continually changing. Therefore, bridges passing over them must be carefully inspected for the effects of these changes.

A record should be maintained of the channel profile and alignment, noting any meandering of the channel both upstream and downstream. Report any skew or improper location of the piers or abutments relative to the stream flow.

Scour, the erosion of a riverbed area caused by stream flow, is the primary concern when evaluating the effects of waterways on bridges (see Figure 3.1.5). The existence and extent of scour must be determined using a grid system and noting the depth of the channel bottom at each grid point.

Figure 3.1.5 Inspection for Scour and Undermining

Embankment erosion should be noted both upstream and downstream of the bridge, as should debris and excessive vegetation. Record their type, size, extent, and location. Note also the high water mark, referencing it to a fixed elevation such as the bottom of the superstructure.

Inspection of Bridge Elements

The inspector must be familiar with several general terms used to describe bridge defects:

- Corrosion – rusting
- Cracking - breaking away without separating into parts
- Splitting - separating into parts
- Connection slippage – relative movement of connected parts
- Overstress - deformation due to overload
- Collision damage - damage caused when a bridge is struck by vehicles or vessels

Refer to Section 2 for a more detailed list and description of types and causes of deterioration. As described in Section 2, each material is subject to unique defects. Therefore, the inspector should be familiar with the different inspection procedures used with each material.

Timber Inspection

When inspecting timber structures, determine the extent and severity of decay, weathering and wear, being specific about dimensions, depths, and locations. Sound and probe the timber to detect hidden deterioration due to decay, insects, or marine borers.

Note any large cracks, splits, or crushed areas. While collision or overload damage may cause these defects, the inspector should be factual, avoiding speculation as to the causes. Note any fire damage, recording the measurements of the remaining sound material. Document any exposed untreated portions of the wood, indicating the type, size, and location.

Concrete Inspection

When inspecting concrete structures, note all visible cracks, recording their type, width, length, and location. Any rust or efflorescence stains should also be recorded. Concrete scaling can occur on any exposed face of the concrete surface, and its area, location, depth, and general characteristics should be recorded. Inspect concrete surfaces for delamination or hollow zones, which are areas of incipient spalling, using a hammer or a chain drag. Delamination should be carefully documented using sketches showing the location and pertinent dimensions.

Unlike delamination, spalling is readily visible. Spalling should also be documented using sketches or photos, noting the depth of the spalling, the presence of exposed reinforcing steel, and any deterioration or section loss that may be present on the exposed bars.

Steel and Iron Inspection

When inspecting steel or iron structures, determine the extent and severity of corrosion, carefully measuring the amount of cross section remaining. All cracks should be noted, recording their length, size, and location. Bent or damaged members should be documented, noting the type of damage and amount of deflection.

Loose rivets or bolts can be detected by striking them with a hammer while holding a thumb on the opposite end of the rivet or bolt. Movement will be felt if it is loose. In addition, any missing rivets or bolts should also be noted.
Note any frozen pins, hangers, or expansion devices. One indication of this is if the hangers or expansion rockers are inclined or rotated in a direction opposite to that expected for the current temperature. In cold weather, a rocker bearing should lean towards the fixed end of the bridge, while in hot weather, it should lean away from the fixed end. A locked bearing is generally caused by heavy rust on the bearing elements.

For the inspector's evaluation to be substantiated, all inspection findings must be documented or recorded. Documentation is referred to as the "condition remarks" in the "inspection report".

3.1.6 Preparing the Report

Documentation is essential for any type of inspection. The inspector must gather enough information to ensure a comprehensive and complete report. Report preparation is a duty, which reflects the effort that the inspector puts into performing the inspection. Both must be comprehensive. The report is a record of both the bridge condition and the inspector's work.

Basic activities in preparing the inspection report include:

- Completion of agency forms
- Objective written documentation of all inspection findings
- Providing photo references and sketches
- Objective evaluation of bridge components
- Recommendations and cost estimates
- Summary

A sample bridge inspection report can be found in Appendix B of this manual.

3.1.7 Identifying Items for Repairs and Maintenance

The final basic duty is to identify items for repairs and maintenance. The inspector must identify such items to promote public safety and maximize longevity of the bridge.

Most recommendations concerning repairs will be in the category of programmed repairs (i.e., repairs that will be incorporated into preprogrammed repair and maintenance schedules). Examples of maintenance activities include: flushing the deck, flushing the scuppers and down spouting, lubricating the bearings and painting the structure.

The inspector must carefully consider the benefits to be derived from making repairs and the consequences if such repairs are not made. Also, the inspector should check the previous report recommendations to see what repairs and/or maintenance was identified and the priority of such items. If the repairs were to be completed before the next inspection, it is the responsibility of the inspector to note if the repairs have been completed and appear satisfactory.

3.1.8

Types of Bridge Inspection

The type of inspection may vary over the useful life of a bridge to reflect the intensity of inspection required at the time of inspection. The five types of inspections listed below will allow a Bridge Owner to establish appropriate inspection levels consistent with the inspection frequency and the type of structure and details.

Initial (Inventory)

An initial inspection is the first inspection of a bridge as it becomes a part of a bridge file, but the elements of an initial inspection may also apply when there has been a change in configuration of the structure (e.g., widening, lengthening, supplemental bents, etc.) or a change in bridge ownership. The initial inspection is a fully documented investigation and is accompanied by load capacity ratings. First, this inspection provides all Structure Inventory and Appraisal (SI&A) data. Second, it provides baseline structural conditions and identification of existing problems.

Routine (Periodic)

Routine inspections are regularly scheduled inspections consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge, to identify any changes from "initial" or previously recorded conditions, and to ensure that the structure continues to satisfy present service conditions. Inspection of underwater portions of the substructure is limited to observations during low-flow periods and/or probing for signs of undermining. The areas of the structure to be closely monitored are those determined by previous inspections and/or load rating calculations to be critical to load-carrying capacity.

Damage

A damage inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions. The scope of inspection should 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 effect a repair. A timely in-depth inspection may eliminate the need for this inspection.

In-Depth

An in-depth inspection is a close-up, hands-on inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, nondestructive field 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. For small bridges, the in-depth inspection should include all critical members of the structure. For large and complex structures, these inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections, or details.

Special (Interim)

A special inspection is an inspection scheduled at the discretion of the Bridge Owner. It is used to monitor a particular known or suspected deficiency, such as foundation settlement or scour, fatigue damage, or the public's use of a load posted bridge. Guidelines and procedures on what to observe and/or measure must be provided, and a timely process to interpret the field results should be in place. These inspections are not usually comprehensive enough to meet NBIS requirements for periodic inspections.

Table of Contents

Section 3
Fundamentals of Bridge Inspection

Topic 3.2 Safety Practices

3.2.1 Key Concerns for Bridge Inspection Safety

While completing the inspection in a timely and efficient manner is important, safety is also a major concern in the field. Bridge inspection is inherently dangerous and therefore requires continual watchfulness on the part of each member of the inspection team. Attitude, alertness, and common sense are three important factors in maintaining safety. To reduce the possibility of accidents, we all need to be concerned about safety.

Five key motivations for bridge inspection safety:

- Injury and pain - Accidents can cause pain, suffering, and even death. Careless inspectors can severely injure or even kill themselves or others on the inspection team. Resulting pain and discomfort can hamper the inspector for the rest of their life.

- Family hardship - A worker's family also suffers hardship when an accident occurs. Not only is there loss of income, but there is also the inability to participate in family activities, or even, in the case of major disability, placing the burden of caring for the injured person on family members.

- Equipment damage - The repair or replacement of damaged equipment can be very costly. Not only is there the cost of fixing the damaged equipment, but there is also a cost due to the loss of time while the equipment is not available for use.

- Lost production - The employer not only loses revenues associated with the employee's work, but also loses time and money spent on safety training and equipment. Additional inspectors must be trained to replace the injured worker.

- Medical expenses - Whether coverage is an employee benefit, personal insurance, or out of pocket, someone has to pay for medical expenses. Ultimately, the tax-paying public pays the bill for accidents through higher insurance premiums.

Inspectors should constantly be aware of safety concerns. Spending the effort to be safe pays big dividends in avoided expenses and grief.

3.2.2 Safety Responsibilities

The employer is responsible for providing a safe working environment, including:

- Clear safety regulations and guidelines
- Safety training
- Proper tools and equipment

The supervisor is responsible for maintaining a safe working environment, including:

- Supervision of established job procedures
- Guidance in application of safety procedures
- Guidance in proper use of equipment
- Enforcement of safety regulations

Bridge inspectors are ultimately responsible for their own safety. The bridge inspector's responsibilities include:

- Recognition of physical limitations – Only you know what you are capable of doing. If you are uncomfortable doing something, let it be known.
- Knowledge of rules and requirements of job - If you do not understand something or do not feel qualified to perform a particular task safely, it is your responsibility to stop and ask questions. If a procedure appears to be unsafe, question it and constructively try to develop a better way.
- Safety of fellow workers - Do not endanger coworkers by your actions. Warn them if you see them doing something unsafe.
- Reporting an accident - If there is an accident, it is essential to report it to a designated individual in your agency or company within the prescribed time frame, usually within 24 hours. Any injury must be promptly reported in order to assure coverage, if necessary, under workmen's compensation or other insurance.

3.2.3 Personal Protection

Proper Inspection Attire

It is important to dress properly for the job. Field clothes should be properly sized for the individual, and they should be appropriate for the climate. For general inspection activities, the inspector should wear leather boots with traction lug soles. For climbing of bridge components, the inspector should wear boots with a steel shank (with non-slip soles without heavy lugs), as well as leather gloves. Wearing a tool pouch enables the inspector to carry tools and notes with hands free for climbing and other inspection activities.

Inspection Safety Equipment

Safety equipment is designed to prevent injury. However, the inspector must use the equipment in order for it to provide protection. The following are some common pieces of safety equipment:

Hard Hat

Wearing a hard hat can prevent serious head injuries in two ways. First, it provides protection against falling objects. The bridge site environment during inspection activities is prone to falling objects. Main concerns are:

- Deteriorated portions of bridge components dislodged during inspection
- Equipment dropped by coworkers overhead
- Debris discarded by passing motorists

Secondly, a hard hat protects the inspector's head from accidental impact with bridge components. When inspections involve climbing or access equipment, the inspector is frequently dodging various configurations of superstructure elements. These superstructure elements can be sharp edged and are always unyielding. If the inspector makes a mistake in judgement during a maneuver and impacts the structure, a hard hat may prevent serious injury.

During the inspection, the inspector never knows when protection will be needed. Therefore, a hard hat should be worn at all times. Also, if the inspector will be free climbing, it is a good practice to wear a chinstrap with the hard hat.

Figure 3.2.1 Inspector Wearing A Hard Hat

Reflective Safety Vest

When performing activities near traffic, the inspector is required to wear a safety vest. The vest should be bright orange with reflective strips. The combination of bright color and reflectivity makes the inspector more visible to passing motorists. When the motorist is aware of the inspector's presence, safety is improved.

Figure 3.2.2 Inspector Wearing A Reflective Safety Vest

Safety Goggles

Eye protection is necessary when the inspector is exposed to flying particles. Glasses with shatterproof lenses are not adequate if side protection is not provided. It is also important to note that only single lens glasses should be worn when climbing (no bifocals).

Eye protection should be worn during activities such as:

- Using a hammer
- Using a scraper or wire brush
- Grinding
- Shot or sand blasting
- Cutting
- Welding

During welding activities, protection with appropriate lenses specifically designed for welding should be used.

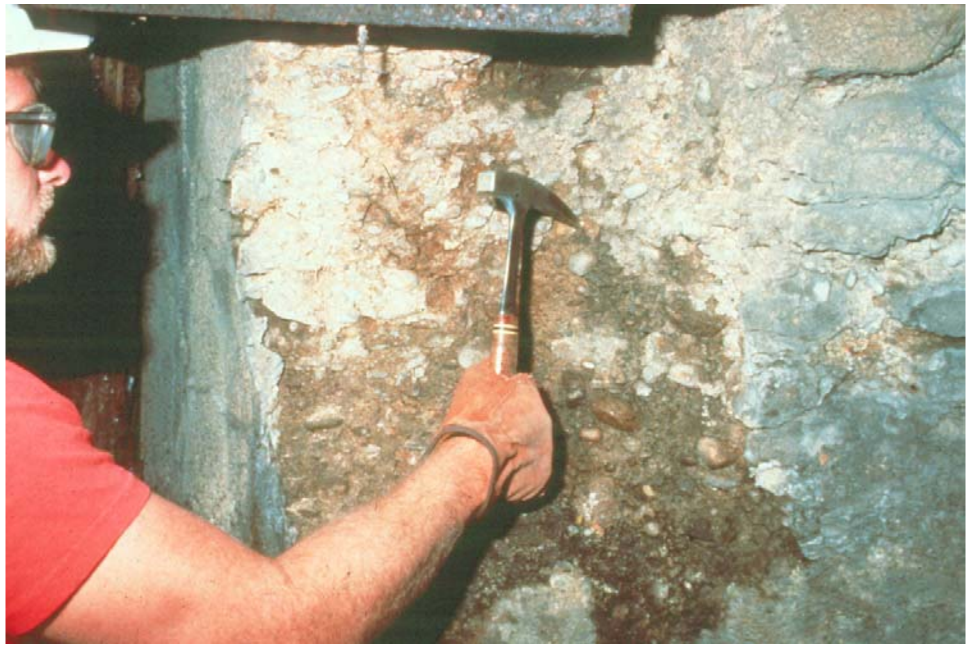


Figure 3.2.3 Inspector Wearing Safety Goggles

Life Jacket

A life jacket should always be worn when working over water or in a boat. If an accident occurs, good swimmers may drown if burdened with inspection equipment. Also, if knocked unconscious or injured due to a fall, a life jacket will keep the inspector afloat. A life jacket should also be worn when wearing hip or chest waders. If an inspector should slip or step in an area that is too deep, their waders can fill with water and drag them under, making swimming impossible.

Figure 3.2.4 Inspector Wearing a Life Jacket

Dust Mask / Respirator

A respirator or dust mask can protect the inspector from harmful airborne contaminants and pollutants. Agency or OSHA regulations should be consulted for approved types and appropriate usage.

Conditions requiring a respirator include:

- Sand blasting
- Painting
- Exposure to dust from pigeon droppings (exposure to pigeon droppings may result in histoplasonosis, a potentially very serious illness)
- Work in closed or constricted areas

Figure 3.2.5 Inspector Wearing a Respirator

Safety Harness and Lanyard

The safety harness and lanyard are the inspector's lifeline in the event of a fall. Use this equipment as required by conditions. Make sure you satisfy agency and OSHA requirements.

For example, one agency requires that a safety belt or harness be worn in the following situations:

- At heights over 6.0 m (20 feet)
- Above water
- Above traffic

Figure 3.2.6 Safety Harness with a Lanyard

To reduce the possibility of injury, the maximum lanyard length limits a fall to 1.8 m (6 feet) per OSHA regulations.

Further protection can be achieved using a shock absorber between the lanyard and the harness. The shock absorber reduces g-forces through the controlled extension of nylon webbing, which is pre-folded and sewn together.

The safety harness should be tied off to a solid structural member or to a safety line rigged for this purpose.

Do not tie off to scaffolding or its supporting cable. One of the reasons for tying off is to limit your fall in case the rigging or scaffold fails.

When working from a snooper or bucket truck, tie off to the structure if possible. Extreme caution must be exercised not to allow the equipment to be moved out from under someone tied to the bridge. If the machine is being moved frequently, it is best to tie off to the bucket or boom.

Gloves

Although one may not immediately think of gloves as a piece of safety equipment, they can prove to be an important safety feature. Wearing gloves will protect the inspector's hands from harmful effects of deteriorated members. In many inspections, structural members have been deteriorated to the point where the edges of the members have become razor sharp. These edges can cause severe cuts and lacerations to the inspector's hands that may become infected.

Boats

When performing an inspection over water, it is required to have a manned boat in the water at all times. In the event of an accident in which someone were to fall into the water, the boat can rescue them quickly. This is especially important if the individual has been rendered unconscious. In addition, it can also be used to retrieve any equipment that may have been accidentally dropped by an inspector.

3.2.4 Causes of Accidents

General

Accidents are usually caused by human error or equipment failure, but almost all accidents are due to human failings. People are not machines. We all make mistakes. Part of safety awareness is acknowledging this and planning ahead to minimize the effects of those mistakes.

Accidents caused by equipment failure can often be traced to inadequate or improper maintenance. Inspection, maintenance, and update of equipment can minimize failures.

Specific Causes

Specific causes of accidents include the following:

- Improper attitude - distraction, carelessness, worries over personal matters.
- Personal limitations - lack of knowledge or skill, exceeding physical capabilities.
- Physical impairment - previous injury, illness, side effect of medication, alcohol or drugs.
- Boredom - falling into an inattentive state while performing repetitive, routine tasks.
- Thoughtlessness - lack of safety awareness and not recognizing hazards.
- Shortcuts - sacrificing safety for time.
- Faulty equipment - damaged ladder rungs, worn rope, or frayed cables
- Inappropriate or loose fitting clothing.

3.2.5 Safety Precautions

General Precautions

Some general guidelines for safe inspections are as follows:

- Keeping well rested and alert - Working conditions encountered during an inspection are varied and can change rapidly requiring the inspector be fit and attentive.

- Maintaining proper mental and physical condition - Inspection tasks require a multitude of motor skills. To perform at acceptable levels, the inspector must be physically fit and free from mental distractions.

- Using proper tools - Do not try to use tools and equipment not suited for the job.

- Keeping work areas neat and uncluttered – Tools and equipment scattered carelessly about the work area present hazards that can result in injury.

- Establishing systematic procedures - Establish procedures early in the job utilizing them so everyone knows what to expect of one another.

- Follow safety rules and regulations - Adhere to the safety rules and regulations established by the Occupational Safety and Health Administration (OSHA), the agency, and your employer.

- Use common sense and good judgment - Do not engage in horseplay, and do not take short cuts or foolish chances.

- Avoid use of intoxicants or drugs - Intoxicants impair judgement, reflexes, and coordination.

- Medication - Prescription and over-the-counter medications can cause drowsiness or other unwanted and potentially dangerous side effects.

- Electricity - This is a potential killer. All cables and wires should be assumed to be hot (live), even if they appear to be telephone cables. The conditions encountered on many bridges are conducive to electric shock. These conditions include steel members, humidity, perspiration, and damp clothing. Transmission lines on a structure should be identified prior to the inspection. All power lines should be shut down. In rural areas, electric fences can be a hazard and should be avoided. Be aware that fiberglass posts eliminate the need for the distinctive porcelain insulation, which once identified electric fences.

- Assistance - Always work in pairs. An inspector should not take any action without someone else there to help in case of an accident. Always make sure someone else knows where you are. If someone seems to be missing, locate that person immediately.

- Inspection over water - A safety boat must be provided when working over bodies of water. It should be equipped with a life ring and have radio communication with the inspection crew.

- Waders - Caution should be used when wearing waders. If the inspector falls into a scour hole, the waders can fill with water, making swimming impossible.

- Inspection over traffic - It is best to avoid working above traffic. If it cannot be avoided, equipment, such as tools and notebooks, should be tied off.

- Entering dark areas - Use a flashlight to illuminate dark areas prior to entering as a precaution against falls, snakebites, and stinging insects.

Climbing Safety

There are three primary areas of preparation necessary for a safe climbing inspection:

1. Organization

2. Inspection Access Equipment

3. Mental attitude

Figure 3.2.7 Inspection Involving Extensive Climbing

Organization

Organization of the Inspection - A good inspection procedure incorporates a climbing strategy that minimizes climbing time. For example, beginning the day with an inspection of a truss span from one bent and finishing at the next bent by lunch time eliminates unproductive climbing across the span.

The inspection procedure should have an inspection plan so the inspector knows where to go, what to do, and what tools are needed to perform the inspection. An organized inspection reduces the chance of the inspector falling or getting stuck in a position in which he is unable to get down.

Weather conditions are a primary consideration when organizing a climbing inspection. Moderate temperatures and a sunny day are desirable.

Rain conditions warrant postponement of steel bridge inspections, as wet steel is extremely slippery.

After a rainy day, the inspector must be sure that boots are free of mud, and he must use extreme caution in areas where debris accumulation may cause a slippery surface.

Traffic should not be obstructed during bad weather.

Figure 3.2.8 Inclement Weather Causing Slippery Bridge Members

Inspection Equipment

The inspection team should be well equipped.

Personal attire should be checked for suitability to the job:

- Clothing - proper for climbing activities and temperature.
- Jewelry - rings, bracelets, and necklaces should never be worn; in an accident, jewelry can become snagged and cause additional injury.
- Eyeglasses - only single lens glasses should be worn; bifocals should not be worn because split vision impairs the inspector's ability to climb safely.

Inspection equipment should be checked for proper use and condition.

Ladders

Accidents involving ladders are the most common type of inspection-related accident.

In order to use a ladder properly, these things are needed:

- Proper length for the job.
- 3:1 tilt with blocked and secured bottom.
- An assistant for ladders over 7.6 m (25 feet), and making sure the top is tied off.
- Inspecting the ladder, prior to use, for cracked or defective rungs and rails.
- Correct climbing technique using both hands, facing the ladder, and keeping your belt buckle over the rungs.
- Using a hand line to lift equipment or tools.

Figure 3.2.9 Proper Use of Ladder

Scaffolding

Scaffolding should be checked for the height and load capacity necessary to support the inspection team.

Load tests can be performed on the ground with planned equipment and personnel. A daily inspection for cracks, loose connections, and weak areas should be performed prior to use.

Timber Planks

Single planks should never be used. Two or more planks securely cleated together should be used. Plank ends should be securely attached to their supports. All planks should be inspected for knots, splits, cracks, and deterioration prior to use.

Inspection Vehicles

Use of platform trucks, bucket trucks, and underbridge inspection vehicles may be necessary to access all elements during an inspection. Confirm that they are in safe operating condition. Such equipment must only be used when placed on a firm surface at a slope not exceeding the rated capacity of the equipment.

Figure 3.2.10 Bucket Truck

Catwalks and Travelers

Permanent inspection access devices are ideal. However, the inspector should be on guard for deterioration of elements, such as flooring, hand-hold rods, and cables.

Figure 3.2.11 Inspection Catwalk

Rigging

The inspector should be familiar with proper rigging techniques. Support cables should be at least 13 mm (1/2 inch) in diameter. The working platform or "stage" should be at least 510 mm (20 inches) wide. A line or tie-off cable separate from the primary rigging should be used.

Use common sense with regard to rigging. Do not trust your life blindly to the riggers. If you feel a procedure is unsafe or doubtful, question it and get it changed if necessary. Do not rely on ropes or planks left on the bridge by prior work. They may be rotted or not properly attached.

Figure 3.2.12 Inspection Rigging

Mental Attitude

The inspector must be mentally prepared to do a climbing inspection. A good safety attitude is of foremost importance. Three precautions that must be addressed are:

- Avoid emotional distress - Do not climb when emotionally upset. The inspector who climbs must have complete control; otherwise the chances of falling increase.

- Know where you are - Always be aware of where you are and what you are doing when climbing. Do not become so engrossed in the job that you step into mid-air.

- Do not do anything you are not confident of doing safely. If there is a feature you cannot safely inspect with the equipment available, do not do it. Highlight this fact in the notes so that appropriate equipment can be scheduled if necessary. Do not hide the fact that something was not inspected.

Confined Spaces

Safety Concerns

Inspection of box girder bridges, steel box pier caps, steel arch rings, arch ties, cellular concrete structures, and long culverts often includes confined spaces. Confined space entry is regulated by OSHA and requires proper training, equipment, and permitting.

There are four major concerns when inspecting a confined space:

- Lack of oxygen - oxygen content must remain above 19% for the inspector to remain conscious
- Toxic gases - generally produced by work processes such as painting, burning, and welding or by operation of internal combustion engines
- Explosive gases - natural gas, methane, or gasoline vapors may be present naturally or due to leaks
- Lack of light – many confined spaces are totally dark (inspector cannot see any potential hazards such as depressions, drop-offs, or dangerous animals)

Safety Procedures

When a confined area must be inspected, the safety procedures prescribed by OSHA and any additional agency requirements must be followed.

The following is a general description of the basic requirements. Refer to OSHA for specifics.

Pre-entry air tests:

- Test for oxygen with an approved oxygen testing device
- Test for other gases, such as carbon monoxide, hydrogen sulfide, methane, natural gas, and combustible vapors

Mechanical ventilation:

- Pre-entry - Oxygen and gas levels must be acceptable for a minimum prescribed time prior to entry.
- During occupancy - Ventilation should be continuous regardless of activities. Test for oxygen and other gases at prescribed intervals during occupancy.

Basic safety procedures:

- Avoid use of flammable liquids in the confined area.
- Position inspection vehicles away from the area entrance to avoid carbon monoxide fumes.
- Position gasoline powered generators "down-wind" of operations.
- Operations producing toxic gases should be performed "down-wind" of the operator and the inspection team.
- Carry approved rescue air-breathing apparatus.
- Use adequate lighting with an appropriate backup system and lifelines when entering dark areas, such as box girders and culverts.
- Inspection should be performed in pairs, with a third inspector remaining outside of dark or confined areas.

Culverts

There are several hazards that can be encountered when performing a culvert inspection. Being aware of these situations and exercising proper precautions will protect the inspector from these dangerous and potentially life threatening hazards. The following are some of the hazardous conditions an inspector may encounter.

- Inadequate Ventilation
- Drowning
- Toxic Chemicals
- Animals
- Quick Conditions
- Insufficient Number of Inspectors

Inadequate Ventilation

Culverts with inadequate ventilation can develop low oxygen levels or high concentrations of toxic and/or explosive gases. This is a big concern when one culvert end may be blocked or inspection is being performed on a long culvert.

If air quality is suspect, tests should be made to determine the concentration of gases. Testing devices may be as simple as badges worn by inspectors that change colors when in the presence of a particular gas. Devices may also be sophisticated instruments that measure the concentration of several gases.

Confined space entry requirements should be observed when inspecting a long culvert or any culvert with restricted ventilation.

Drowning

Extensive streambed erosion may result in scour holes. During periods of low flow the depth of water in these holes may be significantly greater than the remainder of the streambed. This could give the inspector the impression that wading is safe. It is advisable that the inspector use a probing rod to check water depth wherever he/she plans to walk.

Storms may generate high flows in culverts very quickly. This creates a dangerous situation for the inspectors. It is not uncommon for culverts to carry peak flow long before a storm reaches the culvert site. Inspectors should be cautious whenever storms appear imminent.

Toxic Chemicals

Occasionally, stream flow may contain hazardous chemicals from any of a number of sources. Fires, explosions, and serious illness could result from the presence of such chemicals if appropriate precautions are not taken.

Animals

An accumulation of dirt or debris may provide a home for snakes, rodents, or other animals. These could provide a problem to the inspector. An inspector's ability to react to these hazards may be compromised by poor lighting and inadequate room to move. Also, dead animals may be present.

Quick Conditions

Quicksand conditions can occur in sandy streambeds, especially at the outlet end of the culvert. Inspectors should be aware of this and should proceed with caution in geographical areas known to have these problems.

This page intentionally left blank

Table of Contents

Section 3
Fundamentals of Bridge Inspection

This page intentionally left blank

Topic 3.3 Traffic Control

3.3.1 Introduction

This topic describes the traffic control procedures required for a relatively short term closure (only a day or two). Long term or more permanent construction closures using concrete barriers are not included in this topic.

Figure 3.3.1 Traffic Control Operation

Bridge inspection, like construction and maintenance activities on bridges, often presents motorists with unexpected and unusual situations. Most state agencies have adopted the federal *Manual on Uniform Traffic Control Devices for Streets and Highways* (*MUTCD*). Some states and local jurisdictions, however, issue their own manuals.

When working in an area exposed to traffic, the bridge inspector should check and follow the existing standards. These standards will prescribe the minimum procedures for a number of typical applications and the proper use of standard traffic control devices such as cones, signs, and flashing arrow-boards.

3.3.2 Philosophy and Fundamental Principles

All traffic control devices used on street and highway construction or maintenance work should conform to the applicable specifications of the *MUTCD* and the agency.

Principles and procedures which have been shown to enhance the safety of motorists, pedestrians, and workers in the vicinity of work areas include the following:

Inform the Motorists

Traffic safety in work zones should be an integral and high priority element of every inspection project, from the planning stage to performance of the inspection.

The safety of the motorist, pedestrian, and worker must be kept in mind at all times.

The basic safety principles governing the design of permanent traffic control for roadways and roadsides should also govern the design of inspection sites. The goal should be to route traffic through such areas with geometrics and traffic control devices comparable to those for normal highway situations.

The situation and expected actions must be clearly communicated to the driver.

A traffic control plan, in detail appropriate to the complexity of the work project, should be prepared and understood by all responsible parties before the site is occupied. Any changes in the traffic control plan should be approved by an official trained in safe traffic control practices.

Control The Motorists

Traffic movement should be inhibited as little as practical.

Traffic control in work sites should be designed on the assumption that motorists will only reduce their speeds if they clearly perceive a need to do so. Reduced speed zoning should be avoided as much as practical.

The objective is a traffic control plan that uses a variety of traffic control measures and devices in whatever combination necessary to assure smooth, safe vehicular movement past the work area and at the same time provide safety for the equipment and the workers on the job.

Frequent and abrupt changes in geometrics, such as lane narrowing, dropped lanes, or main roadway transitions that require rapid maneuvers, should be avoided.

Provisions should be made for the safe operation of work vehicles, particularly on high speed, high volume roadways. This includes the use of roof mounted flashing lights or flashers when entering or leaving the work zone. This also includes considering the number of lanes that can be closed at one time for an operation. While it might be most cost efficient to inspect a floor system from left to right, traffic control may dictate working full length, a few beams at a time.

Inspection time should be minimized to reduce exposure to potential hazards without compromising the thoroughness of the inspection.

Provide a Clearly Marked Path

Motorists should be guided in a clear and positive manner while approaching and traversing work areas.

Adequate warning, delineation, and channelization should be provided to assure the motorist positive guidance in advance of and through the work area. Proper signing and other devices which are effective under varying conditions of light and weather must be used.

All traffic control devices should be removed immediately when no longer needed.

The maintenance of roadside safety requires constant attention during the life of the work because of the potential increase in hazards.

To accommodate run-off-the-road incidents, disabled vehicles or other emergency situations, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical.

Channelization of traffic should be accomplished by the use of signing, flexible posts, barricades, and other lightweight devices which will yield when hit by errant vehicles.

Whenever practical, equipment and materials should be stored in such a manner as not to be vulnerable to run-off-the-road vehicle impact. When safe storage is not available, adequate attenuation devices should be provided.

The goal of a good traffic control plan is the safe and efficient movement of motorists and pedestrians and the protection of bridge inspectors at work areas.

3.3.3 Inspector Safety Practices

Work Zone

Traffic represents as great, or even greater, threat to the inspector's safety than climbing high bridges. The work zone is intended to be a safe haven from traffic so the inspectors can concentrate on doing their jobs.

As such, the work zone needs to be clearly marked so as to guide the motorist around it and, insofar as possible, prevent errant vehicles from entering. The work zone should be as compact as possible to minimize traffic disruption, but must be wide enough and long enough to permit access to the area to be inspected and allow for safe movement of workers and equipment. The end of the work zone should be clearly signed as a courtesy to the motorist.

Figure 3.3.2 Work Zone

Vehicles and Equipment

Inspection vehicles and equipment need to be made visible to the motorists with flashing marker lights or arrow boards as appropriate.

Vehicles entering and exiting the work zone should use a roof mounted flashing light or flashers to distinguish themselves from other motorists. Also, all vehicles should use extreme caution when moving in and out of the work zone. Allow traffic ample time to react to the vehicle's movements.

Figure 3.3.3 Inspection Vehicles with Flashing Light

Workers

Individuals in a work zone must wear approved safety vests and hard hats for visibility and identification. They also help make the inspector look “official” to the public. The inspectors should also stay within the work zone for their own safety.

Figure 3.3.4 Inspector with a Safety Vest and Hard Hat

3.3.4 Requirements of Traffic Control Devices

Each job is different and has traffic concerns that are unique to that location. Selection of the proper traffic control device's for each location is dependent upon many factors. Though there are several different types of traffic control devises, there are some basic requirements for efficient traffic control devices:

1. They must be visible and attention getting.

   - Bright colors make devices easier to see.

   - All signs must be legible and color distinguishable at night as well as during the day. Nighttime sign visibility is provided through retroreflectivity, which is accomplished by spherical glass beads or prismatic reflectors in the sign material. The headlights reflect off the sign and back to the driver, making the sign visible at night.

   - New sign messages such as "Slow Down. My Daddy Works Here" and "Give Us A Brake. Slow Down" cause the driver to think on a more personal level.

2. They must give clear direction.

3. They must command respect. They should be official (*MUTCD*).

4. They must elicit the proper response at the proper time.

   - The decision process includes the classical chain of sensing, perceiving, analyzing, deciding, and responding.

- The average perception-reaction time of a driver is 2.5 seconds. At 60 mph, that 2.5 seconds translates to 220 feet (67 m).

- Traffic control must accommodate a wide range of vehicles (from small compact cars to large combination tractor-trailers) and driver skills, which may be impaired by alcohol, drugs, or drowsiness.

All of these requirements for traffic control devices have been factored into the various agencies' guidelines for work area traffic control. These guidelines represent efforts by trained people. Do not make up your own traffic patterns.

3.3.5 Types of Traffic Control Devices

Signs

Examples of traffic control signs include the following:

- Regulatory - "Speed Limit 40 mph", "DO NOT PASS", may require special authority
- Warning - "Bridge Inspection", "Work Area Ahead", "Slow"
- Guide Signs - Directional and destination signs; not used for bridge inspection traffic control unless a detour is established
- Changeable Message Signs – Can display more than one message

Figure 3.3.5 Work Area Speed Limit Sign (Regulatory)

Figure 3.3.6 Traffic Control Sign (Warning)

Channelizing Devices

The functions of channelizing devices are to warn and alert drivers of hazards created by construction or maintenance activities in or near the traveled way and to guide and direct drivers safely past the hazards.

Devices used for channelization should provide a smooth and gradual transition in moving traffic from one lane to another, onto a bypass or detour, or in reducing the width of the traveled way. If possible, they should be constructed so as not to inflict any undue damage to a vehicle that inadvertently strikes them.

Channelizing devices are elements in a total system of traffic control devices for use in highway construction and maintenance operations. These elements should be preceded by a subsystem of warning devices that are adequate in size, number, and placement for the type of highway on which the work is to take place.

Typical channelizing devices include the following:

- Cones
- Drums
- Wands
- Vertical panels
- Portable concrete barrier sections (these are seldom applicable to bridge inspection due to the short duration of the work)

Figure 3.3.7 Traffic Control Cones

Figure 3.3.8 Vertical Panels – Note panels attached to drums

Lighting

Another type of control device is lighting. Examples of lighting include the following:

- Flashers - attached to signs or other devices to attract attention or for night visibility

- Arrowboards - for lane control

- Floodlights - to illuminate the work area at night and/or to assist motorists in negotiating a restricted area; should only be required for bridge inspection in emergencies or in extremely high traffic volume areas where lane restrictions are only feasible at night; aim floodlights so driver's vision is not impaired.

Figure 3.3.10 Arrowboard

Flaggers

A number of hand signalizing devices, such as STOP/SLOW paddles, lights, and red flags, are used to control traffic through work zones. The sign paddle bearing the clear messages "STOP" or "SLOW" provides motorists with more positive guidance than flags and is generally the primary hand signaling device. Flag use should be limited to emergency situations and at spot locations that can best be controlled by a single flagger, if permitted by the agency.

Since flaggers are responsible for human safety and make the greatest number of public contacts of all construction personnel, it is important that qualified personnel be selected. A flagger should possess the following minimum qualifications:

- Good common sense
- Good physical condition, including sight and hearing
- Mental alertness
- Courteous but firm manner
- Neat appearance
- Sense of responsibility for safety of public and crew
- Training in safe traffic control practices

The use of hard hat and orange clothing, such as a vest, shirt, or jacket, should be required for flaggers. For nighttime conditions, similar outside garments should be reflectorized.

Flaggers are provided at work sites to stop traffic intermittently as necessitated by work progress. They also maintain continuous traffic past a work site at reduced speeds to help protect the work crew. For both of these functions, the flagger must, at all times, be clearly visible to approaching traffic for a distance sufficient to permit proper response by the motorist to the flagging instructions and to permit traffic to reduce speed before entering the work site (generally several hundred feet, depending on site conditions). In positioning flaggers, consideration must be given to maintaining color contrast between the work area background and the flagger's protective garments.

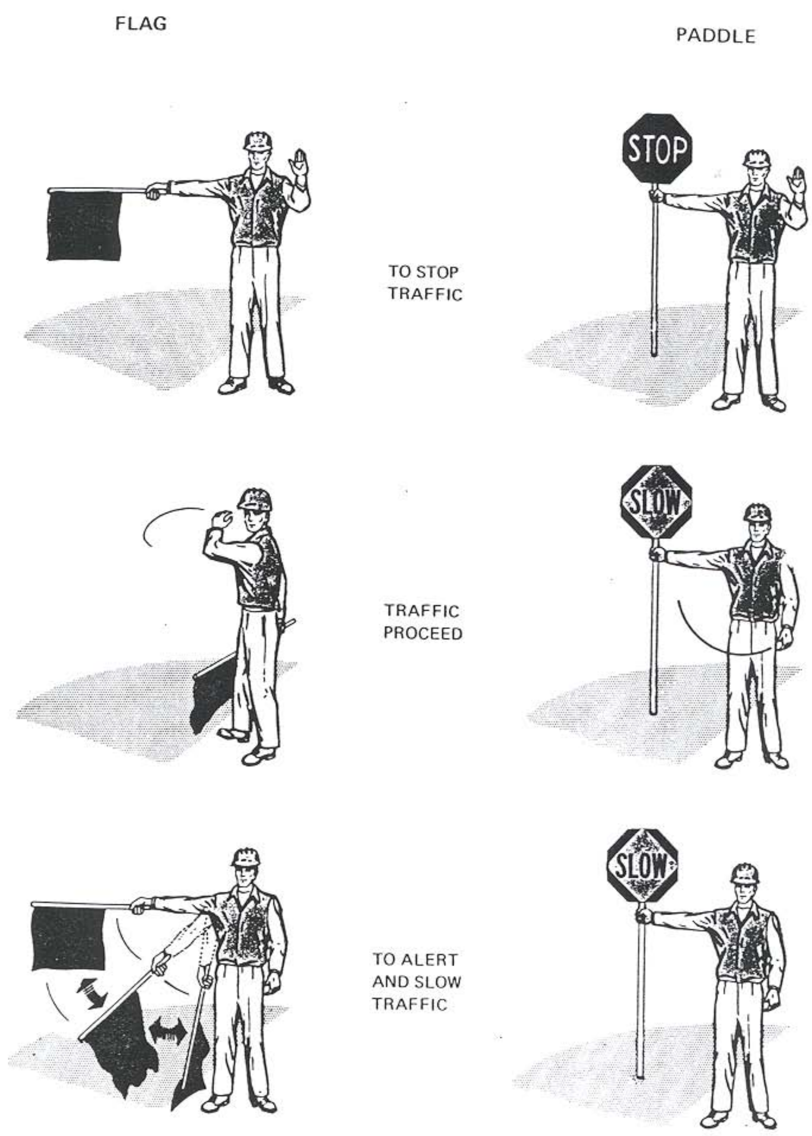


Figure 3.3.11 Use of Hand Signaling Devices by Flagger (from *MUTCD*)

The following methods of signaling with sign paddles should be used:

- To stop traffic - The flagger should face traffic and extend the STOP sign paddle in a stationary position with the arm extended horizontally away from the body. The free arm is raised with the palm toward approaching traffic.

- When it is safe for traffic to proceed - The flagger should face traffic with the SLOW sign paddle held in a stationary position with the arm extended horizontally away from the body. The flagger motions traffic ahead with the free hand.

- When it is desired to alert or slow traffic - The flagger shall face traffic with the SLOW sign paddle held in a stationary position with the arm extended horizontally away from the body.

The following methods of signaling with a flag should be used:

- To stop traffic - The flagger should face traffic and extend the flag horizontally across the traffic lane in a stationary position so that the full area of the flag is visible hanging below the staff. For greater emphasis, the free arm may be raised with the palm toward approaching traffic.

- When it is safe for traffic to proceed - The flagger should stand parallel to the traffic movement and, with flag and arm lowered from view of the driver, motion traffic ahead with the free arm. Flags should not be used to signal traffic to proceed.

- Where it is desired to alert or slow traffic - The flagger should face traffic and slowly wave the flag in a sweeping motion of the extended arm from the shoulder level to straight down without raising the arm above a horizontal position.

Lights approved by the appropriate highway authority or reflectorized sign paddles or reflectorized flags should be used to flag traffic at night.

Whenever practicable, the flagger should advise the motorist of the reason for the delay and the approximate period that traffic will be halted. Flaggers and operators of machinery or trucks should be made to understand that every reasonable effort must be made to allow the driving public the right-of-way and prevent excessive delays.

Flagger stations should be located far enough in advance of the work site so that approaching traffic will have sufficient distance to reduce speed before entering the project. This distance is related to the approach speed and physical conditions at the site; however, 200 to 300 feet (60 to 90 m) is desirable. In urban areas, where speeds are low and streets are closely spaced, the distance necessarily must be decreased.

The flaggers should stand either on the shoulder adjacent to the traffic being controlled or in the barricaded lane. At a spot obstruction, a position may have to be taken on the shoulder opposite the barricaded section to operate effectively.

Under no circumstances should a flagger stand in the lane being used by moving traffic. The flagger must be clearly visible to approaching traffic at all times. For this reason, the flagger must stand alone, never permitting a group of workers to congregate around the flagger station. The flagger should be stationed sufficiently in advance of the work force to warn them of approaching danger, such as out-of-control vehicles.

Figure 3.3.12 Flagger with Stop/Slow Paddle

Flagger stations should be adequately protected and preceded by proper advance warning signs. At night, flagger stations should be adequately illuminated.

At short lane closures where adequate sight distance is available for the safe handling of traffic, the use of one flagger may be sufficient.

One-way Traffic Control

Where traffic in both directions must, for a limited distance, use a single lane, provisions should be made for alternate one-way movement to pass traffic through the constricted section. At a spot obstruction, such as a short bridge, the movement may be self-regulating. However, where the one-lane section is of any length, there should be some means of coordinating movements at each end so that vehicles are not simultaneously moving in opposite directions in the section and so that delays are not excessive at either end. Control points at each end of the route should be chosen so as to permit easy passing of opposing lines of vehicles.

Alternate one-way traffic control may be facilitated by the following means:

- Flagger control
- Flag-carrying or official car
- Pilot car
- Traffic signals

Flagger control is usually used for bridge inspection, where the one-lane section is short enough so that each end is visible from the other end. Traffic may be controlled by means of a flagger at each end of the section. One of the two should be designated as the chief flagger to coordinate movement. They should be able to communicate with each other verbally or by means of signals. These signals should not be such as to be mistaken for flagging signals.

Where the end of a one-way section is not visible from the other end, the flaggers may maintain contact by means of radio or field telephones. So that a flagger may know when to allow traffic to proceed into the section, the last vehicle from the opposite direction can be identified by description or license.

Shadow Vehicles

Shadow Vehicles with truck Mounted attenuators (TMAs) are used to prevent vehicles from entering the work zone if the operator ignores the lane closure signs and channelization. Each agency has its own specific requirements, but a shadow vehicle should generally be employed any time a shoulder or travel lane will be occupied by workers or equipment.

- The requirements for the truck itself vary, but high visibility with flashing lights, a striped panel, or an arrow board on the rear of a vehicle of a specified minimum weight are generally required.

- Some jurisdictions use truck or trailer mounted attenuators. This protects the motorist, as well as the inspectors.

Figure 3.3.13 Shadow Vehicle with Attenuator

Police Assistance

On some inspection jobs, police assistance may be helpful and even required. The presence of a patrol car aids in slowing and controlling the public. At a signalized intersection near a job site, a police officer may be required to ensure traffic flows properly and smoothly.

3.3.6 Public Safety

Since the fundamental goal of bridge inspection is to enhance public safety, it would make little sense to endanger that same public by inadequate traffic control measures. Traffic control does take time, money, and effort. It is, however, a necessary part of the business of bridge inspection.

In the broadest sense, the motorist is the customer of everyone in the transportation industry. Like everyone else, bridge inspectors need to treat customers well, inconveniencing them as little as possible and protecting their safety. This means providing well thought out, clear, and effective traffic control measures.

Pedestrians also must be considered. If a walkway must be closed, it should be properly signed and barricaded. An alternate route for the walker should be indicated, if necessary through or preferably around the work zone.

Training

Each person whose actions affect maintenance and construction zone safety (from the upper-level management personnel to construction and maintenance field personnel) should receive training appropriate to the job decisions each individual is required to make. Only those individuals who are qualified by means of adequate training in safe traffic control practices and have a basic understanding of the principles established by applicable guidelines and regulations should supervise the selection, placement, and maintenance of traffic control devices in bridge safety inspection, maintenance, and construction areas.

Responsibility

Legally and morally, it is the inspector’s responsibility to follow the regulations and guidelines of the agency having jurisdiction.

The primary goal of good traffic control is safety – safety of the workers and safety of the motorists. A secondary goal is to be able to defend yourself and your employer should there be an accident. Accidents bring lawsuits. Lawsuits bring inquiries about who did what. One thing investigated during a lawsuit will be whether or not the standards and regulations were followed. Anything not done in accordance with published standards, regulations, and directives could bring blame upon whoever violated the regulation. Being blamed for an accident is expensive and damaging.

This page intentionally left blank

Table of Contents

Section 3
Fundamentals of Bridge Inspection

This page intentionally left blank

Topic 3.4 Inspection Equipment

3.4.1 Equipment Necessity

Several factors play a role in what type of equipment is needed for an inspection. Bridge location and type are two of the main factors in determining equipment needs. If the bridge is located over water, certain pieces of equipment such as life jackets and boats are important to have. Also, if the bridge is made of timber, then specific pieces of equipment like increment borers and ice picks are needed, whereas they would not be necessary on a steel or concrete bridge. Another factor influencing equipment needs is the type of inspection. It is therefore important to review every facet about the bridge before heading out on an inspection. A few minutes spent reviewing the bridge files and making a list of the necessary equipment can save hours of wasted inspection time in the field if the inspectors do not have the required equipment.

3.4.2 Standard Tools

In order for the inspector to perform an accurate and comprehensive inspection, the proper tools must be used. Standard tools that an inspector should have available at the bridge site can be grouped into seven basic categories:

- Tools for cleaning (see Figure 3.4.1)
- Tools for inspection (see Figure 3.4.2)
- Tools for visual aid (see Figure 3.4.3)
- Tools for measuring (see Figure 3.4.4)
- Tools for documentation
- Tools for access
- Miscellaneous equipment

Figure 3.4.1 Tools for Cleaning

Figure 3.4.2 Tools for Inspection

Figure 3.4.3 Tools for Visual Aid

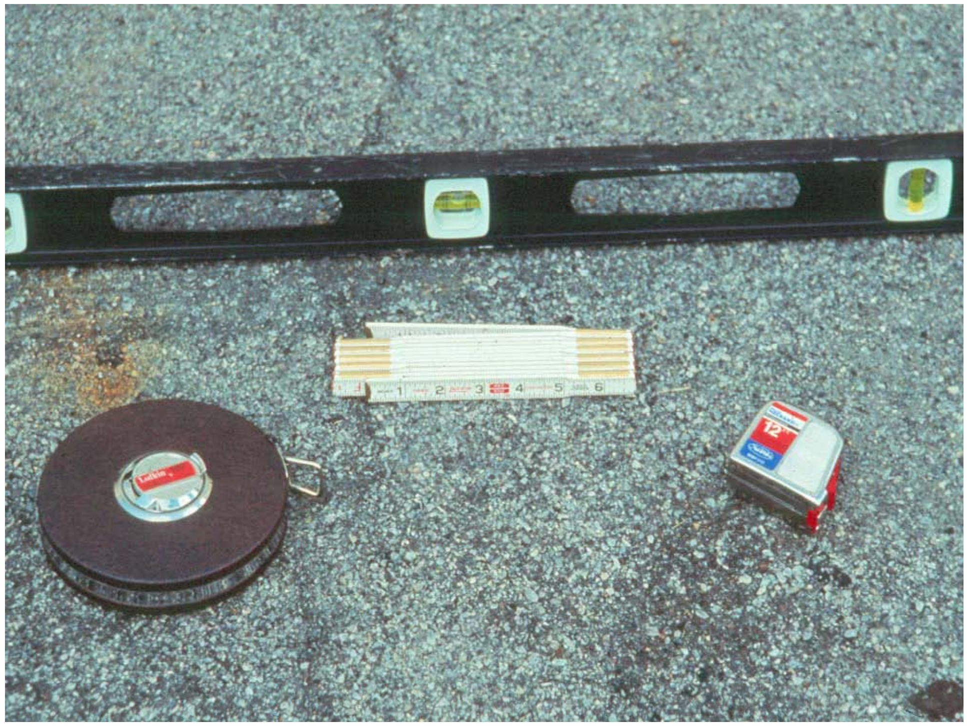


Figure 3.4.4 Tools for Measuring

Tools for Cleaning

Tools for cleaning include:

- Wisk broom - used for removing loose dirt and debris
- Wire brush - used for removing loose paint and corrosion from steel elements
- Scrapers (2 inch or 50 mm) - used for removing corrosion or growth from element surfaces
- Flat bladed screwdriver - used for general cleaning and probing
- Shovel - used for removing dirt and debris from bearing areas

Tools for Inspection

Tools for inspection include:

- Pocket knife - used for general duty
- Ice pick - used for surface examination of timber elements
- Hand brace and bits - used for boring suspect areas of timber elements
- Increment borer - used for internal examination of timber elements
- Chipping hammer with leather holder (16 ounce geologist's pick) - used for loosening dirt and rust scale, sounding concrete, and checking for sheared or loose fasteners
- Plumb bob - used to measure vertical alignment of a superstructure or substructure element
- Tool belt with tool pouch - used for convenient holding and access of small tools
- Chain drag - used to identify areas of delamination on concrete decks
- Range pole / probe - used for probing for scour holes

Tools for Visual Aid

Tools for visual aid include:

- Binoculars - used to preview areas prior to inspection activity and for

examination at distances
- Flashlight - used for illuminating dark areas
- Lighted magnifying glass (e.g., five power and 10 power) - used for close examination of cracks and areas prone to cracking
- Inspection mirrors - used for inspection of inaccessible areas (e.g., underside of deck joints)
- Dye penetrant - used for identifying cracks and their lengths

Tools for Measuring

Tools for measuring include:

- Pocket tape (6 foot rule) - used to measure defects and element and joint dimensions
- 25 foot and 100 foot tape - used for measuring component dimensions
- Calipers - used for measuring the thickness of an element beyond an exposed edge
- Optical crack gauge - used for precise measurements of crack widths
- Paint film gauge - used for checking paint thickness
- Tiltmeter and protractor - used for determining tilting substructures and for measuring the angle of bearing tilt
- Thermometer - used for measuring ambient air temperature and superstructure temperature
- 4 foot carpenter's level - used for measuring deck cross-slopes and approach pavement settlement
- D-Meter (ultrasonic thickness gauge) - used for accurate measurements of steel thickness
- Electronic Distance Meter (EDM) - used for accurate measurements of span lengths and clearances when access is a problem
- Line level and string line

Tools for Documentation

Tools for documentation include:

- Inspection forms, clipboard, and pencil - used for record keeping for most bridges
- Field books - used for additional record keeping for complex structures
- Straight edge - used for drawing concise sketches
- 35 mm camera - used for visual documentation of the bridge site and conditions
- Polaroid camera - used to provide instant documentation for serious conditions which require immediate review by office personnel
- Digital camera - used to provide digital images of defects which can be downloaded and e-mailed for instant assessment
- Chalk, keel, paint sticks, or markers - used for element and defect identification for improved organization and photo documentation
- Center punch - used for applying reference marks to steel elements for movement documentation (e.g., bearing tilt and joint openings)
- "P-K" nails - Parker Kalon masonry survey nails used for establishing a reference point necessary for movement documentation of substructures and large cracks

Tools for Access

Some common tools for access include:

- Ladders - used for substructures and various areas of the superstructure
- Boat - used for soundings and inspection; safety for over water work
- Rope - used to aid in climbing
- Waders - used for shallow streams

Tools for access are described in further detail in Topic 3.5.2.

Miscellaneous Equipment Miscellaneous equipment should include:

- "C"-clamps - used to provide a "third hand" when taking difficult measurements
- Penetrating oil - aids removal of fasteners, lock nuts, and pin caps when necessary
- Insect repellent - reduces attack by mosquitoes, ticks, and chiggers
- Wasp and hornet killer - used to eliminate nests to permit inspection
- First-aid kit - used for small cuts, snake bites, and bee stings
- Dust masks or respirators - used to protect against inhalation in dusty condition or work around pigeon droppings
- Coveralls - used to protect clothing and skin against sharp edges while inspecting
- Life jacket - used for safety over water
- Cell phone - used to call in emergencies
- Toilet paper - used for other "emergencies" (better safe than sorry)

3.4.3 Special Equipment

For the routine inspection of a common bridge, special equipment is usually not necessary. However, with some structures, special inspection activities require special tools. These special activities are often subcontracted by the agency responsible for the bridge. The inspector should be familiar with special equipment and its application.

Survey Equipment Special circumstances may require the use of a transit, a level, an incremental rod, or other survey equipment. This equipment can be used to establish a component's exact location relative to other components, as well as an established reference point.

Nondestructive Testing Equipment Nondestructive testing (NDT) is the in-place examination of a material for structural integrity without damaging the material. NDT equipment allows the inspector to "see" inside a bridge element and assess deficiencies that may not be visible with the naked eye. Generally, a trained technician is necessary to conduct NDT and interpret their results. For a more detailed description of NDT, refer to Topics 13.1.2, 13.2.2, and 13.3.2.

Underwater Inspection Equipment Underwater inspection is the examination of substructure units and the channel below the water line. When the waterway is shallow, underwater inspection can be performed above water with a simple probe. Probing can be performed using a piece of reinforcing steel, a survey rod, a folding rule, or even a tree limb.

When the waterway is deep, underwater inspection must be performed by trained divers. This requires special diving equipment that includes a working platform, fathometer, ground penetrating radar, air supply systems, radio communication, and sounding equipment. Refer to Topic 11.3 for a more detailed description of underwater inspection equipment.

Other Special Equipment An inspection may require special equipment to prepare the bridge prior to the inspection. Such special equipment includes:

- Air-water jet equipment - used to clean surfaces of dirt and debris
- Sand or shot blasting equipment - used to clean steel surfaces to bare metal
- Burning, drilling, and grinding equipment

3.4.4 Primary Safety Concerns

The main safety concerns in any inspection are the public and the inspectors. Having the proper equipment can play a key role in maintaining the safety of both. When inspectors do not have the right equipment, they may attempt to use an alternate piece of equipment that is not really designed for the job. This cannot only prove dangerous for the inspector, but for the public as well. Inspectors should never try to replace equipment in the interest of saving time. The best way to avoid these circumstances is to ensure the inspectors have the proper equipment for the job. This responsibility lies not only with the inspector but also their employer. It is important that the employer make every effort to properly equip all the inspectors. Also, the inspector should be familiar with every piece of equipment and how to use and operate it properly.

Table of Contents

Section 3
Fundamentals of Bridge Inspection

This page intentionally left blank

Topic 3.5 Methods of Access

3.5.1 Introduction

The two primary methods of gaining access to hard to reach areas of a bridge are access equipment and access vehicles. Common access equipment includes ladders, rigging, and scaffolds, while common access vehicles include manlifts, bucket trucks, and snoopers. In most cases, using a manlift or bucket truck will be less time consuming than using a ladder or rigging to inspect a structure. The time saved, however, must offset the high costs associated with operating access vehicles.

3.5.2 Types of Access Equipment

The purpose of access equipment is to position the inspector close enough to the bridge component so that a "hands-on" inspection can be performed. The following are some of the most common forms of access equipment.

Ladders

Ladders can be used for inspecting the underside of a bridge or for inspecting substructure units. However, a ladder should be used only for those portions of the bridge that can be reached safely, without undue leaning.

Figure 3.5.1 Inspection with a Ladder

Ladders can also be used to climb down to access elements of the bridge. The hook-ladder, as it is commonly referred to, is fastened securely to the bridge framing.

Figure 3.5.2 Use of a Hook-ladder

When using a hook-ladder, the inspector should be tied off to a separate safety line, independent of the ladder.

Rigging

Rigging of a structure consists of cables and platforms. Rigging is used to gain access to floor systems and the bottom of main load carrying members in areas where access by other means is not feasible or where special inspection procedures are required (e.g., nondestructive testing and pin removal). Rigging is often used over water, over busy highways or railroads where sufficient clearance exists. Rigging is a good choice for a load posted bridge that does not have the capacity to support an inspection vehicle.

Figure 3.5.3 Rigging for Substructure Inspection

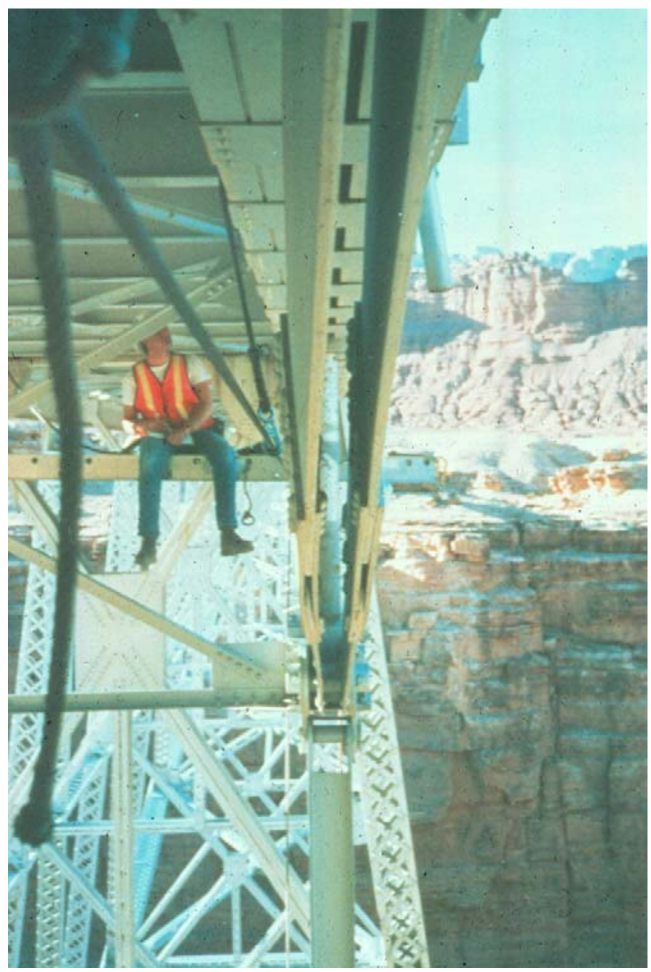


Figure 3.5.4 Rigging for Superstructure Inspection

Scaffolds

They provide an efficient access alternative for structures that are less than 40 feet (12 m) high and over level ground with little or no traffic.

Figure 3.5.5 Scaffold

Boats or Barges

A boat or barge may be needed for structures over water. A boat can be used for some inspection, as well as for taking photographs. Also, a safety boat is required when performing an inspection over water.

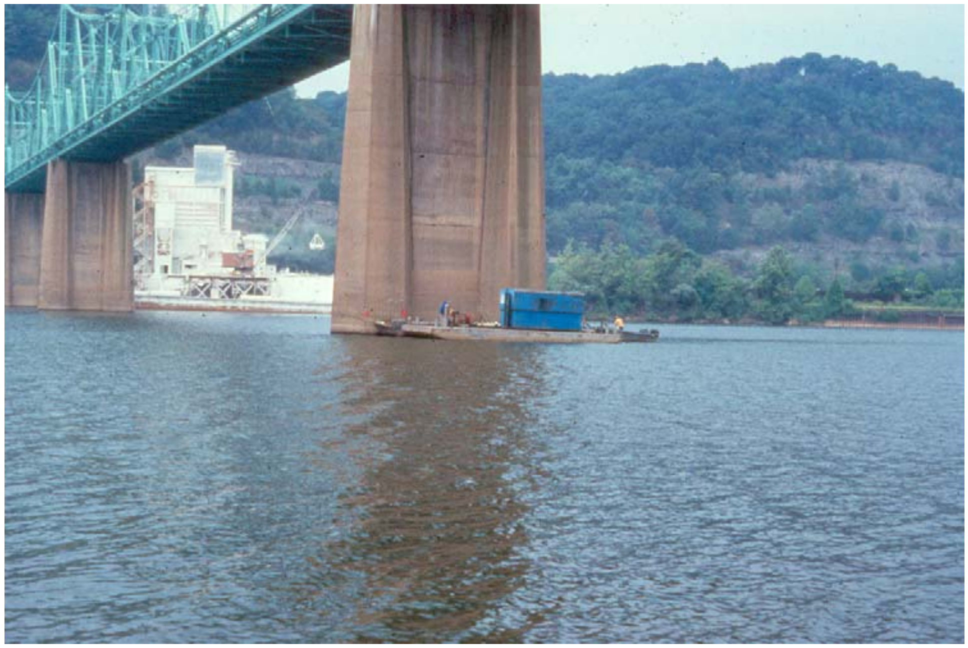


Figure 3.5.6 Inspection Operations from a Barge

A barge may also be used in combination with other access equipment or vehicles to perform an inspection. The barge may be temporarily anchored in place to provide a platform for a manlift or for underwater inspections.

Climbers

Climbers are mobile inspection platforms or cages that "climb" steel cables. They are well suited for the inspection of high piers and other long vertical faces of bridge members. A few of the more common climbers are Spiders and Skyclimbers.

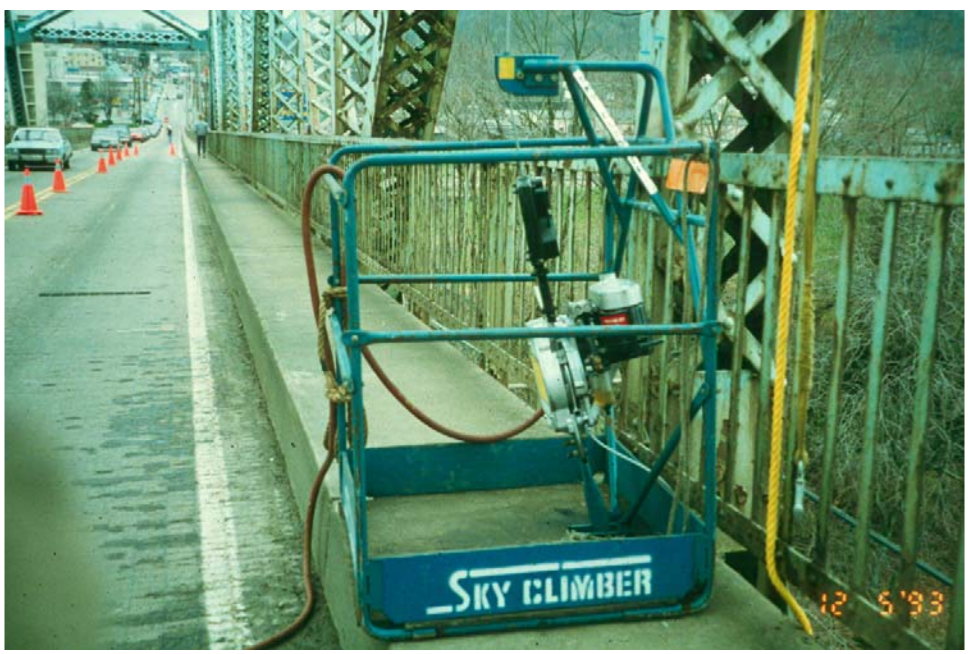


Figure 3.5.7 Climber

Floats

A float is a wood plank work platform hung by ropes. Floats are generally used for access in situations where the inspector will be at a particular location for a relatively long period of time.

Figure 3.5.8 Float

Bosun (or Boatswain) Chairs / Rappelling

Bosun (or boatswain) chairs are suspended with a rope and can carry only one inspector at a time. They can be raised and lowered with block and tackle devices. Rappelling is a similar access method to the Bosun chair but utilizes different equipment and techniques. However, both methods require the use of independent safety lines.

Figure 3.5.9 Bosun Chair

Free Climbing

On some structures, if other methods of access are not practical, inspectors must climb the bridge elements. Safety awareness should be foremost in the inspector's mind when utilizing this technique. Climbing can be divided into two categories. The first category is free climbing, in which the inspector climbs freely, unsecured to the bridge. (Whenever possible, a safety line should be provided for the inspector to secure a lanyard.) The second category employs rappelling techniques and safety equipment.

Figure 3.5.10 Climbing

Permanent Inspection Structures

On some structures, inspection access is included in the design and construction of the bridge. These are typically found on long span structures or more complex designs. Although these inspection platforms only give access to a limited portion of the bridge, they do provide a safe and effective means for the inspector to work. The following are some examples of permanent inspection structures.

Catwalks

A catwalk is an inspection platform typically running parallel to the girders from abutment to abutment under the superstructure. Catwalks can be used to inspect parts of the superstructure and some portions of the substructure. The range of inspection area is limited to those locations near the catwalk.

Figure 3.5.11 Catwalk

Traveler

A traveler is another permanent inspection platform similar to a catwalk except that it is movable. A traveler platform is typically perpendicular to the girders and the platform runs on a rail system between substructure elements. Having the platform perpendicular to the girders allows the inspectors a wider range of movement and enables them to see more if not all of the superstructure elements.

Figure 3.5.12 Traveler Platform

Handrails

Handrails are also used to aid an inspector. Handrails can be used in a number of different locations on the bridge. On the main suspension cables, on top of the pier caps, and on the inside of a girder are just a few locations where handrails may be built. Handrails are typically provided to assist the inspector when free climbing on the bridge and give the inspector a place to secure their lanyard and safety harness.

Figure 3.5.13 Handrail

Inspection Robots

Currently, efforts are being made for robots to be used for inspection purposes. Though still early in the development stage, robots may prove to be an important addition to the inspector's access equipment. Although a robot can never replace a qualified inspector, it can provide information that may not be visible to the human eye. A robot equipped with sonar capabilities can detect internal flaws in bridge members. Also, a robot can be used in situations that are too difficult to reach or extremely dangerous for a human.

3.5.3 Types of Access Vehicles

There are also many types of vehicles available to assist the inspector in gaining access to bridge elements. The following are some of the most common types of access vehicles.

Manlift

A manlift is a vehicle with a platform or bucket capable of holding one or more inspectors. The platform is attached to a hydraulic boom that is mounted on a carriage. An inspector "drives" the carriage using controls in the platform. This type of vehicle is usually not licensed for use on highways. However, some manlifts are nimble and can operate on a variety of terrains. Although four wheel drive models are available, manlifts are limited to use on fairly level terrain. Manlifts come in a number of different sizes with vertical reaches ranging from 12 m (40 feet) to over 52 m (170 feet).

Figure 3.5.14 Manlift **(Note: Adjacent power lines that must be cleared.)**

Bucket Truck

A bucket truck is similar to a manlift. However, a bucket truck can be driven on a highway, and the inspector controls only the bucket. As with the manlift, a bucket truck should be used on fairly level terrain. Bucket trucks have a number of different features and variations:

- Lift capability - varies 7.5 to 15 m (25 to 50 feet).

- Rotating turret - turning range (i.e., the rotational capability of the turret) varies with each vehicle.

- Outriggers - tucket trucks that offer extended reach and turning range have outriggers or supports that are lowered from the chasis of the vehicle to help maintain stability.

- Telescoping toom - some booms may be capable of extending and retracting, providing a greater flexibility and reach area from a given truck location.

- Truck movement - some vehicles offer stable operations without outriggers and can move along the bridge during inspection activities. Vehicles that require outriggers for stable operations cannot be moved during the inspection unless the outriggers have wheels.

- Multiple booms - some bucket trucks have more than one boom, and provide reach up to 15 m (50 feet).

Figure 3.5.15 Bucket Truck

Underbridge Inspection Vehicle

An underbridge inspection vehicle is a specialized bucket truck with an articulated boom designed to reach under a structure while parked on the deck. A rotating turret provides maximum flexibility, and outriggers with wheels allow the truck to be moved during operations. Usually the third boom has the capacity for extending and retracting, allowing for greater reach under a structure. Some of the larger underbridge inspection vehicles have four booms, allowing an even greater reach.

Figure 3.5.16 Underbridge Inspection Vehicle

Many of the features on an underbridge inspection vehicle are standardized on all models. Some of the common features include:

- Rotating turret - provides maximum flexibility.

- Outriggers with wheels - allow for moving the truck during operations.

- Telescoping third boom - usually the third boom has the capability for extending and contracting; this allows for greater reach under a structure.

Variations and options available on different models include:

- Capacity - Some underbridge inspection vehicles have a two or three person bucket on the end of the third boom. Other models are equipped with a multiple-person platform on the third boom with a ladder on the second boom. Still other models may have the capability of interchanging a bucket and a platform in the shop.

- Telescoping second boom - Some underbridge inspection vehicle models have a second boom that can extend and contract, providing greater movement in the vertical direction.

- Articulated third (or fourth) boom - Some underbridge inspection vehicle models have a small third or fourth boom that allows for greater vertical movement under the structure. This option is particularly useful on bridges with deep superstructure members.

Platform Truck

Another underbridge inspection vehicle is a platform truck, which combines the underbridge reach capability of a snooper truck with the freedom of movement of platform rigging. The platform is lowered from the truck to under the bridge by means of an articulated boom, similar to the underbridge inspection crane. The platform can then telescope out to provide inspection access to a wide range of the superstructure and substructure. Outriggers with wheels allow the truck to be moved during operation. The inspector is now free to walk from beam to beam without having to reposition the platform. This combination allows for an efficient and thorough inspection.

Figure 3.5.17 Platform Truck

3.5.4 Method of Access and Cost Efficiency

In most cases, even the most sluggish lift device will be quicker than using a ladder or rigging to inspect a structure. The time saved, however, must offset the high costs associated with obtaining and operating the vehicle.

In assessing the time-saving effectiveness of a lift device, the following questions should be answered:

- Can the bridge be inspected by other reasonable methods?
- What type of vehicle is available?
- How much of the bridge can be inspected using the vehicle?
- How much of the bridge can be inspected from one setup?
- How much time does it take to inspect at each setup?

- How much time does it take to move from one setup to the next?
- Does the vehicle require an operator or driver other than the inspector?
- Will the use of the vehicle require special traffic control?

The inspection time and vehicle costs can then be compared to costs associated with using standard access equipment.

3.5.5 Safety Considerations

Safety should be a primary concern on any job site, not only of the workers but of the public as well. The equipment and vehicles being used also have safety considerations.

Access Equipment

Before the bridge inspection begins, an equipment inspection should be performed. As a minimum, inspect access equipment as per the manufacture's guidelines. Using faulty equipment can lead to serious accidents and even death. The inspector should check all the equipment and verify that it is in good working condition with no defects or problems. If rigging or scaffolding is being used, it should be checked to ensure that it was installed properly and all cables and planks are secured tightly. When climbing, check for loose clothing or articles that can get caught. OSHA-approved safety harnesses with shock absorbing lanyards should be worn at all time when using access equipment and vehicles.

Access Vehicles

If the inspector is not familiar with the inspection vehicle being used, he should take the time required to become accustomed to the operation. In some cases, formal operator training may be necessary or required. When operating any inspection vehicle, always be aware of any overhead power lines or any other hazards that may exist. It is also important to be aware of any restrictions on the vehicle, such as weight limits for the bucket, support surface slope limits, and reach restrictions, so as to avoid situations that will cause accidents. Always be alert to your location also. You do not want to boom out into an unprotected traffic lane or near electrical lines.

Table of Contents

This page intentionally left blank

Section 4

Bridge Inspection Reporting System

Topic 4.1 Structure Inventory

4.1.1 Introduction

A good bridge inspection reporting system is essential to document bridge conditions and to protect the public's safety and investment in bridge structures. It is, therefore, essential that bridge inspection data be clear, accurate, and complete, since it is an integral part of the lifelong record file of the bridge.

Because of the requirements that must be fulfilled for the National Bridge Inspection Standards (NBIS), it is necessary to employ a uniform bridge inspection reporting system. A uniform reporting system is essential to evaluate the condition of a structure correctly and efficiently. It is a valuable aid in establishing maintenance priorities and replacement priorities, and in determining structure capacity and the cost of maintaining the nation's bridges. Consequently, importance of the reporting system cannot be overemphasized. Success of any bridge inspection program is dependent upon its reporting system.

4.1.2 FHWA Structure Inventory, Appraisal and Condition Ratings

The FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*Coding Guide*) is used for establishing the bridge inventory and the overall condition of the deck, superstructure, substructure, and channel. It is not an inspection guide. Each state is encouraged to use codes and instructions in the *Coding Guide*, but its direct use is optional. Each state may use its own coding scheme, provided that the data is directly translatable into the format of the *Coding Guide*. In other words, the states are responsible for having the capability to obtain, store, and report certain information about bridges, whether or not the *Coding Guide* is used by the inspectors.

The Structure Inventory and Appraisal (SI&A) sheet is a tabulation of information that must be submitted for each individual structure (see Figure 4.1.1). Any requests by the FHWA for submittals of SI&A data will be based on the definitions, explanations, and codes supplied in this manual, its supplements, and the *Coding Guide*.

Sometimes inventory data is not available for new or small bridges and culverts.

For the small bridges and culverts that are less than 20 feet, some states still collect the inventory information and generate a “local” database. The inspector must gather enough information in order to establish inventory data.

It is important to note that the SI&A sheet is not an inspection form. Rather, it is a summary sheet of bridge data required by the FHWA to effectively monitor and manage the National Bridge Inspection Program and the Highway Bridge Replacement and Rehabilitation Program.

Substitutes for the SI&A Sheet

NBIS allows the use of suitable substitutes for the SI&A sheet. The only requirement is that the forms must be standardized. Some states simply reprint the federal form with the same items and item numbers. A few states have elaborate Bridge Management Systems (BMS) with different item numbers that collect all the data listed on the SI&A form plus additional items not reported to the FHWA (see Figures 4.1.1, 4.1.2 and 4.1.3).

FHWA — *Office of Asset Management DOT*

Structure Inventory and Appraisal Sheet

Bridge Key: 11 0013 | Agency ID: 11 0013 | Sufficiency Rating: 96.8

IDENTIFICATION

State 1: 06 California | Struc Num 8: 11 0013
Facility Carried 7: STATE ROUTE 162 | Location 9: 03-GLE-162-_-73.55
Rte.(On/Under)5A: Route On Structure | Rte. Signing Prefix 5B: 3 State Hwy
Level of Service 5C: 1 Mainline | Rte. Number 5D: 00162
Directional Suffix 5E: 0 N/A (NBI) | % Responsibility : Unknown
SHD District 2: District 3 | County Code 3: (11)GLENN
Place Code 4: Unknown | Kilometer Post 11: 73.6 km
Feature Intersected 6: BRUSH CANAL
Latitude 16: 39d 31' 18" | Longitude 17: 122d 03' 42"
Border Bridge Code 98: Unknown (P)
Border Bridge Number 99: Unknown

STRUCTURE TYPE AND MATERIALS

Number of Approach Spans 46: 0 | Number of Spans Main Unit 45: 2
Main Span Material/Design 43A/B:
2 Concrete Continuous | 01 Slab
Deck Type 107: 1 Concrete-Cast-in-Place
Wearing Surface 108A: 1 Monolithic Concrete
Membrane 108B: 0 None
Deck Protection 108C: None

AGE AND SERVICE

Year Built 27: 1963 | Year Reconstructed 106: Unknown
Type of Service on 42A: 1 Highway
Type of Service under 42B: 5 Waterway
Lanes on 28A: 2 | Lanes Under 28B: 0 | Detour Length 19: 13 km
ADT 29: 1,900 | Truck ADT 109: 12 % | Year of ADT 30: 1994

GEOMETRIC DATA

Length Max Span 48: 6.40 m | Structure Length 49: 13.70 m
Curb/Sdwlk Wdth L 50A: 0.00 m | Curb/Sidewalk Width R 50B: 0.00 m
Width Curb to Curb 51: 10.80 m | Width Out to Out 52: 11.30 m
Approach Roadway Width 32: (w/ shoulders) 9.80 m | Median 33: 0 No median
Deck Area: 155.00 m²
Skew 34: 5.00 ° | Structure Flared 35: 0 No flare
Minimum Vertical Clearance Over Bridge 53: 99.99 m
Minimum Vertical Underclearance Reference 54A: N Feature not hwy or RR
Minimum Vertical Underclearance 54B: 00.00 m
Minimum Lateral Underclearance Reference R 55A: N Feature not hwy or RR
Minimum Lateral Undrclearance R 55: 99.90 m
Minimum Lateral Undrclearance L 56: 00.00 m

INSPECTION

Frequency 91: 24 months | Inspection Date 90: 10/28/1997 | Next Inspection: 10/28/1999
FC Frequency 92A: NA | FC Inspection Date 93A: NA | Next FC Inspection: NA
UW Frequency 92B: NA | UW Inspection Date 93B: NA | Next UW Inspection: NA
SI Frequency 92C: NA | SI Date 93C: NA | Next SI: NA
Element Frequency: 24 months | Element Inspection Date: 12/11/1997 | Next Elem. Insp. Due: 10/28/1999

CLASSIFICATION

Defense Highway 100: 0 Not a STRAHNET hwy | Parallel Structure 101: No || bridge exists
Direction of Traffic 102: 2 2-way traffic | Temporary Structure 103: Unknown (NBI)
Highway System 104: 0 Not on NHS | NBIS Length 112: Long Enough
Toll Facility 20: 3 On free road | Functional Class 26: 06 Rural Minor Arterial
Historical Significance 37: 5 Not eligible for NRHP
Owner 22: 1 State Highway Agency
Custodian 21: 1 State Highway Agency

CONDITION

Deck 58: 7 Good | Super 59: 7 Good | Sub 60: 7 Good
Culvert 62: N N/A (NBI) | Channel/Channel Protection 61: 8 Protected

LOAD RATING AND POSTING

Inventory Rating Method 65: 1 LF Load Factor | Operating Rating Method 63: 1 LF Load Factor
Inventory Rating 66: MS20.7 | Operating Rating 64: MS34.2
Design Load 31: 5 MS 18 (HS 20) | Posting 70: 5 At/Above Legal Loads
Posting status 41: A Open, no restriction

APPRAISAL

Bridge Rail 36A: 1 Meets Standards | Approach Rail 36C: 0 Substandard
Transition 36B: 0 Substandard | Approach Rail Ends 36D: 0 Substandard
Str. Evaluation 67: 7 Above Min Criteria | Deck Geometry 68: 6 Equal Min Criteria
Underclearance, Vertical and Horizontal 69: N Not applicable (NBI)
Waterway Adequacy 71: 8 Equal Desirable | Approach Alignment 72: 8 Equal Desirable Crit
Scour Critical 113: 6 Calcs not made

PROPOSED IMPROVEMENTS

Bridge Cost 94: $ 0 | Type of Work 75: Unknown (P)
Roadway Cost 95: $ 0 | Length of Improvment 76: 00.00 m
Total Cost 96: $ 0 | Future ADT 114: 2,900
Year of Cost Estimate 97: Unknown | Year of Future ADT 115: 2010

NAVIGATION DATA

Navigation Control 38: 0 Permit Not Required
Vertical Clearance 39: 0.00 m | Horizontal Clearance 40: 0.00 m
Pier Protection 111: Unknown (NBI) | Lift Bridge Vertical Clearance 116:

ELEMENT CONDITION STATE DATA

Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
2	38/2	Bare Concrete Slab	sq.m.	160	100 %	160	0 %	0	0 %	0	0 %	0	0 %	0
2	205/2	R/Conc Column	ea.	5	100 %	5	0 %	0	0 %	0	0 %	0	0 %	0
2	215/2	R/Conc Abutment	m.	23	100 %	23	0 %	0	0 %	0	0 %	0	0 %	0
2	226/2	P/S Conc Submgd Pile	ea.	13	100 %	13	0 %	0	0 %	0	0 %	0	0 %	0
2	333/2	Other Bridge Railing	m.	35	100 %	35	0 %	0	0 %	0	0 %	0	0 %	0
2	358/2	Deck Cracking SmFlag	ea.	1	100 %	1	0 %	0	0 %	0	0 %	0	0 %	0

INSP001_Inspection_SIA_Metric — **Mon 6/10/2002 07:54:09** **Page 1 of 1**

Figure 4.1.1 FHWA SI&A Sheet with Element Level Data

Appendix A

Structure Inventory and Appraisal Sheet

OMB No. 2125-0501

NATIONAL BRIDGE INVENTORY - - - - - - - STRUCTURE INVENTORY AND APPRAISAL 10/15/94

********** IDENTIFICATION **********************

(1) STATE NAME - ________ CODE ___
(8) STRUCTURE NUMBER #________
(5) INVENTORY ROUTE (ON/UNDER) - ____ = ______
(2) HIGHWAY AGENCY DISTRICT __
(3) COUNTY CODE ___ (4) PLACE CODE _____
(6) FEATURES INTERSECTED - ________
(7) FACILITY CARRIED - ________
(9) LOCATION - ________
(11) MILEPOINT/KILOMETERPOINT ___.___
(12) BASE HIGHWAY NETWORK - ________ CODE _
(13) LRS INVENTORY ROUTE & SUBROUTE #________
(16) LATITUDE __ DEG __ MIN __.__ SEC
(17) LONGITUDE __ DEG __ MIN __.__ SEC
(98) BORDER BRIDGE STATE CODE ___ % SHARE __ %
(99) BORDER BRIDGE STRUCTURE NO. #________

********** STRUCTURE TYPE AND MATERIAL *********

(43) STRUCTURE TYPE MAIN: MATERIAL - ________
TYPE - ________ CODE ___
(44) STRUCTURE TYPE APPR: MATERIAL - ________
TYPE - ________ CODE ___
(45) NUMBER OF SPANS IN MAIN UNIT ___
(46) NUMBER OF APPROACH SPANS ____
(107) DECK STRUCTURE TYPE - ________ CODE _
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE - ________ CODE _
B) TYPE OF MEMBRANE - ________ CODE _
C) TYPE OF DECK PROTECTION - ________ CODE _

********** AGE AND SERVICE *********************

(27) YEAR BUILT ____
(106) YEAR RECONSTRUCTED ____
(42) TYPE OF SERVICE: ON - ________
UNDER - ________ CODE __
(28) LANES: ON STRUCTURE __ UNDER STRUCTURE __
(29) AVERAGE DAILY TRAFFIC ______
(30) YEAR OF ADT ____ (109) TRUCK ADT __ %
(19) BYPASS, DETOUR LENGTH ___ KM

********** GEOMETRIC DATA **********************

(48) LENGTH OF MAXIMUM SPAN ____._ M
(49) STRUCTURE LENGTH _____._ M
(50) CURB OR SIDEWALK: LEFT __._ M RIGHT __._ M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB ___._ M
(52) DECK WIDTH OUT TO OUT ___._ M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) ___._ M
(33) BRIDGE MEDIAN - ________ CODE _
(34) SKEW __ DEG (35) STRUCTURE FLARED _
(10) INVENTORY ROUTE MIN VERT CLEAR __.__ M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR ___._ M
(53) MIN VERT CLEAR OVER BRIDGE RDWY __.__ M
(54) MIN VERT UNDERCLEAR REF - ____ __.__ M
(55) MIN LAT UNDERCLEAR RT REF - ____ __._ M
(56) MIN LAT UNDERCLEAR LT __._ M

********** NAVIGATION DATA *********************

(38) NAVIGATION CONTROL - ________ CODE _
(111) PIER PROTECTION - ________ CODE _
(39) NAVIGATION VERTICAL CLEARANCE ___._ M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR ___._ M
(40) NAVIGATION HORIZONTAL CLEARANCE ____._ M

****

SUFFICIENCY RATING = ___._
STATUS = ________

********** CLASSIFICATION ***************** CODE

(112) NBIS BRIDGE LENGTH - __
(104) HIGHWAY SYSTEM - ________ _
(26) FUNCTIONAL CLASS - ________ __
(100) DEFENSE HIGHWAY - ________ _
(101) PARALLEL STRUCTURE - ________ _
(102) DIRECTION OF TRAFFIC - ________ _
(103) TEMPORARY STRUCTURE - ________ _
(105) FEDERAL LANDS HIGHWAYS - ________ _
(110) DESIGNATED NATIONAL NETWORK - ________ _
(20) TOLL - ________ _
(21) MAINTAIN - ________ __
(22) OWNER - ________ __
(37) HISTORICAL SIGNIFICANCE - ________ _

********** CONDITION ********************** CODE

(58) DECK _
(59) SUPERSTRUCTURE _
(60) SUBSTRUCTURE _
(61) CHANNEL & CHANNEL PROTECTION _
(62) CULVERTS _

********** LOAD RATING AND POSTING ******** CODE

(31) DESIGN LOAD - ______ OR ______ _
(63) OPERATING RATING METHOD - ________ _
(64) OPERATING RATING - ______ __._
(65) INVENTORY RATING METHOD - ________ _
(66) INVENTORY RATING - ______ __._
(70) BRIDGE POSTING - ________ _
(41) STRUCTURE OPEN, POSTED OR CLOSED - _
DESCRIPTION - ________

********** APPRAISAL ********************** CODE

(67) STRUCTURAL EVALUATION _
(68) DECK GEOMETRY _
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL _
(71) WATERWAY ADEQUACY _
(72) APPROACH ROADWAY ALIGNMENT _
(36) TRAFFIC SAFETY FEATURES ____
(113) SCOUR CRITICAL BRIDGES _

********** PROPOSED IMPROVEMENTS ***************

(75) TYPE OF WORK - ________ CODE ___
(76) LENGTH OF STRUCTURE IMPROVEMENT ____._ M
(94) BRIDGE IMPROVEMENT COST $__,___,000
(95) ROADWAY IMPROVEMENT COST $__,___,000
(96) TOTAL PROJECT COST $__,___,000
(97) YEAR OF IMPROVEMENT COST ESTIMATE ____
(114) FUTURE ADT ______
(115) YEAR OF FUTURE ADT ____

********** INSPECTIONS *************************

(90) INSPECTION DATE __/__ (91) FREQUENCY __ MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL - __ - __ MO A) __/__
B) UNDERWATER INSP - __ - __ MO B) __/__
C) OTHER SPECIAL INSP - __ - __ MO C) __/__

Figure 4.1.2 FHWA SI&A Sheet with NBI Data Only

Printed: 5/15/2000 | **ARIZONA DEPARTMENT OF TRANSPORTATION** | Page 1 of 1

BRIDGE GROUP

Structure Inventory & Appraisal

Structure Number: 4023	Structure Name: RCB		Feature Under: WASH
Route: 60 MP 56.85	Road Name: US 60	Agency: ADOT	Location: 7.3 M E JCT SR 72

LOCATION INFORMATION

N1-State Code:	049
N2-State Hwy District:	88
N3-County Code:	029
N4-Place Code:	00000
N16-Latitude:	33 deg 47.1 min
N17-Longitude:	113 deg 36.5 min
N98-Border St Code - % Resp:	- 0
N99-Border Bridge Number:	

INVENTORY ROUTE DATA

		2nd Record
N19-Detour Length (miles):	20	
N20-Toll:	3	
N28-Lanes On / Under:	2 / 0	
N5-Inv Rte:	1 2 0 00060 0 -	
N10-Inv Rte Min Vert Clr (feet):	99.99	0
N11-Inv Rte Milepoint:	56.85	0
N26-Functional Class:	07	
N29-Avg Daily Traffic:	2417	0
N30-Year of ADT:	1998	
N47-Inv Rte Tot Horiz Clr (feet):	39	0
N100-Defense Hwy:	0	
N101-Parallel Bridge:	N	
N102-Direction of Traffic:	2	
N104-Hwy System:	0	
N109-Percent Truck Traffic:	46	0
N110-National Truck Network:	1	
N114-Future ADT:	2427	0
N115-Year of Future ADT:	2020	
A200-Is N5 the Princ. Rte?	Y	N

RESPONSIBILITY

N21-Maint Responsibility:	01
N22-Bridge Owner:	01
A203-ADOT Org Number:	8852
A224-Insp Team Number:	4
A229-Agency:	ADOT

NAVIGATION

38-Navigation Control:	0
39-Nav Vert Clr (feet):	0
40-Nav Horiz Clr (feet):	0
111-Nav Pier/Abut Prot:	
116-Nav Min Vert Clr (feet):	0

GENERAL DATA

33-Bridge Median:	0
34-Skew:	0
35-Structure Flared:	0
37-Historical Significance:	5
107-Deck Str Type:	1
108-Wear Surf Prot System:	6 0 0
201-Wear Surf Thickness (inches):	4

DIMENSIONS

N32-Appr Rdwy Width (feet):	36
N48-Max Span Length (feet):	10
N49-Structure Length (feet):	32
N50a-Lt Curb/Swlk Width (feet):	1
N50b-Rt Curb/Swlk Width (feet):	1
N51-Br Width Curb-Curb (feet):	39
N52-Deck Width Out-Out (feet):	41.6
N112-NBIS Br Length?	Y

VERTICAL and HORIZONTAL CLEARANCE

N53-Min Vert Over Clr (feet):	99.99
N54-Min Vert Under Clr (feet):	N 0
N55-Min Lat Under Clr Rt (feet):	N 99.9
N56-Min Lat Under Clr Lt (feet):	0

SERVICE, TYPE, and SPAN INFORMATION

N42-Service Type:	1 5
N43-Str Type, Main:	2 19
N44-Str Type, Appr:	0 00
N45-Number of Main Spans:	3
N46-Number of Appr Spans:	0

CONDITION RATINGS

N58-Deck:	8
N59-Superstructure:	N
N60-Substructure:	N
N61-Channel:	7
N62-Culvert:	7

APPRAISAL RATINGS

N67-Struct Evaluation:	7
N68-Deck Geometry:	5
N69-Underclearance Rtg:	N
N71-Waterway Adequacy:	6
N72-Appr Rdw Align:	8
N36-Traffic Safety Features:	0 0 0 0

BRIDGE SCOUR DATA

N113-Scour Critical Rtg:	8
A202-Foundation Type:	
A220-Found Embed (feet):	0
A221-Scour Countermeasure:	0 1 0

LOAD, RATE, and POST

N31-Design Loading:	5
N41-Open, Post, Close:	A
N63-Method Used for Oper. Rtg.:	5
N64-Operating Load Rtg:	2 - 36
N65-Method Used for Inv. Rtg.:	5
N66-Inventory Load Rtg:	2 - 36
N70-Bridge Posting:	5
N103-Temp Str Designation:	
A211-Posted Limit (Tons):	0
A222-Date of Load Rtg:	
A233-Posted Vert Clr NB/EB (ft-in):	0 - 0
A233-Posted Vert Clr SB/WB (ft-in):	0 - 0

PROPOSED IMPROVEMENTS

N75-Type of Work:	
N76-Length of Str Imp (feet):	0
N94-Br Improv Cost (x1000):	$0
N95-Rdwy Improv Cost (x1000):	$0
N96-Total Project Cost (x1000):	$0
N97-Year of Cost Estimate:	

CONSTRUCTION PROJECT DATA

N27-Year Built:	1958
N106-Year of Reconstruction:	0000
A204-Orig Project Number:	F-022-1(1)
A205-Orig Project Station:	3045+14.34
A223-TRACS Number:	
A225-Deck Area (sq. feet):	0
A226-Superstr Unit Cost:	$0
A227-Substr Unit Cost:	$0

INSPECTION

N90-Inspection Date:	2/1/2000
N91-Insp Freq (months):	48
A207-Inspection Quarter:	1
A208-Inspection Number:	14
A228-Next Insp Date:	Quarter 1, 2004

CRITICAL FEATURES

N92A-Fracture Critical:	N 0
N92B-Underwater Insp:	N 0
N92C-Special Insp:	N 0
N93A-Date Fract Crit Insp:	0
N93B-Date Underwtr Insp:	0
N93C-Date Spec Insp:	0
A234-Steel In-Depth Insp Freq (mo):	0

CULVERT INFORMATION

A217-Culv Barrel Height (feet)	6
A218-Culv Length (feet):	41
A219-Culv Fill Height (feet):	1

BRIDGE RAILING

A206a-Bridge Rail Type:	6
A206b-Geometric Conform:	0
A206c-Structural Conform:	0

SUFFICIENCY RATING

Sufficiency Rating: 92.32

GENERAL COMMENTS

Figure 4.1.3 Arizona Structural Inventory and Appraisal Sheet

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

Page 1 of 4
DATE PRINTED: 11/14/2000 13:43

Structure ID: 520002

4 Description

Structure Unit Identification

Bridge/Unit ID	520002 0
Description	MAIN SPAN 1
Type	Main Span
NBI Unit Flag	Main ✓ Approach ☐
Curb/Sidewalk (50)	Left 0 ft Right 0 ft
Deck width (52)	0 ft
Bridge Median (33)	No median

Structure Unit Type and Material

Struct Material (43)	Concrete
Design Type	Culvert
Deck Type (107)	Not Applicable
Surface (108)	Not Applicable
Membrane	None
Deck Protection	None
Skew (34)	0 deg

Roadway Identification:

NBI Structure No (8)	520002
Position/Prefix (5)	Route On Structure
Kind Hwy (Rte Prefix)	U.S. Numbered Hwy
Design Level of Service	Mainline
Route Number/Suffix	00090 / Not Applicable
Feature Intersect (6)	US90 SR10/GUM CREEK
Critical Facility	Not Defense-crit
Facility Carried (7)	US 90 SR 10
Mile Point (11)	20.815
Latitude (16)	030d47'39" Long (17) 085d43'28"

Roadway Traffic and Accidents

Lanes (28)	2 Medians 0 Speed 54.681 mph
ADT Class	ADT Class 3
Recent ADT (29)	5100 Year (30) 1998
Future ADT (114)	9490 Year (115) 2020
Truck % ADT (109)	7
Detour Length (19)	1.243 mi
Detour Speed	44.739 mph
Accident Count	-1 Rate -1

Roadway Classification

Nat. Hwy Sys (104)	Not on NHS
National base Net (12)	On Base Network
LRS Inventory Rte (13a)	52 010 000 Sub Rte (13b) 00
Functional Class (26)	Rural Minor Arterial
Eligible for Federal Aid ?	Yes
Defense Hwy (100)	Not a STRAHNET hwy
Direction of Traffic (102)	2-way traffic
Critical Travel Route	☐

Roadway Clearances

Vertical (10)	99.99 ft Appr. Road (32) 34.121 ft
Horiz. (47)	34.121 ft Roadway (51) 0 ft
Truck Network (110)	Not part of natl network
Toll Facility (20)	On free road
Fed. Lands Hwy (105)	Not Applicable
School Bus Route	☐
Transit Route	☐

Figure 4.1.4 Florida Structural Inventory and Appraisal Sheet

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

Page 2 of 4
DATE PRINTED: 11/14/2000 13:43

Structure ID: 520002

Structure Identification

Admin Area	Not located in area
District (2)	D3 - Chipley
County (3)	(52)Holmes
Place Code (4)	No city involved
Location (9)	3.2 KM W OF BONIFAY
Border Br St/Reg (98)	Not Applicable Share 0 %
Border Struct No (99)	
FIPS State/Region (1)	Florida Region 4-Atlanta
NBIS Bridge Len (112)	Meets NBI Length
Parallel Structure (101)	No ll bridge exists
Temp. Structure (103)	Not Applicable
Maint. Resp. (21)	State Highway Agency
Owner (22)	State Highway Agency
Historic Signif. (37)	Not eligible for NRHP

Geometrics

Spans in Main Unit (45)	4
Approach Spans (46)	0
Length of Max Span (48)	9.843 ft
Structure Length (49)	42.979 ft
Deck Area	-1 sqft
Structure Flared (35)	No flare

Age and Service

Year Built (27)	1954
Year Reconstructed (106)	-1
Type of Service On (42a)	Highway
Under (42b)	Waterway
Fracture Critical Details	Not Applicable

3 Appraisal

Structure Appraisal

Open/Posted/Closed (41)	Open, no restriction
Deck Geometry (68)	Not Applicable
Underclearances (69)	Not Applicable
Approach Alignment (72)	No speed red thru curve
Bridge Railings (36a)	Not Applicable
Transitions (36b)	Not Applicable
Approach Guardrail (36c)	Meets Standards
Approach Guardrail ends (36d)	Meets Standards
Scour Critical (113)	Stable Above Footing

Navigation Data

Navigation Control (38)	Permit Not Required
Nav Vertical Clr (39)	0 ft
Nav Horizontal Clr (40)	0 ft
Min Vert Lift Clr (116)	0 ft
Pier Protection (111)	Not Applicable

NBI Condition Rating

Sufficiency Rating	* 99.5
Structural Eval (67)	Above Min Criteria
Deficiency	Not Deficient

Minimum Vertical Clearance

Over Structure (53)	99.99 ft
Under (reference) (54a)	Feature not hwy or RR
Under (54b)	0 ft

Minimum Lateral Underclearance

Reference (55a)	Feature not hwy or RR
Right Side (55b)	0 ft
Left Side (56)	0 ft

Load Rating

Design Load (31)	M 13.5 (H 15)
Rating Date	08/08/1994 Initials JF
Posting (70)	At/Above Legal Loads

Operating Type (63)	LF Load Factor	
Operating rating (64)	68.894 tons	Alternate -1
Inventory Type (65)	LF Load Factor	
Inventory Rating (66)	40.896 tons	Alternate -1
		Alt Meth -1

6 Schedule

Current Inspection

Inspection Date	01/06/2000
Inspector	MT338TK - Tom Klopfenstein
Primary Type	Regular NBI
Review Required	☐

Next Inspection Date **Scheduled**

NBI	01/06/2002
Element	01/06/2002
Fracture Critical	
Underwater	
Other Special	

Inspection Types Performed NBI ☑ Element ☑ Fracture Critical ☐ Underwater ☐ Other Special ☐

Figure 4.1.4 Florida Structural Inventory and Appraisal Sheet (Continued)

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

Page 3 of 4
DATE PRINTED: 11/14/2000 13:43

Structure ID: 520002

Inspection Intervals	Required (92)	Frequency (92)		Last Date (93)	Inspection Resources	
Fracture Critical	☐	mos			Crew Hours	8
Underwater	☐	mos			Flagger Hours	0
Other Special	☐	mos			Helper Hours	0
NBI		24 mos	(91)	01/06/2000 (90)	Snooper Hours	0
					Special Crew Hours	0
					Special Equip Hours	0

5 Custom

General Bridge Information

Parallel Bridge Seq		Bridge Rail 1	Not applicable-No rail
Channel Depth	0.328 ft	Bridge Rail 2	Not applicable-No rail
Radio Frequency	-1	Electrical Devices	No electric service
Phone Number	(000) 000-0001	Culvert Type	Not applicable
Exception Date		Maintenance Yard	Marianna Yard
Exception Type	Unknown		

Bridge Load Rating Information

Govr. Span Length	9.843 ft	Single Unit Truck 2 Axles	48.502 tons
L-Rating Origination	Design Plans	Single Unit Truck 3 Axles	60.627 tons
Load Rating Date	08/08/1994	Single Unit Truck 4 Axles	74.957 tons
Method Calculation	AASHTO formula	Combination Unit Truck 3 Axles	79.366 tons
Load Dist. Factor	0.168	Combination Unit Truck 4 Axles	79.366 tons
Impact Factor	0	Combination Unit Truck 5 Axles	87.083 tons
Design Method	Load Factor	Truck Trailer 5 Axles	95.901 tons
Design Measure	English	Posting Weight	tons
Recommended Single Unit	-1 tons	Posting Single Unit	-1 tons
Recommended Combination	-1 tons	Posting Combination Unit	-1 tons
Recommended Tandem	-1 tons	Posting Tandem Unit	-1 tons

Bridge Scour and Storm Information

Pile Driving Record	Not Applicable	Scour Recommended I	Stop scour evaluations
Foundation Type	Foundation details	Scour Recommended II	Unknown
Mode of Flow	Riverine	Scour Recommended III	Unknown
Rating Scour Eval	Low Risk - Low	Scour Elevation	-1 ft
Highest Scour Eval	Phase I completed	Action Elevation	-1 ft
		Storm Frequency	-1

1 Condition

NBI Rating

Channel (61)	No Deficiencies	Culvert (62)	Minor Deterioration
Deck (58)	Not Applicable	Waterway (71)	8 - Equal Desirable
Superstructure (59)	Not Applicable	Unrepaired Spalls	-1 sq.ft.
Substructure (60)	Not Applicable	Review Required	☐

Figure 4.1.4 Florida Structural Inventory and Appraisal Sheet (Continued)

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

Page 4 of 4
DATE PRINTED: 11/14/2000 13:43

Structure ID: 520002

Elements

Inspection Date: 01/06/2000 GKXW

Span Id	Elem/En	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	290/4	Channel	1	100	0	0	0	0	0	0	0	0	1 ea.
Notes													
0	475/4	R/Conc Walls	154	100	0	0	0	0	0	0	0	0	154 lf.
Notes													
0	241/4	Concrete Culvert	299	82	66	18	0	0	0	0	0	0	364 lf.
Notes	There are a few vertical cracks in the side walls of the original section of culvert.												

Total Number of Elements: 3

Past Inspections

Inspection Date: 01.06.2000 **Type:** Regular NBI
Inspector: MT338TK - Tom Klopfenstein

Inspection Notes: Sufficiency Rating Calculation Accepted by mt338tk at 01/10/2000 13:45:43
MT338TK inspection comments - The left extended portion of culvert is skewed 24 degrees to the left due to stream alignment.
Structure 520002 -
Date 01/06/2000 -
Previous comments > (none)

Inspection Date: 04.01.1998 **Type:** Regular NBI
Inspector: BID

Inspection Notes:

Bridge Notes

Figure 4.1.4 Florida Structural Inventory and Appraisal Sheet (Continued)

Some agencies furnish standardized sketch sheets and photo sheets to inspectors for report generation. Some agencies have developed their forms on software packages for use on portable computers or wearable computers (see Figures 4.1.5 and 4.1.6).

Figure 4.1.5 Wearable Computer with Case

Figure 4.1.6 Inspector Using Wearable Computer

The data and information required of states by the FHWA is listed on the SI&A sheet. It is important to note that the items listed on this sheet apply to both the field and office personnel responsible for bridge inspections. The bridge inspector is not required to obtain the data for all the items during every inspection of a bridge. Once a bridge has been inventoried, the majority of the SI&A items will remain unchanged. The inspector should spot check to see if inventoried items are consistent with findings from the bridge site.

4.1.3 Inventory Items

Inventory items pertain to a bridge's characteristics. For the most part, these items are permanent characteristics, which only change when the bridge is altered in some way, such as reconstruction or load restriction. Inventory items are grouped as follows:

- Identification - identifies the structure using location codes and descriptions.
- Structure Type and Material - categorizes the structure based on the material, design and construction, the number of spans, and wearing surface.
- Age and Service - information showing when the structure was constructed or reconstructed, features the structure carries and crosses, and traffic information.
- Geometric Data - includes pertinent structural dimensions.
- Navigation Data - identifies the existence of navigation control, pier protection, and waterway clearance measurements.
- Classification - classification of the structure and the facility carried by the structure are identified.

- Load Rating and Posting - identifies the load capacity of the bridge and the current posting status.
- Proposed Improvements - items for work proposed and estimated costs for all bridges eligible for funding from the Highway Bridge Replacement and Rehabilitation Program, and other structures the highway agency chooses to include.
- Inspection - includes latest inspection dates, designated frequency, and critical features requiring special inspections or special emphasis during inspection.

All inventory items are explained in the *Coding Guide*. Although inventory items are usually provided from previous reports, the inspector must be able to verify and update the inventory data needed. See Topic 4.2 for condition and appraisal rating items.

4.1.4 Appraisal Items

Appraisal items are a judgment of a bridge component condition in comparison to current standards. Appraisal items are used to evaluate the structure based on the level of service it provides on the highway system. Appraisal rating items include the following SI&A items:

- Structural Evaluation – Overall condition of the structure based on all major deficiencies, and its ability to carry loads.
- Deck Geometry – Evaluates the curb-to-cub bridge roadway width and the minimum vertical clearance over the bridge roadway.
- Under-clearances, Vertical and Horizontal – The vertical and horizontal under-clearances from the through roadway under the structure to the superstructure or substructure units.
- Waterway Adequacy – Appraises waterway opening with respect to passage of flow under the bridge.
- Approach Roadway Alignment – Comparing the alignment of the bridge approaches to the general highway alignment of the section of highway that the structure is on.
- Traffic Safety Features – Record information on bridge railings, transitions, approach guiderail, approach guiderail ends, so that evaluation of their adequacy can be made.
- Scour Critical Bridges – Identify the current status of the bridge regarding its vulnerability to scour.

4.1.5 The Role of Inventory Items in Bridge Management Systems

Inventory items are an important part of an owner's Bridge Management System (BMS). Bridge owners use the inventory items to help plan inspection, maintenance, and reconstruction of their bridges, as well as sort their bridges. There have been times when there has been a problem on a particular bridge and the owners used the inventory items of that bridge to search for the same potential problems that might exist on other bridges.

Table of Contents

Section 4
Bridge Inspection
Reporting System

This page intentionally left blank

Topic 4.2 Condition and Appraisal

4.2.1 Introduction

The reported condition of an element or component is an evaluation of its current physical state compared to what it was on the day it was built. The condition rating is not influenced by the ability of the element or component to carry legal loads. Appraisal rating items are used to evaluate a bridge in relation to the level of service it provides on the highway system of which it is a part. Condition rating is based on a comparison with the bridge's condition when it was built, whereas appraisal rating is based on a comparison with current service standards.

4.2.2 Condition Rating Items

Bridge Components and Elements

Accurate assignment of condition ratings is dependent upon the bridge inspector's ability to identify the bridge components and their elements. Bridge components are the major parts comprising a bridge including the deck, superstructure, substructure, channel and channel protection, and culverts. Bridge elements are individual members comprised of basic shapes and materials connected together to form bridge components.

The overall condition rating of bridge components is directly related to the physical deficiencies of bridge elements.

Evaluating Elements

The inspector should evaluate each element of a each component and assign to it a descriptive condition rating of "good," "fair," or "poor," based on the physical deficiencies found on the individual element. The following guidelines should be used in establishing an element's condition rating:

- Good - element is limited to only minor problems.
- Fair - structural capacity of element is not affected by minor deterioration, section loss, spalling, cracking, or other deficiency.
- Poor - structural capacity of element is affected or jeopardized by advanced deterioration, section loss, spalling, cracking, or other deficiency.

To ensure a comprehensive inspection and as a part of the requirements of record keeping and documentation, an inspector should record the location, type, size, quantity, and severity of deterioration and deficiencies for each element of a given component.

Evaluating Components

The following Structure Inventory and Appraisal (SI&A) items receive an overall condition rating:

- Item No. 58 – Deck
- Item No. 59 – Superstructure
- Item No. 60 – Substructure
- Item No. 61 – Channel and Channel Protection
- Item No. 62 – Culverts

Items 58 through 60 are major components of bridges. Item 62 and the inspection of culverts is discussed in Topics 7.12, 12.3, and 12.4. Item 61 is used only for structures over waterways.

Condition Rating Guidelines

Numerical condition ratings should characterize the general condition of the entire component being rated. They should not attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition rating must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated. Condition ratings assigned to elements of a component must be combined to establish the overall component condition rating.

If the bridge has multiple spans, the inspector must evaluate all elements both quantitatively and qualitatively. However, in some cases, a deficiency will occur on a single element or in a single location. If that one deficiency reduces the load carrying capacity or serviceability of the component, the element can be considered a "weak link" in the structure, and the rating of the component must be reduced accordingly.

The following general condition rating guidelines (obtained from the 1995 edition of the *Coding Guide*) are to be used in the evaluation of the deck, superstructure, and substructure:

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling, or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	“IMMINENT” 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 bridge back in light service.
0	FAILED CONDITION - out of service; beyond corrective action.

The condition rating guidelines presented above are general in nature and can be applied to all bridge components and material types. Additional component specific condition rating guidelines are provided for Item 61, Channel and Channel Protection, and for Item 62, Culverts. (These component specific guidelines are shown below.) Rate and code the condition for Item 61 and Item 62 using the specific condition rating guidelines in accordance with the previously noted general condition rating guidelines.

Item 61 – Channel and Channel Protection

This item describes the physical conditions associated with the flow of water through the bridge such as stream stability and the condition of the channel, riprap, slope protection, or stream control devices, including spur dikes. The inspector should be particularly concerned with visible signs of excessive water velocity which may cause undermining of slope protection, erosion of banks, and realignment of the stream. Accumulation of drift and debris on the superstructure and substructure should be noted on the inspection form but not included in the condition rating.

Rate and code the condition in accordance with the previously described general condition ratings and the following descriptive codes:

Code	Description
N	Not applicable. Use when bridge is not over a waterway (channel).
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.
7	Bank protection is in need of minor repairs. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor streambed movement evident. Debris is restricting the channel slightly.
5	Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the channel.
3	Bank protection has failed. River control devices have been destroyed. Streambed aggradation, degradation, or lateral movement has changed the channel to now threaten the bridge and/or approach roadway.
2	The channel has changed to the extent the bridge is near a state of collapse.

1 Bridge closed because of channel failure. Corrective action may put bridge back in light service.

0 Bridge closed because of channel failure. Replacement necessary.

Item 62 - Culverts

This item evaluates the alignment, settlement, joints, structural condition, scour, and other items associated with culverts. The rating code is intended to be an overall condition evaluation of the culvert. Integral wingwalls to the first construction or expansion joint should be included in the evaluation.

Item 58 – Deck, Item 59 – Superstructure, and Item 60 – Substructure should be coded N for all culverts.

Rate and code the condition in accordance with the previously described general condition ratings and the following descriptive codes:

Code Description

N Not applicable. Use if structure is not a culvert.

9 No deficiencies.

8 No noticeable or noteworthy deficiencies which affect the condition of the culvert. Insignificant scrape marks caused by drift.

7 Shrinkage cracks, light scaling, and insignificant spalling which does not expose reinforcing steel. Insignificant damage caused by drift with no misalignment and not requiring corrective action. Some minor scouring has occurred near curtain walls, wingwalls, or pipes. Metal culverts have a smooth symmetrical curvature with superficial corrosion and no pitting.

6 Deterioration or initial disintegration, minor chloride contamination, cracking with some leaching, or spalls on concrete or masonry walls and slabs. Local minor scouring at curtain walls, wingwalls, or pipes. Metal culverts have a smooth curvature, non-symmetrical shape, significant corrosion, or moderate pitting.

5 Moderate to major deterioration or disintegration, extensive cracking and leaching, or spalls on concrete or masonry walls and slabs. Minor settlement or misalignment. Noticeable scouring or erosion at curtain walls, wingwalls, or pipes. Metal culverts have significant distortion and deflection in one section, significant corrosion or deep pitting.

4 Large spalls, heavy scaling, wide cracks, considerable efflorescence, or opened construction joint permitting loss of backfill. Considerable settlement or misalignment. Considerable scouring or erosion at curtain walls, wingwalls, or pipes. Metal culverts have significant distortion and deflection throughout, extensive corrosion or deep pitting.

3 Any condition described in Code 4 but which is excessive in scope.

Severe movement or differential settlement of the segments, or loss of fill. Holes may exist in walls or slabs. Integral wingwalls nearly severed from culvert. Severe scour or erosion at curtain walls, wingwalls, or pipes. Metal culverts have extreme distortion and deflection in one section, extensive corrosion, or deep pitting with scattered perforations.

2 Integral wingwalls collapsed, severe settlement of roadway due to loss of fill. Section of culvert may have failed and can no longer support embankment. Complete undermining at curtain walls and pipes. Corrective action required to maintain traffic. Metal culverts have extreme distortion and deflection throughout with extensive perforations due to corrosion.

1 Bridge closed. Corrective action may put bridge back in light service.

0 Bridge closed. Replacement necessary.

A bridge's load-carrying capacity is not to influence condition ratings. The fact that a bridge was designed for less than current legal loads, and may even be posted, should have no influence upon condition ratings.

Structural capacity is defined as the designed strength of the member. However, structural capacity is different than load-carrying capacity. Load-carrying capacity refers to the ability of the member to carry the legal loads of the highway system of which the bridge is a part. Therefore, a bridge could possibly have good structural capacity yet be load posted because it is unable to carry the legal loads.

The load-carrying capacity of a bridge is reflected in the Structural Evaluation appraisal rating. A bridge's structural capacity is reflected in the condition ratings of the bridge components. Component ratings are determined by applying condition descriptions, which are general in nature, covering a broad array of bridge components and material types. The inspector must be familiar with terminology concerning material types and associated deterioration to utilize condition descriptions for accurately assigning condition ratings. The following illustrates several common deterioration terms found in condition descriptions and their associated material types:

- Section loss - usually applies to steel members or reinforcing steel
- Fatigue crack - applies to steel members
- Cracking/spalling - usually are used to describe concrete
- Shear crack - usually applies to concrete but may apply to timber as well
- Checks/splits - applies to timber members
- Scour - can apply to substructure or channels

Establishing a link between material type and deterioration allows for accurate component ratings determined by utilizing condition descriptions for ratings 9 through 1 found in the general condition rating guidelines.

Supplemental rating guidelines, which may be developed by individual states, are intended to be used in addition to the *Coding Guide* to make it easier for the inspector to assign the most appropriate condition rating to the component being considered and improve uniformity.

Using the material and component specific supplemental rating guidelines (found in the 1995 edition of the *Coding Guide*) helps to clarify how each type of defect affects the condition rating. Care must be taken not to "pigeonhole" the rating based on only one word or phrase. The following is one suggested method for determining proper condition ratings:

- Identify phrases that describe the component
- Read through the rating scale until encountering phrases that describe conditions that are more severe than what actually exists
- Be sure to read down the ratings list far enough
- Correct rating number then is one number higher

This procedure should generally work with all of the condition rating guidelines.

Condition State Assessment

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the bridge element. 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 specifically describe deck cracking (top and underside), fatigue cracking, pack rust, settlement, and scour. Refer to Topic 4.6 for a more detailed explanation of the Element Level Bridge Management System.

4.2.3 Appraisal Rating Items

Appraisal Rating Guidelines

The following SI&A items are known as appraisal rating items:

- Item No. 67 – Structural Evaluation
- Item No. 68 – Deck Geometry
- Item No. 69 – Underclearances, Vertical and Horizontal
- Item No. 71 – Waterway Adequacy
- Item No. 72 – Approach Roadway Alignment

Appraisal rating items are used to evaluate a bridge in relation to the level of service it provides on the highway system of which it is a part. The level of service for a bridge describes the function the bridge provides for the highway system carried by the bridge. The structure should be compared to a new one that is built to current standards for that particular class of road. The exception is Item 72, Approach Roadway Alignment. Rather than comparing the alignment to current standards, it is compared to the general existing alignment of the section of roadway that the highway is on.

The level of service goals used to appraise bridge adequacy vary depending on the highway functional classification, traffic volume, and other factors. The goals are set with the recognition that widely varying traffic needs exist throughout highway systems. Many bridges on local roads can adequately serve traffic needs with lower load capacity and geometric standards than would be necessary for bridges on heavily traveled main highways.

If national uniformity and consistency are to be achieved, similar structure, roadway, and vehicle characteristics must be evaluated using identical standards. Therefore, tables and charts have been developed which must be used to evaluate the appraisal rating items for all bridges submitted to the National Bridge Inventory, regardless of individual State criteria used to evaluate bridges.

The following general appraisal rating guidelines (obtained from the 1995 edition of the *Coding Guide*) are used to evaluate structural evaluation, deck geometry, underclearances, waterway adequacy, and approach roadway alignment.

Code	Description
N	Not applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Meets minimum tolerable limits to be left in place as is
3	Basically intolerable, requiring high priority of corrective action
2	Basically intolerable, requiring high priority of replacement
1	This value of rating code not used
0	Bridge closed

The specific tables for Items 67 through 69, 71 and 72 appear in the *Coding Guide* and are detailed enough that several states now program their computerized bridge management system to automatically calculate several of the appraisal rating items. Thus, some inspectors may not be responsible for coding these items. Inspectors may be asked to field verify the computed appraisal ratings.

Item 67 - Structural Evaluation - The item description and procedures used to determine the Structural Evaluation Appraisal Rating are located in Item 67 of the *Coding Guide*. The correct way to evaluate this item for bridges is to consider the following factors:

- The lowest rating dictated by Item 59 - Superstructure, Item 60 - Substructure or Comparison of Item 29 - ADT and Item 66 - Inventory Rating.
- For culverts, the lower of Item 62 - Culverts or Comparison of Item 29 - ADT and Item 66 - Inventory Rating.

Item 68 - Deck Geometry - The deck geometry appraisal evaluates the curb to curb bridge roadway width and the minimum vertical clearance over the bridge roadway. This item is coded by determining two appraisal ratings, one for bridge roadway width and one for the minimum vertical clearance. The lower of these two is the appraisal rating. The *Coding Guide* includes the following scenarios to choose from for the bridge roadway width appraisal:

- Bridges with two lanes carrying two-way traffic.
- Bridges with one lane carrying two-way traffic.

- All other two-way traffic situations.
- Bridges with one-way traffic.

Item 69 - Underclearances, Vertical and Horizontal - This item refers to the vertical and horizontal underclearances from the through roadway under the structure to the superstructure or substructure units. The item description and coding guidelines, which are located in Item 69 of the *Coding Guide*, are used to determine the Underclearance Appraisal Rating. This item is similar to Item 68 in that two different ratings are developed: one for vertical underclearance and one for horizontal underclearance. The lower of these two is the appraisal rating.

Item 71 - Waterway Adequacy - Waterway adequacy is appraised with respect to passage of flow through the bridge. The rating is tied to flood frequencies and traffic delays. Appraisal ratings are assigned by the table contained in Item 71 of the *Coding Guide* and are based on the functional classification of the road carried by the structure, hydraulic and traffic data for the structure, and site conditions.

Item 72 - Approach Roadway Alignment – This appraisal is based on comparing the alignment of the bridge approaches to the general highway alignment of the section of roadway on which the structure is located. The rating guidelines are correctly applied by determining if the vertical or horizontal curvature of the bridge approaches differs from the section of highway the bridge is on, resulting in a reduction of vehicle operating speed to cross the bridge. The guidelines for FHWA Item 72, Appraisal or Approach Roadway Alignment, are as follows:

- If no reduction in the operating speed of a vehicle is required compared to the highway, code Item 72 as an "8."
- If only a very minor reduction in the operating speed of a vehicle is required compared to the highway, code Item 72 as a "6."
- If a substantial reduction in the operating speed of a vehicle is required compared to the highway, code Item 72 as a "3."

The following guidelines indicate a means of determining the difference between a minor reduction and substantial reduction of operating speed:

- Minor reduction in operating speed - ≤ 9 mph
- Substantial reduction in operating speed - ≥ 10 mph

The remaining codes between these general values should be applied at the inspector's discretion.

A narrow bridge does not affect the Approach Roadway Alignment Appraisal. The narrow bridge would be accounted for in Item 68, Deck Geometry.

Items affecting sight distance at the bridge, unrelated to vertical and horizontal curvature of the roadway, such as vegetation growth and substructure units of overpass structures do not affect the Approach Roadway Alignment Appraisal.

Item 36 - Traffic Safety Features - For structures on the National Highway System (NHS), this appraisal is based on comparing the traffic safety features in place at the bridge site to current national standards set by regulation, so that an evaluation of their adequacy can be made. For structures not on the National Highway

System (NHS), the procedure is the same, however, it shall be the responsibility of the highway agency (state, county, local, or federal) to set standards. The item description and procedures used to determine the Traffic Safety Feature Appraisal Rating are located in Item 36 of the *Coding Guide*. The following are the traffic safety features to be coded:

- Bridge Railings
- Transitions
- Approach Guiderail
- Approach Guiderail Ends

Item 113 - Scour Critical Bridges – This item is used to identify the current status of the bridge regarding its vulnerability to scour. A scour critical bridge is one with abutment or pier foundations that are rated as unstable due to observed scour at the bridge site, or a scour potential as determined form a scour evaluation study including a scour analysis made by hydraulic, geotechnical, or structural engineers. The item description, procedures, and code descriptions are located in Item 113 of the *Coding Guide.*

4.2.4 Maintenance Rating Guidelines

It is usually necessary to evaluate the condition of more items than those rated on the SI&A forms, because the SI&A condition items cover such broad components. For example, SI&A Item 62 covers all structural components of a culvert. Additionally, the SI&A numerical rating system is not well suited for evaluating minor items. Minor items are essentially limited to ratings of "N", "9", "8", or "7" since the other rating numbers imply a significant impact on the overall integrity or safety of the structure. Therefore, a modified rating system should be used for rating the condition of items added to supplement the SI&A items. Since items are added primarily to identify potential maintenance problems, the modified rating scale should be oriented toward maintenance.

A sample maintenance rating system is shown in Table 4.2.1. The rating system shown provides a numerical scale that is related to the urgency of maintenance action required, as well as the action which should be taken by the inspector.

It is important to note that the inspector basically has three courses of action, depending on the severity of conditions found. Each of these actions involves noting the condition of the components in the inspection report. When no immediate maintenance actions are required, the note in the report is all that is necessary. When a high priority should be assigned for correcting problems found during the inspection, some type of special notification to maintenance personnel is recommended. When immediate action is required to address a hazardous situation or preserve the integrity of the structure, maintenance personnel should be notified on an emergency basis.

Care must be exercised when using different rating systems, particularly when combining the ratings given to supplemental items to arrive at ratings for SI&A items. SI&A item ratings usually represent a composite rating of a group or broad category of supplemental items. The SI&A ratings should not merely be an average of the ratings assigned to the supplemental items but should be based on the inspector's judgement. A low rating in one supplemental item will usually control the composite rating.

Maintenance Urgency Index	Maintenance Immediacy of Action	Inspection Course of Action
9	No repairs needed.	
8	No repairs needed. List specific items for special inspection during next regular inspection.	
7	No immediate plans for repair. Examine possibility of increased level of inspection.	Note in inspection report only.
6	Repair by end of next season – add to scheduled work.	
5	Place in current schedule – current season, first reasonable opportunity.	
4	Priority – current season, review work plan for relative priority, adjust schedule if possible.	Special notification to superior is warranted.
3	High priority – current season, as soon as can be scheduled.	
2	Highest priority – discontinue other work if required, emergency basis or emergency subsidiary actions if needed (post, one-lane traffic, no trucks, reduced speed, etc.).	Verbally notify superiors immediately and confirm in writing.
1	Emergency actions required – reroute traffic and close.	
0	Facility is closed for repairs.	

Table 4.2.1 Maintenance Rating Scale

4.2.5 Overall Culvert Ratings

General

Topics 7.12, 12.3, and 12.4 address the individual components of various culverts. Overall ratings consider all of the components which make up a culvert and are useful in establishing maintenance, rehabilitation, and replacement programs and priorities.

Some of a culvert's individual components are not rated in the SI&A sheet. However, they are useful supplemental items in defining the condition and in determining the overall ratings. The SI&A sheet has several items that require evaluation of the culvert as a whole. The SI&A items can be divided into three categories: overall condition, load-carrying capacity, and remaining life.

Overall Condition

Two items on the SI&A sheet pertain to the overall condition of culverts. Item 62, Culverts, covers the condition of the culvert's structural and hydraulic components (alignment, settlement, culvert barrel, end treatment, and embankment). Item 67, Structural Evaluation, covers the evaluation of the structural components and the load-carrying capacity.

Overall ratings must not be an average of the ratings assigned to individual components. Very often a low rating for one component will control the overall rating, but when assigning an overall rating, the inspector should consider each component and its possible effect on the culvert. The inspector should consider whether the component is functioning properly, whether it could pose a threat to safety or cause property damage, whether it could cause more extensive damage if not repaired, and whether the repairs represent rehabilitation or maintenance.

Load-carrying Capacity

SI&A Items 64, 66, and 70 are based on the loads which the structure can carry. Item 64, Operating Rating, is the maximum load the structure can carry. Item 66, Inventory Rating, is the load which can be carried repeatedly for an indefinite period of time. Item 70, Bridge Posting, is a rating based on an evaluation of the culvert's load-carrying capacity and the state's legal load limits. The procedures used for determination of these capacity ratings should take into account the condition of the culvert at the time of the inspection.

Remaining Life

The inspector estimates the number of years that remain before major rehabilitation or replacement of the culvert is required. The estimate should be based on the design life of the barrel material, the years of service prior to the inspection, and the condition of the culvert at the time of the inspection. The current condition and the performance of the culvert material under similar conditions are the key considerations. Where durability is a problem, electrical resistivity and pH measurements of the surrounding soil and the stream may be helpful in estimating the remaining life.

4.2.6 Sufficiency Rating

Definition

Sufficiency rating (S.R.) is a calculated numeric value used to indicate the sufficiency of a bridge to remain in service. The rating is calculated using the sufficiency rating formula. Sufficiency rating is discussed in detail in Appendix B of the *Coding Guide*.

Sufficiency Rating Formula

$S.R. = S_1 + S_2 + S_3 - S_4$

0% (entirely deficient)	$\leq$	S.R.	$\leq$	100% (entirely sufficient)

where: S_1 = 55% max.; based on structural adequacy and safety (i.e., superstructure or substructure condition and load capacity).

S_2 = 30% max.; deals with serviceability and functional obsolescence (items such as deck condition, clearances, roadway alignment and width, etc.).

S_3 = 15% max.; concerns essentiality for public use (items such as detour length, average daily traffic, and defense highway designation).

S_4 = 13% max.; deals with special reductions based on detour length, traffic safety features, and structure type.

Eighteen SI&A sheet items are used to calculate these four factors which therefore determine the sufficiency rating. Sufficiency rating is not normally calculated manually. Usually, it is included in the agency's inventory computer program and is calculated automatically by the computer based upon the inventory data collected by the bridge inspector.

Uses

Sufficiency Rating (SR) is used by the federal and state agencies to determine the relative sufficiencies of all of the nation's bridges. In the recent past, eligibility for federal funding with Highway Bridge Replacement and Rehabilitation Program funds has been determined by the following criteria:

$S.R. \leq 80$	Eligible for rehabilitation
$S.R. < 50$	Eligible for replacement

Some states use the sufficiency rating as the basis for establishing priority for repair or replacement of bridges; the lower the rating, the higher the priority. Several states are developing specific bridge management procedures with priority guidelines for repair or replacement of bridges. By using these types of procedures, priority ratings can be established by considering the significance or impact of such level-of-service parameters as traffic volume and class of highway.

Table of Contents

Section 4
Bridge Inspection
Reporting System

Abbreviations for Field Inspection Notes

Abut. = Abutment
Adj. = Adjacent
B. = Bent
Betw. = Between
Bot. = Bottom
B.S. = Both Sides
[= Channel (Steel Shape)
cm = Centimeter
Col. = Column
Conc. = Concrete
Cond. = Condition
Conn. = Connection
Cr. = Crack
Delam. = Delamination, Delaminated
Deter. = Deterioration
Diag. = Diagonal
Diam. = Diameter
Diaph. = Diaphragm
D.S. = Downstream
E = East
Eff. = Efflorescence
Elev. = Elevation
Expan. = Expansion
F.B. = Floorbeam
F.L. = Full Length
Flg. = Flange
F.S. = Far Side
Ft. = Feet
Gus. = Gusset
H.L. = Hairline
Horz. = Horizontal
Hvy. = Heavy
Int. = Interior
Lac. = Lacing
Lat. = Lateral
Lat. Br. = Lateral Brace
Len. = Length
Low. = Lower
Lt. = Light
M = Meters
Med. = Medium
Mid. = Middle
N = North
No Vis. Def. = No Visible Defects
N.S. = Near Side
P = Pier
Pl. = Plate
S = South
S.I.P. = Stay-in-Place Forms
SF = Square Feet
Stiff. = Stiffener
Str. = Stringer
T. Welds = Tack Welds
Typ. = Typical
U = Upper
U.S. = Upstream
Vert. = Vertical
Vis. = Visible
Vis. S. = Visible Signs
W = West
W = Wide Flange (Steel Shape)
L = Angle (Steel Shape)

Topic 4.3 Record Keeping and Documentation

4.3.1 Introduction

While the inspection of small bridges usually only requires the use of the standard inspection form, the inspection of large or complex bridges requires the use of an inspection notebook, in addition to any standard inspection forms. The inspection notebook should contain:

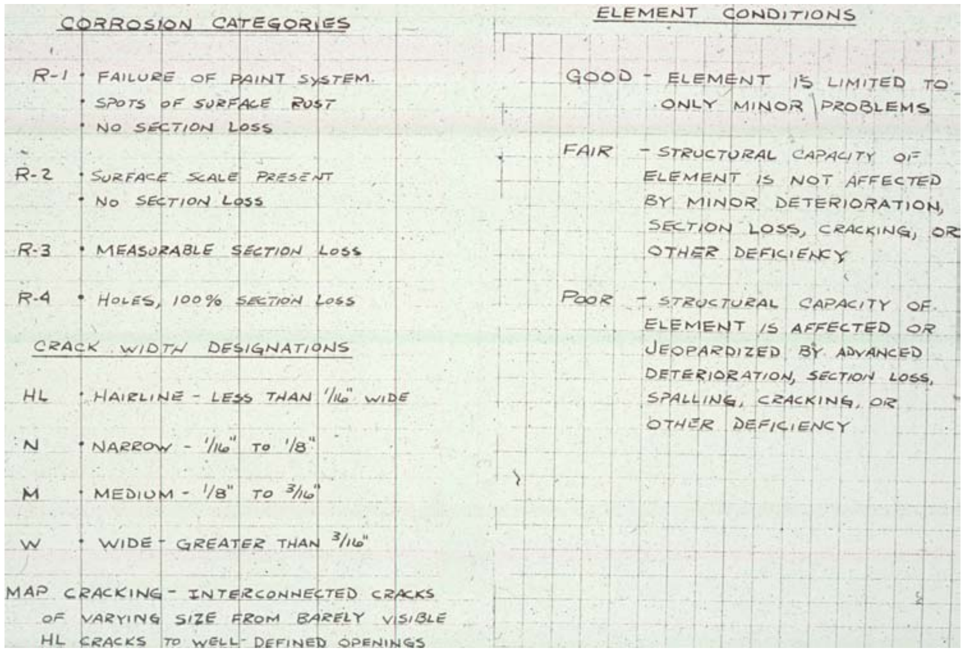
- A standard notation system for indicating the condition of the elements or members
- Sketches of elements or members showing typical and deteriorated conditions (some of these can be pre-made to allow more expediency during the inspection)
- Standard nomenclature and abbreviations for the elements of members and the components made up of these members
- A log or index for photographs
- Brief narrative descriptions of general and component conditions

When the notebook format is selected for recording bridge inspection results, the information should be recorded systematically. However, many state agencies differ significantly in their required format. Most of the above information, if not provided on the inspection report, should be available in the structure file.

4.3.2 Methods of Record Keeping

Traditional

All signs of distress and deterioration should be noted with sufficient precision so that future inspectors can readily make a comparison of conditions. The most commonly used method for record keeping is pencil and paper. The inspector writes findings on forms, sketches, and notebooks (see Figure 4.3.1). This method is extremely flexible in that the inspector can draw whatever configurations are necessary to best describe and document deficiencies. During an inspection, trying to climb and write at the same time is obviously not a safe practice. Therefore, it is a good idea to secure a safe position before attempting to record any findings.



CORROSION CATEGORIES

R-1 • FAILURE OF PAINT SYSTEM.
• SPOTS OF SURFACE RUST
• NO SECTION LOSS

R-2 • SURFACE SCALE PRESENT
• NO SECTION LOSS

R-3 • MEASURABLE SECTION LOSS

R-4 • HOLES, 100% SECTION LOSS

CRACK WIDTH DESIGNATIONS

HL • HAIRLINE - LESS THAN 1/16" WIDE

N • NARROW - 1/16" TO 1/8"

M • MEDIUM - 1/8" TO 3/16"

W • WIDE - GREATER THAN 3/16"

MAP CRACKING - INTERCONNECTED CRACKS OF VARYING SIZE FROM BARELY VISIBLE HL CRACKS TO WELL-DEFINED OPENINGS

ELEMENT CONDITIONS

GOOD - ELEMENT IS LIMITED TO ONLY MINOR PROBLEMS

FAIR - STRUCTURAL CAPACITY OF ELEMENT IS NOT AFFECTED BY MINOR DETERIORATION, SECTION LOSS, CRACKING, OR OTHER DEFICIENCY

POOR - STRUCTURAL CAPACITY OF ELEMENT IS AFFECTED OR JEOPARDIZED BY ADVANCED DETERIORATION, SECTION LOSS, SPALLING, CRACKING, OR OTHER DEFICIENCY

Figure 4.3.1 Sample Notation

Electronic Data Collection

A new method of record keeping is electronic data collection (see Figure 4.3.2). This new technology provides a significant advantage in a number of areas. With all the bridge data available at the site, the inspector can retrieve and edit previous records. This not only saves time but eliminates the need for reentering data. Also, it eliminates errors that can occur when transferring the inspector's field notes to the computer back at the office. Electronic data collection provides a logical and systematic sequence of inspection, ensuring that no bridge elements are overlooked. It also allows the inspector to compare the current deficiencies with previous reports and note if the situation has gotten worse.

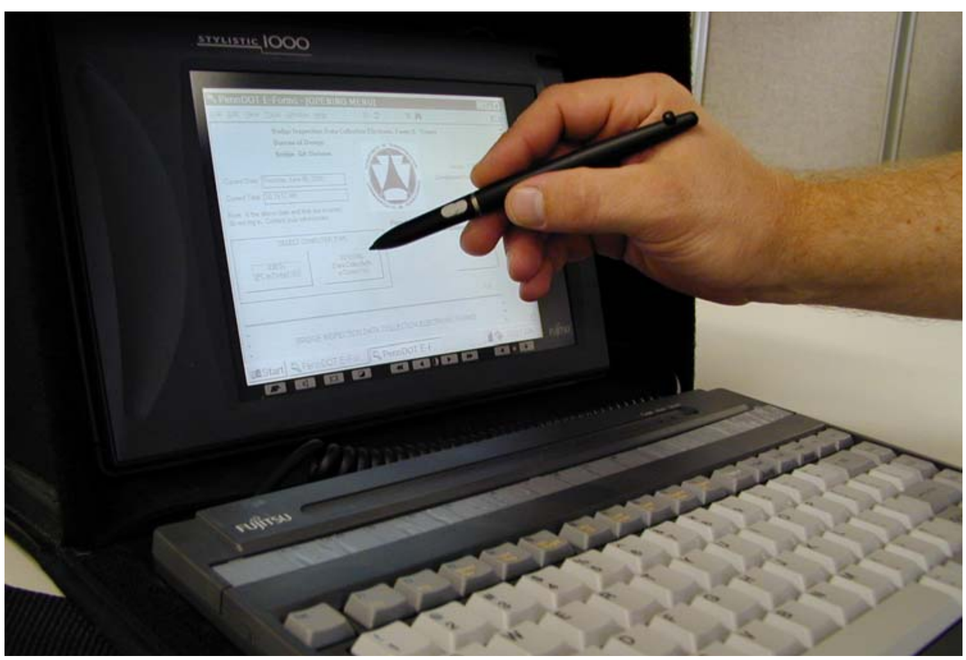


Figure 4.3.2 Electronic Data Collection

4.3.3

Typical Record Setup

A typical field notebook should contain the following:

- Title page
- Table of contents
- Inspection notes and sketches
- Photo log
- Summary of findings
- Inspection forms

Title Page

The front of a title page should contain:

- Name of structure
- Structure Identification Number
- Location
- Features intersected
- District
- County

The back of the title page should contain:

- Date
- Names of inspectors (indicating the team leader)
- Field book number
- Temperature
- Weather conditions

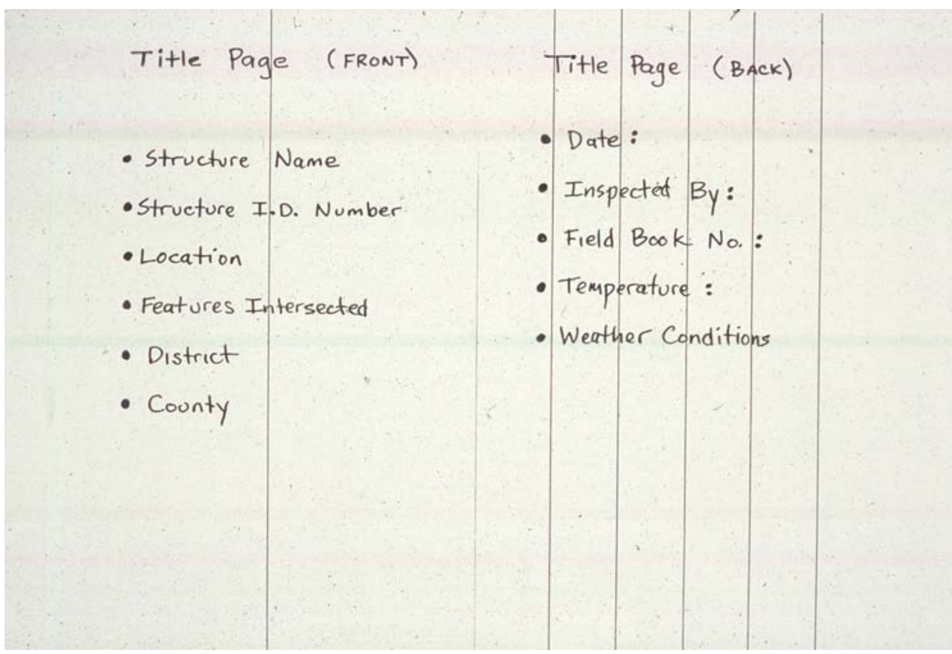


Figure 4.3.3 Sample Notebook Title Page

Table of Contents

The table of contents should be used to provide an outline of the notebook for quick reference. Figure 4.3.4 illustrates an example of a table of contents setup.

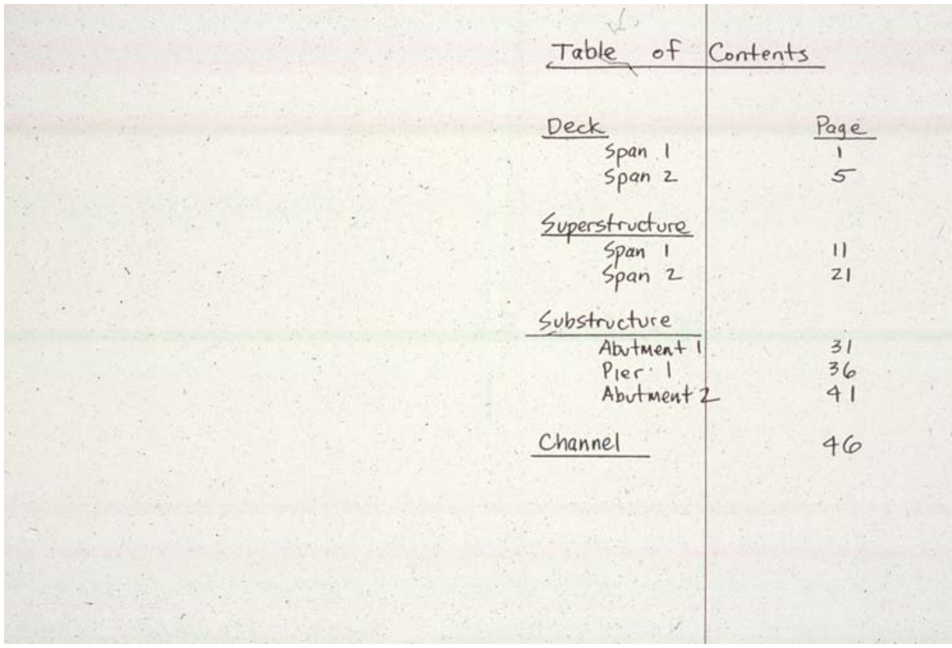


Figure 4.3.4 Sample Notebook Table of Contents

Inspection Notes and Sketches

The left-hand page should contain the element identification, descriptive rating (i.e., good, fair, poor), and comments. The right-hand page should be reserved for sketches or drawings of the elements (see Figure 4.3.5).

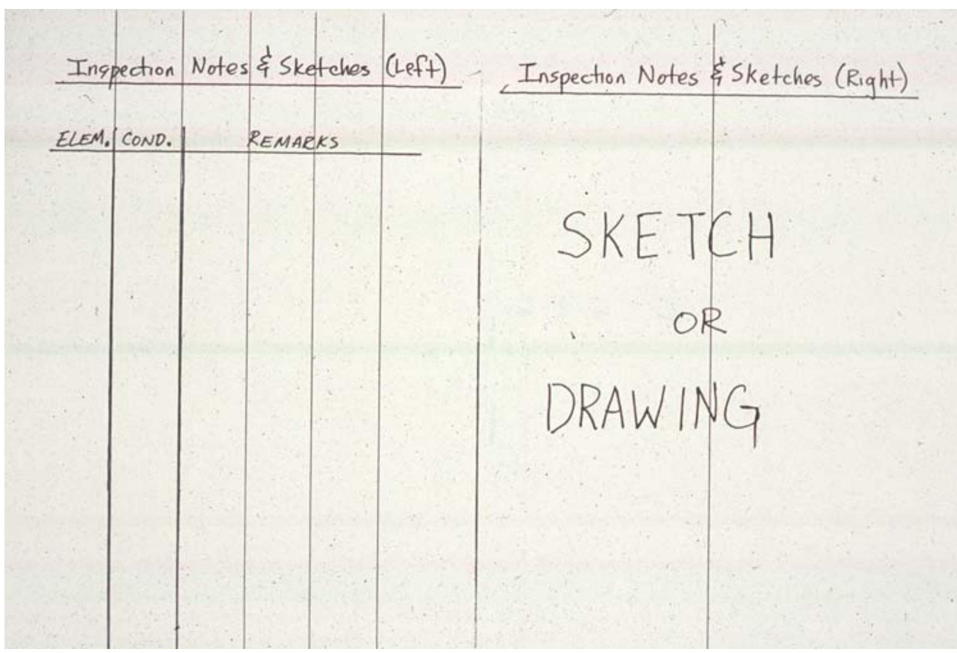


Figure 4.3.5 Sample Notes and Sketches Page

In most cases, it will be possible to insert reproductions of portions of the plans in the notebook. However, in some instances, sketches will have to be drawn. The inspector may be able to pre-draw the sketches in the office and fill them out in the field (see Figures 4.3.6 through 4.3.8).

S.O. No. ______________________________

Subject: ______________________________

______________________________ Sheet No. ______ of ______

______________________________ Drawing No. ____________

Computed by __________ Checked By __________ Date ________________

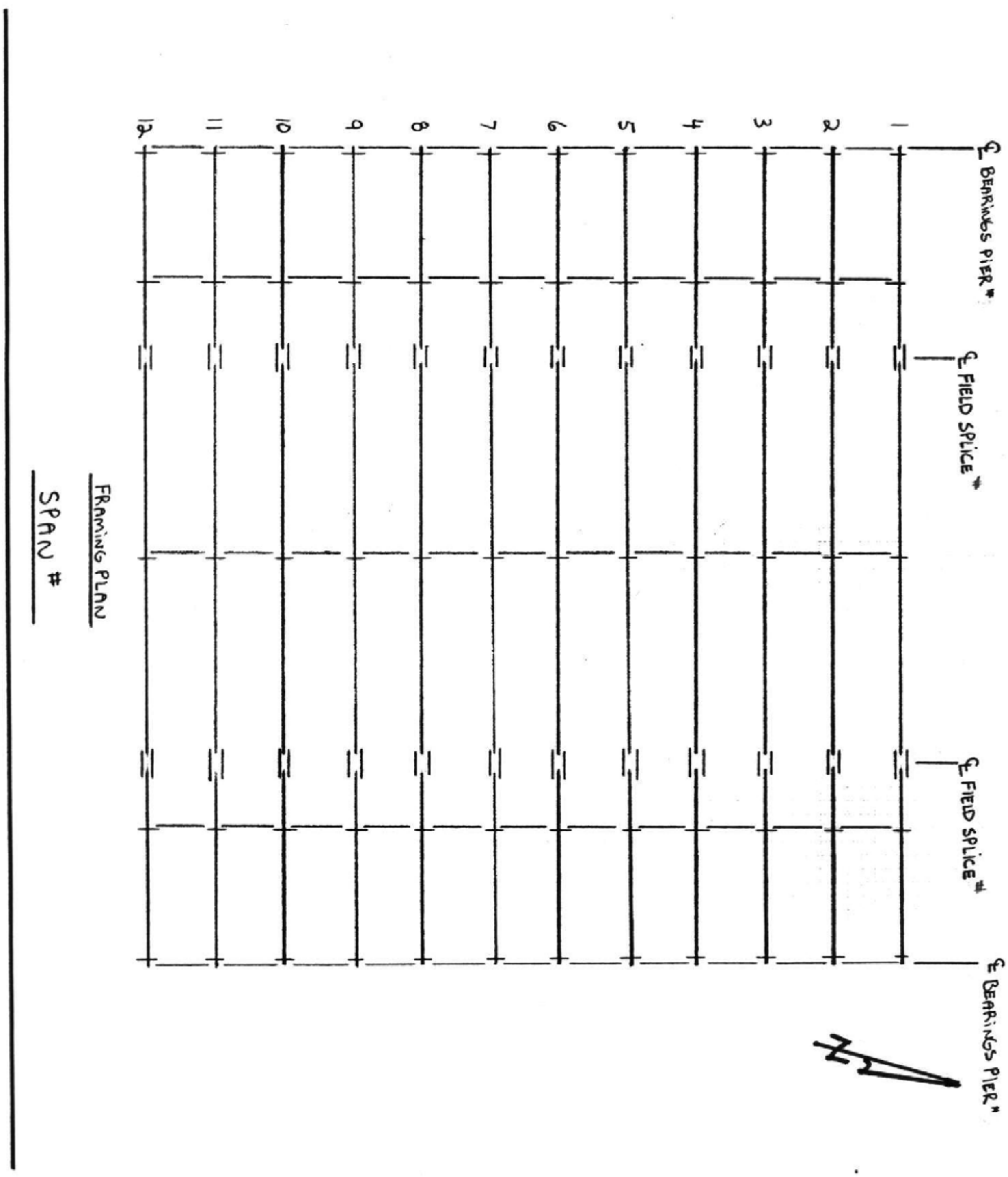


Figure 4.3.6 Framing Plan

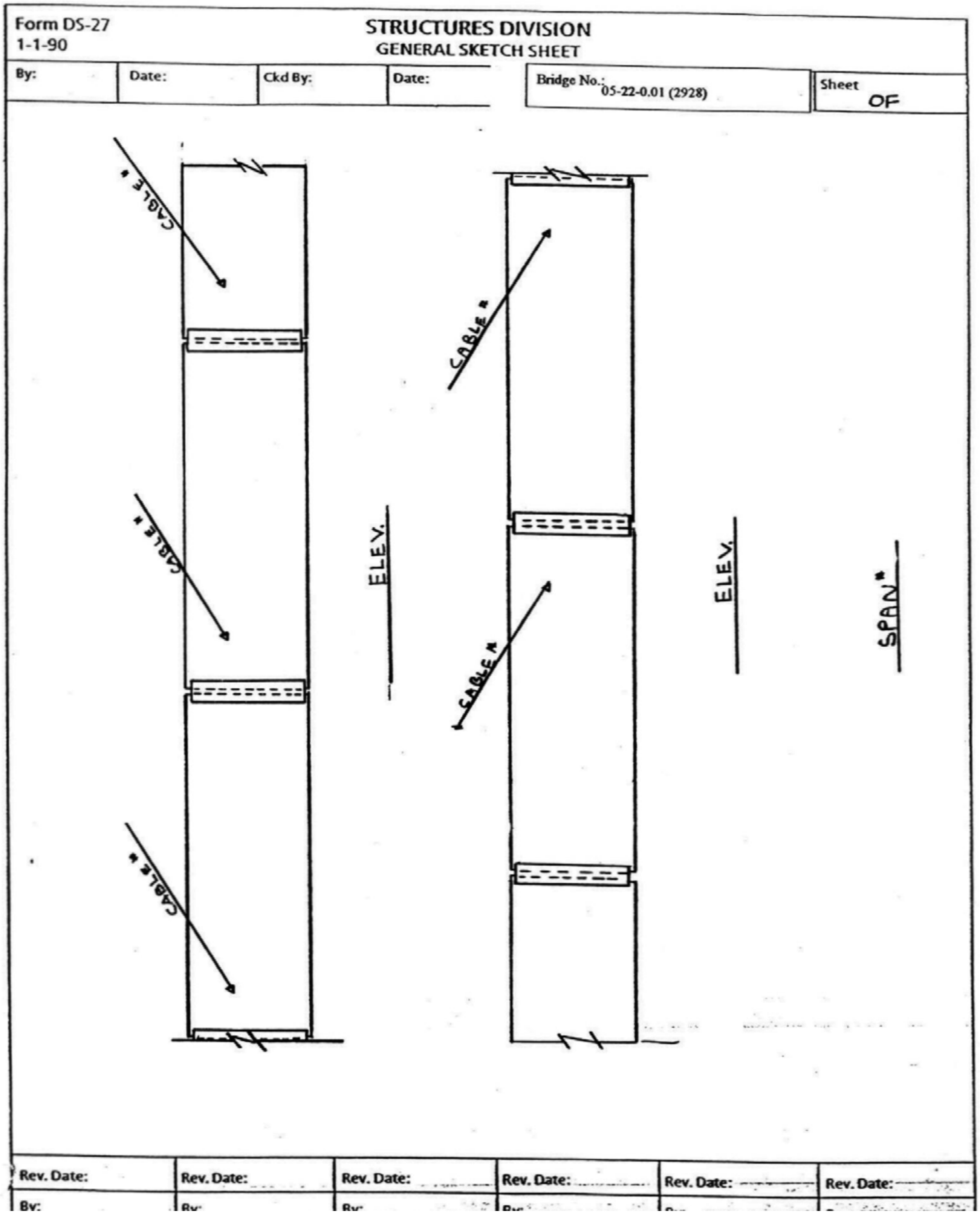


Figure 4.3.7 Girder Elevation

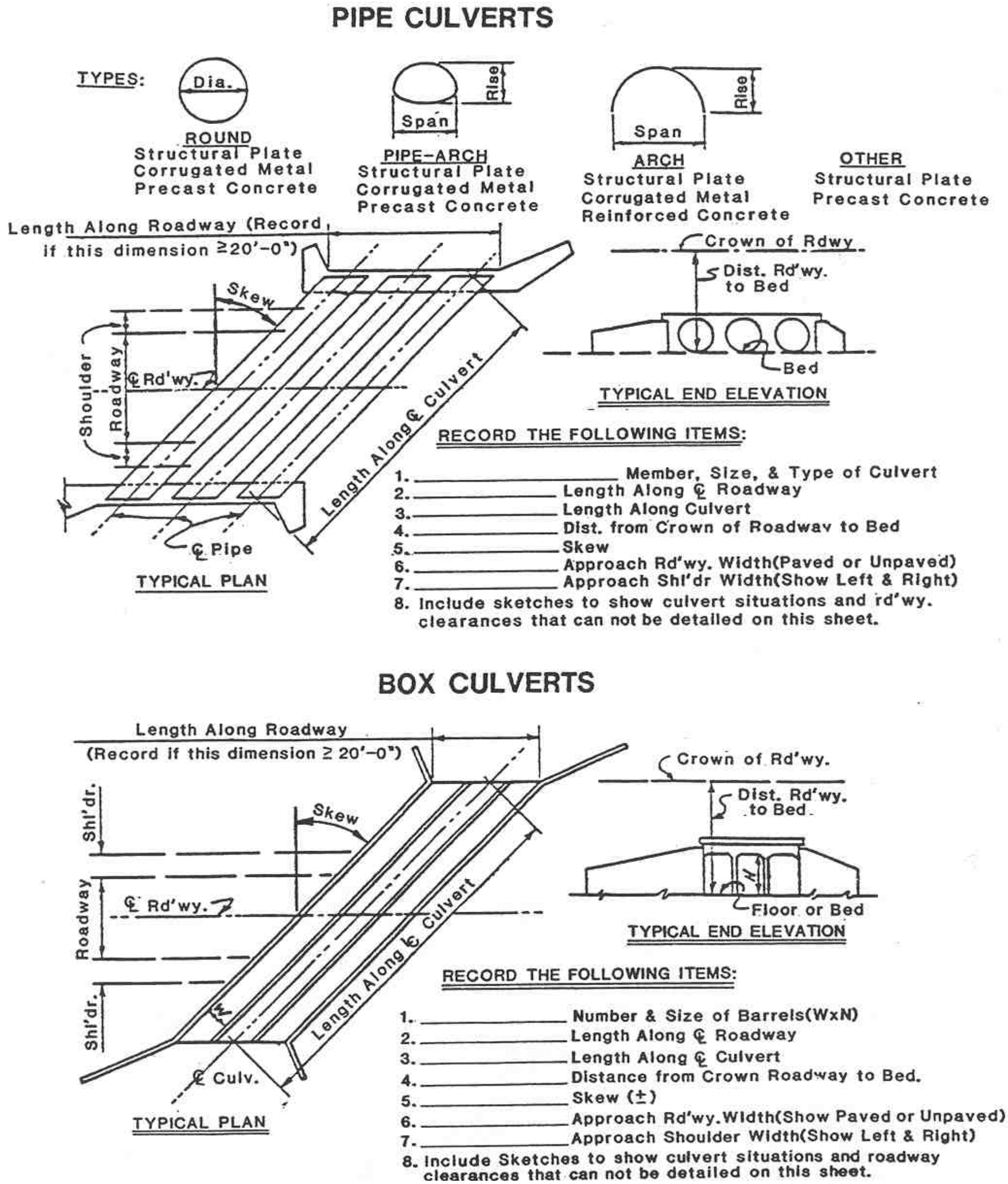


Figure 4.3.8 Typical Prepared Culvert Sketches

The first sketch in the field notebook should schematically portray the general layout of the bridge and site information, illustrating the structure plan and elevation data (see Figures 4.3.9 and 4.3.10). The immediate area, the stream or terrain obstacle layout, major utilities, and any other pertinent details should be included.

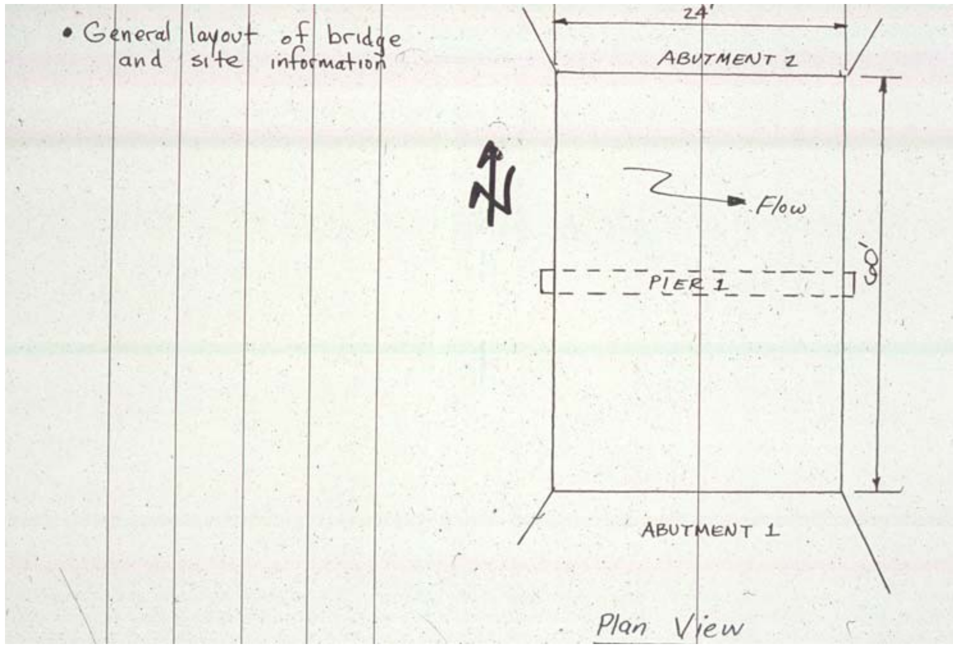


Figure 4.3.9 Sample General Plan Sketch

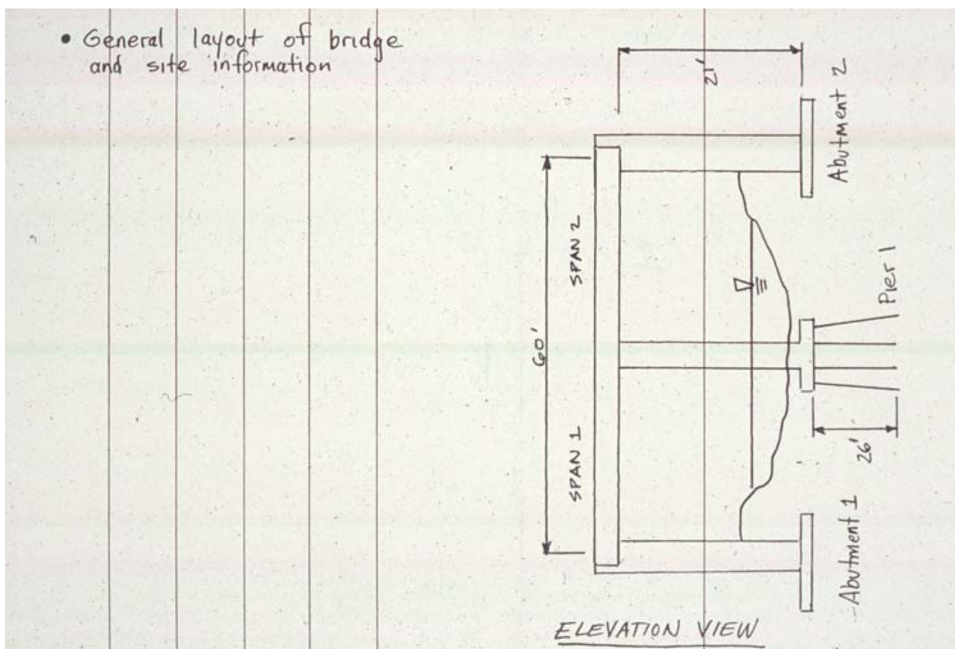


Figure 4.3.10 Sample General Elevation Sketch

Deck sketches should include expansion joints, construction joints, curbs, sidewalks, parapets, and railings (see Figure 4.3.11).

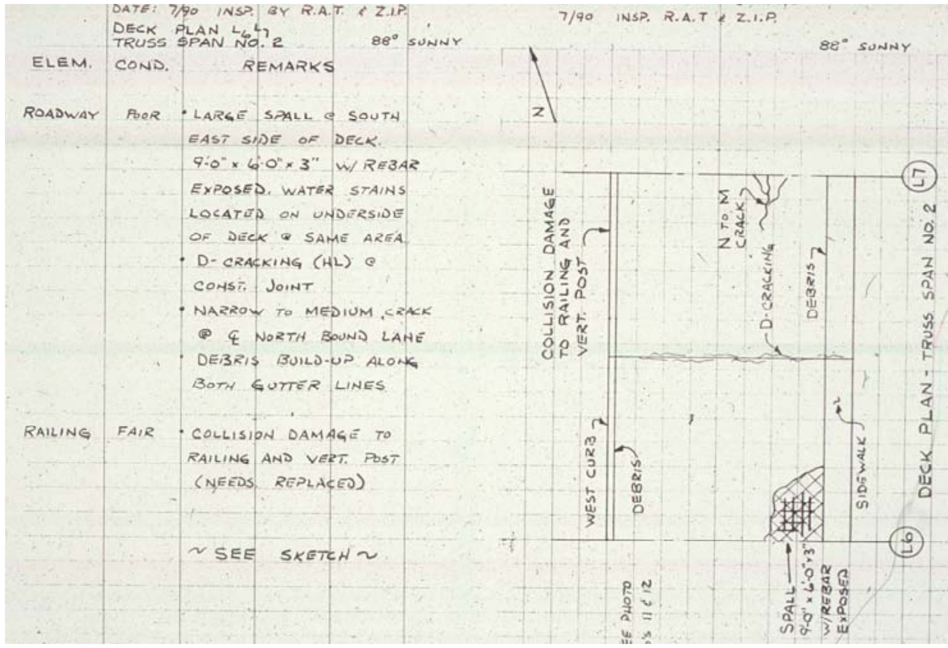


Figure 4.3.11 Sample Deck Inspection Notes

Superstructure units should be sketched both in cross section, plan, and elevation views. Items to be numbered include bearings, main supporting members, floorbeams, stringers, bracing, and diaphragms (see Figure 4.3.12).

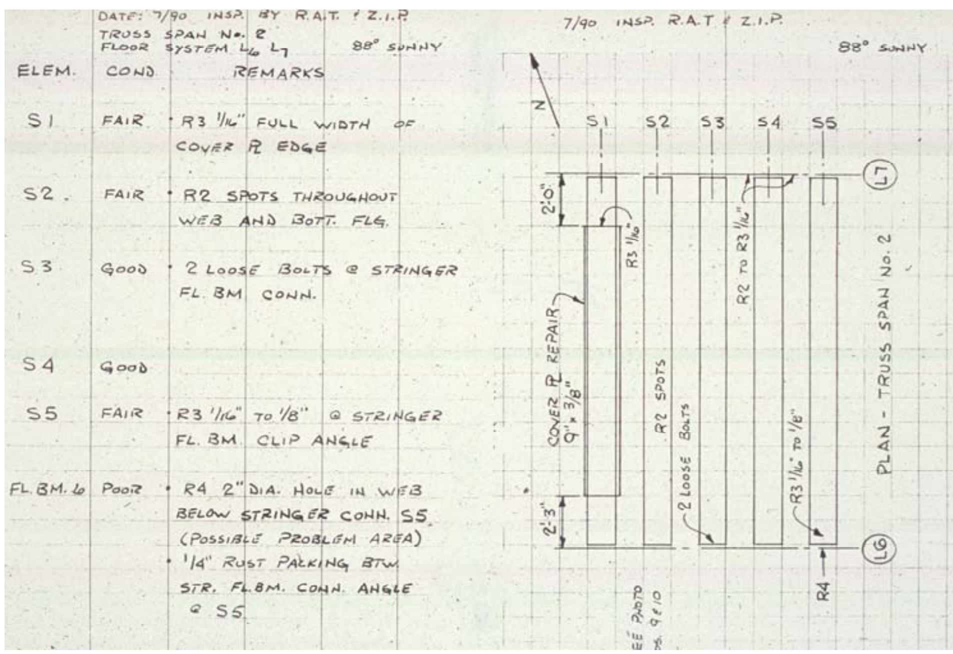


Figure 4.3.12 Sample Superstructure Inspection Notes

Sketches or drawings of each substructure unit should be included (see Figure 4.3.13). In many cases, it is sufficient to draw typical units that identify the principal elements of the substructure. Each of the elements of a substructure unit should be numbered so that they can be cross referenced to the information appearing on the data page on the left-hand side of the sketch. Items to be numbered include piling, footings, vertical supports, lateral bracing of members, and caps.

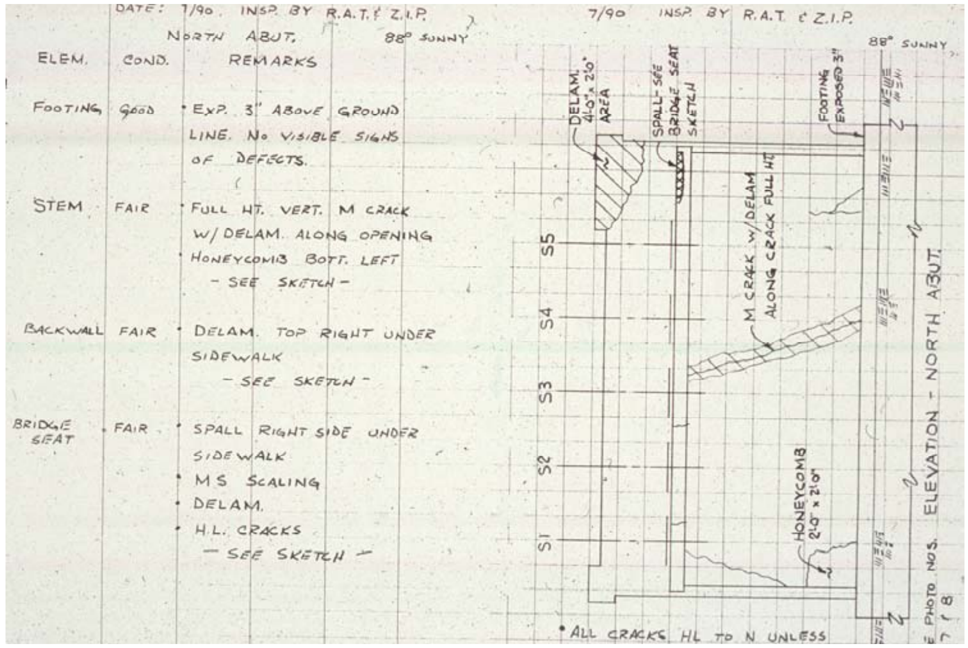


Figure 4.3.13 Sample Substructure Inspection Notes

Photo Log

A photo log should also be kept during the inspection. The photo log should include the date, roll or disk number, photo number, and description of each photograph. It is best to be very specific when describing the photos (see Figure 4.3.14). Descriptions should include both the location of the member and a brief description of any deficiencies.

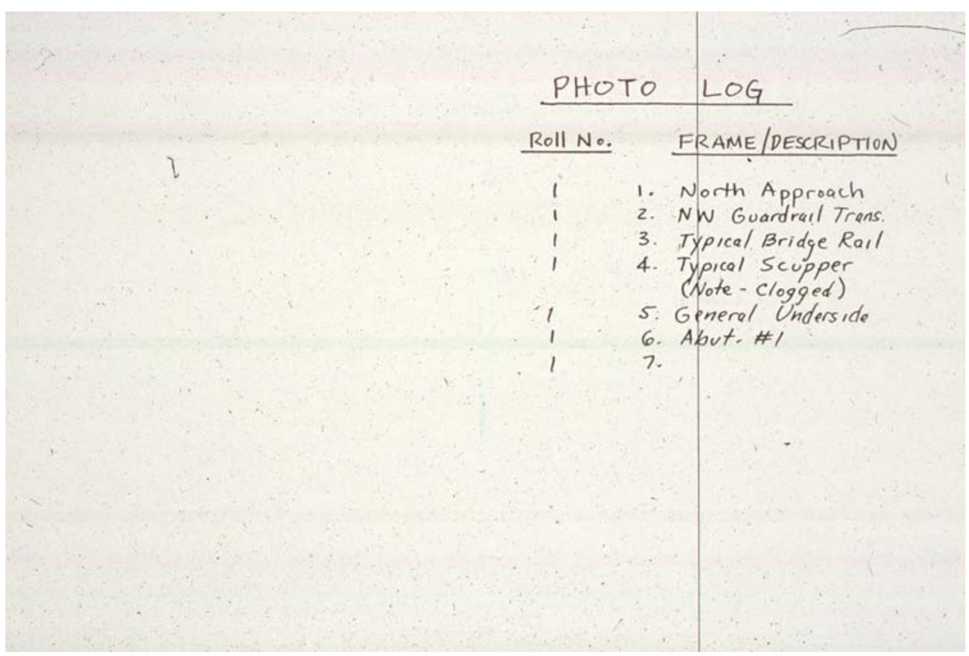


Figure 4.3.14 Sample Photo Log

Summary of Findings

The most important section of any field notebook is the inspector's findings. All deficiencies should be reported, no matter how minor they may seem. The inspector should be as descriptive as necessary to report not only the severity of the defect but the location as well. This will be described in further detail later in this section.

Inspection Forms

Many state agencies have standard inspection forms. These forms are used for each bridge in their system and give the inspector a checklist of items that are to be reviewed. Another benefit is that it organizes all bridge reports into one standard format (see Figures 4.3.15 and 4.3.16).

PDT Form D-450A (DEC 1996) **Site Data** **BRIDGE MANAGEMENT SYSTEM** BRIDGE INSPECTION REPORT BMS Updated b ______ Date ______

A01 ______ C05 Structure Type (Dept.) Main ______

Over ______ Approach ______

Inspection Date E06 ______ Name of Consultant and/or Inspectors E12 ______

Inspection Type E07 ___ Inspected by E08 ___ Hired by E13 ___ Time started ______ Weather Conditions: Temp: ______

Time completed ______

☐ 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	T					
D15	Except Combination	T					
D14	One Truck at a Time	Yes / No					
B22/B23	Vert. Clearance - On						See Sketch
B22/B23	Vert. Clearance - Und						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 ___ Controlling: Lateral ______ Vertical ______

E28-A Traffic Safety Features (Subfields shown vertically) Posted Speed Limit ______ mph

___ Bridge Railing ______

___ Transition ______

___ Approach Guiderail ______

___ Approach Rail Ends ______

E28 Approach Alignment ___ ______

E15 Approach Roadway ___ ______

Pavement ______

Drainage ______

Shoulders ______

E14 Approach Slab ___ ______

Bump at Bridge Yes ☐ No ☐ ______

C19 Relief Joint ___ ______

SITE DATA Sheet______of______

Figure 4.3.15 Example Inspection Form – PADOT Form D-450

PDT Form D-450B (DEC 1996) **Bridge 1 Data**

A01 — Inspection Date E06

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 — Table — Controlling Values: B27 / B34 / B22 — A31 / A31 / B18

Design Exception granted ?

E16 Deck Wearing Surface

C10 Wearing Surface Type — C10A Wearing Surface Thickness

E17 Deck — Estimated Spall or Delamination — % — Est. Chloride Content

Top

Underside

Exp Joint — No. — C22 Exp Jt Types

Deck Drainage

E18 Superstructure — See Sheet — for Additional Details. — Form 491-J attached for FCM details Yes/No

Girders / Beams

Floorbeams

Stringers

Diaphrams

Truss Members

Portals / Bracing

Bearings

Drainage System (Below Deck)

BRIDGE 1 DATA Sheet________of________

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450C
(DEC 1996)

Abutment Data

A01

Inspection Date

E06

E20 Substructure Details on Sheet

NAB - Near Abutment (Use same notation as W09)

Backwall

Bridge Seats

Cheekwalls

Stem

Wings

Footing

Piles

Scour / Undermine Yes No See Details on Form Sheet

Settlement

Embank-Slope-Wall

Wall Drainage

FAB - Far Abutment (Use same notation as W09)

Backwall

Bridge Seats

Cheekwalls

Stem

Wings

Footing

Piles

Scour / Undermine Yes No See Details on Form Sheet

Settlement

Embank-Slope-Wall

Wall Drainage

ABUTMENT DATA Sheet of

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450D (DEC 1996)

Pier Data

A01

Inspection Date

E06

Substructure (Cont.)

Pier / Bent Number (Use same notation as W09)

Bridge Seats

Caps

Cheekwalls

Columns/Stems

Footings

Piles

Scour / Undermine Yes ☐ No ☐ See Details on Form ______ Sheet ______

Settlement

Pier / Bent Number (Use same notation as W09)

Bridge Seats

Caps

Cheekwalls

Columns/Stems

Footings

Piles

Scour / Undermine Yes ☐ No ☐ See Details on Form ______ Sheet ______

Settlement

PIER DATA Sheet______of______

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450E (DEC 1996) **Waterway 1 Data** **BRIDGE MANAGEMENT SYSTEM** BRIDGE INSPECTION REPORT BMS Updated b ________ Date ________

A01 [ ] [ ] [ ] [ ] U.W. Inspection Date W01-A [ ]

________ Over ________ Weather Conditions ________

Inspection Type W02 [ ] 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 [ ]

E29A Scour Critical Rating W06 [ ] based on: [ ] Observed Scour [ ] Scour Calculation No. of Units Inspected W14 [ ]

Streambed Material W07 [ ] [ ] [ ] (36 SPACES)

E21 Channel/Channel Protection - Cond. Rating [ ] Details on Sheet ________

Channel ________

Banks ________

Streambed Movements ________

Debris, Vegetation ________

River (Stream) Control Devices ________

Embankment / Streambed Controls ________

Drift, Other ________

E27 Waterway Adequacy [ ] ________

Risk of Overtopping	[ ] Remote	[ ] Slight	[ ] Occasional	[ ] Frequent
Traffic Delay	[ ] 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
[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]

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: ________

WATERWAY 1 DATA Sheet________of________

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

Form D-450F
(DEC 1996)

Waterway 2 Data

A01

U.W. Inspection Date

W01-A

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:

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:

WATERWAY 2 DATA Sheet________of________

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450G **Waterway 3 Data**

(DEC 1996)

U.W. Inspection Date

A01 | | | | | W01-A |

OBSERVED SCOUR RATING GUIDE

Rating	1 Change Since Last Inspection	2 Scour Hole	3 Debris Potential	4 Substructure Scourability	5 Opening Adequacy/ Channel	6 Sediment	7 Alignment	8 Velocity/ Stream Slope	Rating
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	7/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.

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 Change Since Last Inspection	2 Scour Hole	3 Debris Potential	4 Scourability	5 Opening Adequacy/ Channel	6 Sediment	7 Alignment	8 Velocity/ Stream Slope	W11-A Overall Observed Scour Rating

<u>If Underwater Inspection only</u>

Signatures and Date:

WATERWAY 3 DATA Sheet________of________

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450H **Culvert Data** Inspection Date

(DEC 1996) A01 E06

For Non-State Roadways B01 Ref B27 ADT B28 ADT YR B30-A ADTT %

For State highways, data from RMS will be used.

E25 Deck Geometry Table Controlling Values: B27 / B34 / B22 A31 / A31 / B18

Design Exception granted ?

E16 Dk. Wearing Surf. C10 W. Surface Type C10A W. Surf. Thick.

E22 Culverts Condition Rating Details on Sheet C06 Culvert Length

Top Slab

Barrel

Floor / Paving

Headwall

Wings

Settlement

Debris

Re-calculate IR/OR: Yes Due to: Deterioration New Wearing Surf. Other

No Previous Rating Dated is still valid

E30 Inventory Rating

E31 Operating Rating

H HS ML-80 Other Other HS Load Factor

E32 Rate Meth E33 Typ Mem AASHTO E37 Spec E38 Manual

E29 Bridge Post CONTROLLING: H HS ML80 Engineering Judgement

E24 Structural Condition Appraisal Based upon Table 1 B27-ADT B30-IR

or E18-Super E20-Sub , E22-Culvert ,

E01 Next Insp. Freq. E03 Equip. Next Insp.

E04 Spec. Insp. Type E05 By Date E23 Remaining Life

Is culvert over water? Yes. Complete Forms D-450E through G E17 = N E18 = N E20 = N

No. E21 = N E27 = N E29A = N E17 = N E18 = N E20 = N

Signatures and Date:

CULVERT DATA Sheet of

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450K **Bridge 2 Data**
(DEC 1996) A01 Inspection Date E06

E19 Paint Condition New Paint Y / N If Yes: Spot Zone Full Revise item G08-G17

Interior Beam / Girder

Fascias

Splash Zone: Truss / Girder

Truss

Bearings

Other

E23 Est. Remaining Life BMS to Calculate Yes/No Comments

Recalculate IR/OR: Yes Due to: Deterioration New Wearing Surf. Other
No Previous Rating Dated ______ is still valid

E30 Inventory Rating

E31 Operating Rating

H HS ML-80 Other Other HS Load Factor

E32 Rate Meth E33 Typ Mem AASHTO E37 Spec E38 Manual

E29 Bridge Post CONTROLLING: H ______ HS ______ ML80 ______ Engineering Judgement ______

E24 Structural Condition Appraisal Based upon Table 1 B27-ADT ______ B30-IR ______
or E18-Super ______ E20-Sub ______, E22-Culvert ______,

E01 Next Insp. Freq. E03 Equip. Next Insp.

E04 Spec. Insp. Type E05 By Date

Is bridge over water? Yes. E22 = N Complete Forms D-450E through G
No. E22 = N E21 = N E27 = N E29A = N

Notes:

Signatures and Date:

BRIDGE 2 DATA Sheet______of______

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

PDT Form D-450M **Maintenance Needs Data**

(DEC 1996) A01 | | Inspection Date | E06

H01 H03 H05 H08 H09

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			

H01 H03 H05 H0 H09

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			
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	REPL......	Replace	IMP.............	Improve
N............	Near	UP..........	Upstream	LNR............	Near Left/Right
F............	Far	DN..........	Downstream	LFR.............	Far Left/Right
O............	Other	UN..........	Under	1,2,3, etc......	Span/Pier No.
IN...........	Inlet	OUT	Outlet	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** | F04 **Improvement Length**

F02 **Type Work** | F06 **Bridge Width**

F10 **Future ADT** | F11 **Future ADT Year**

Reviewed On: ______

By: ______

MAINTENANCE NEEDS DATA Sheet________of________

Figure 4.3.15 Example Inspection Form – PADOT Form D-450 (continued)

Michigan Department of Transportation

BRIDGE INSPECTION REPORT 03032-B03

Bridge Number	Inspector Name	Insp Key	Pontis ID
Facility Carried			Region
Feature Intersected			Inspection Date Old New
Location			

PONTIS BRIDGE INSPECTION **English Units**

Element Number	Element Name	Total Quant	State 1 Old	State 1 New	State 2 Old	State 2 New	State 3 Old	State 3 New	State 4 Old	State 4 New	State 5 Old	State 5 New

CREW RECOMMENDATIONS

Deck Patching		
Approach Pavement		
Joint Repair		
Railing Repair		
Detailed Insp		
Zone Paint		
Substr. Repair		
Slope Repair		
Brush Cut		
Other Crew Work		

CONTRACT RECOMMENDATIONS

Bridge Replacement		
Superstructure Replacement		
Deck Replacement		
Overlay		
Widen		
Paint		
Zone Paint		
Pin and Hanger		
Substructure Repair		
Other Contract Work		

Note: This form is for use on site only, not to be submitted

Figure 4.3.16 Core Element Example Inspection Form – Michigan Department of Transportation

4.3.4 Documenting Elements Using Proper Nomenclature

Element Identification

Elements should be identified by the type of material, construction method, and by the function that each element performs.

Some material types and construction methods employed include:

- Timber
  - Solid sawn
  - Laminated
- Concrete
  - Cast-in-place: voided or solid
  - Precast: regular reinforcement or prestressed
- Steel
  - Rolled
  - Welded
  - Riveted
  - Bolted

Some examples of element functions and the abbreviations used with them are:

- Multi-beam (B1 – B6)
- Deck slab
- Stringer (S1 – S4)
- Floorbeam (FB0 – FB15)
- Girder (G1, G2)
- Truss chord (U0U1 – U.S.)
- Truss diagonal (U0L2 – D.S.)
- Secondary bracing (Top Lat. Br. U0 U.S. to U1 D.S.)
- Arch
- Spandrel column (Col. 1 – Col. 14)
- Spandrel wall (U.S., D.S. or N, S, E, W)
- Abutment (Abut. 1, Abut. 2)
- Pier (P1 – P4)

Element Orientation

Structure orientation is normally established according to highway direction of inventory, mile markers, or stationing. It is important that the orientation of each element be clearly established. The following are some examples:

- Number the substructure units (e.g., Abutment 1 and Pier 3)
- Identify sides of floorbeams with near/far (e.g., north/south or east/west) designations.
- Sides of members can be identified by direction (e.g., "south side of Floorbeam 2" or "northeast elevation of Beam 4").
- Span numbers and bay numbers should be used to identify general areas

on the bridge (see Figure 4.3.17).

- Individual beams or stringers should be numbered left to right, looking in the direction of inventory (see Figure 4.3.18).
- Upstream or downstream designations can be assigned to structures over waterways (e.g., "upstream truss," "downstream girder," or "upstream arch") (see Figure 4.3.19).
- For truss elements, identify the member with joint designations (see Figure 4.3.20). Number floorbeams in accordance with the panel point numbers.

If the orientation used during the inspection differs in any way with that used in existing documents, these differences should be clearly stated in the inspection notebook.

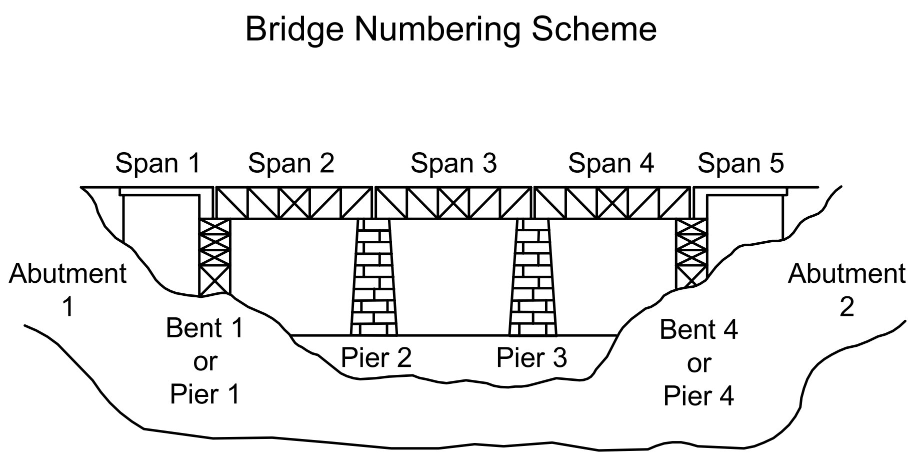


Figure 4.3.17 Sample Span Numbering Scheme

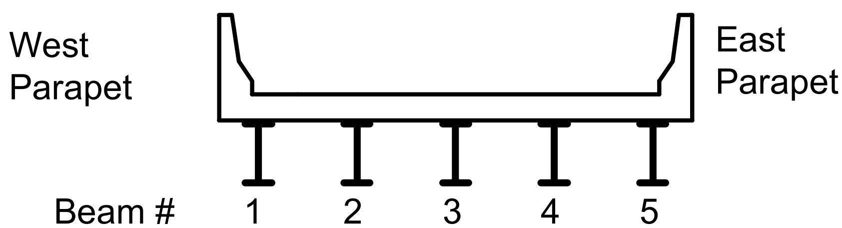


Figure 4.3.18 Sample Typical Section Numbering Scheme

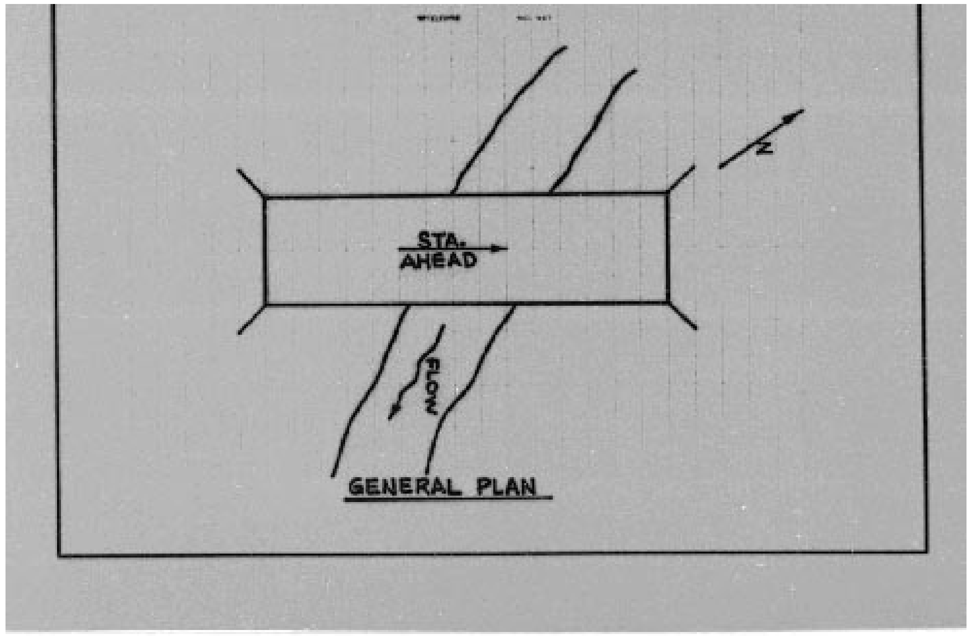


Figure 4.3.19 Sample Structure Orientation Sketch

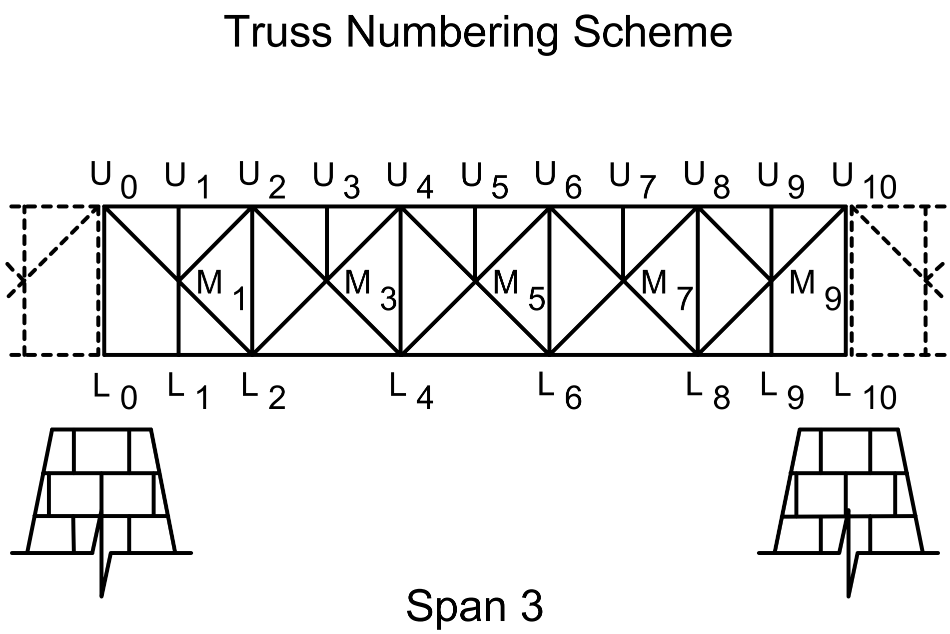


Figure 4.3.20 Sample Truss Numbering Scheme

Element Dimensions

Sufficient dimensions must be documented to establish the cross section and other pertinent dimensions of elements. These should include:

- Deck sizes – length, width, and depth
- Beam, girder, floorbeam, stringer, and truss member sizes - length, width, and depth of each; spacing and span length (see Figures 4.3.21 and 4.3.22)
- Abutment and columns - width and depth (for rectangular shapes), diameter (for round columns), length, spacing, and pile batter and spacing

(for pile bents)

- Caps and struts - width, depth, clear span, and cantilever span

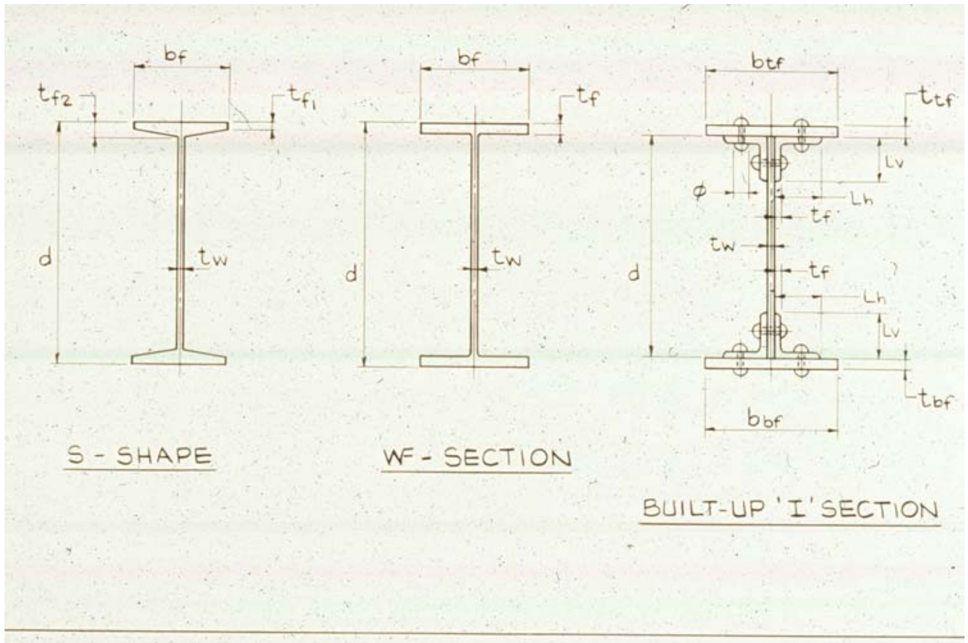


Figure 4.3.21 Steel Beam and Girder Dimensions

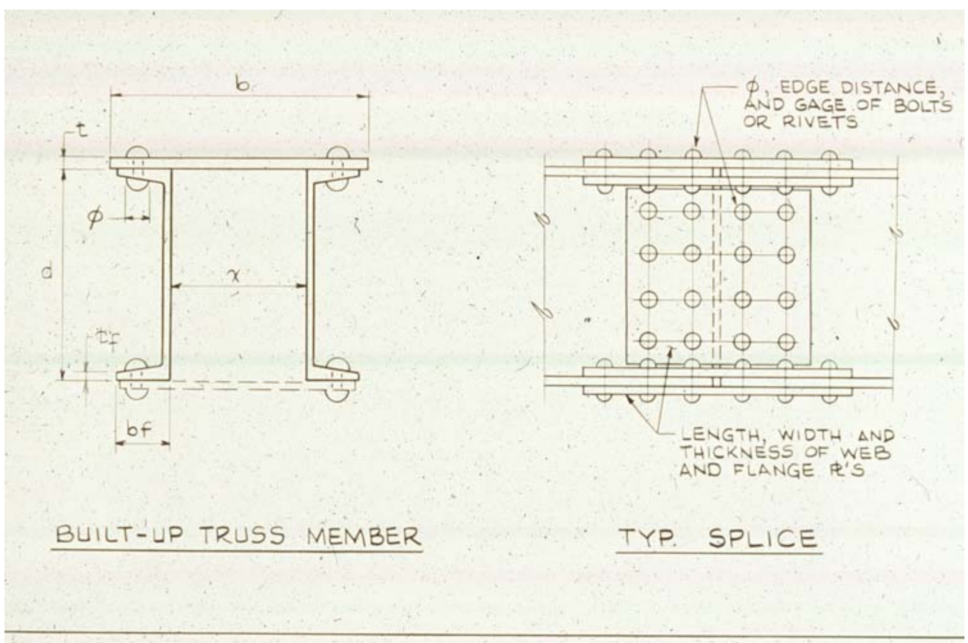


Figure 4.3.22 Truss Member and Field Splice Dimensions

4.3.5 Documenting Defects Using Proper Nomenclature

Defect Identification

Defects should be identified by their specific types.

Defects that are likely to occur in timber elements include:

- Decay - caused by either fungi or insects
- Checks - partial depth
- Splits - full depth
- Knots
- Shakes
- Wear - caused by traffic or water
- Fire damage
- Delamination of glulam beams
- Weathering
- Collision damage
- Warping

Typical concrete defects to look for include:

- Delaminations
- Spalls
- Scaling
- Cracks (structural or nonstructural)
- Exposed rebar or strands
- Corrosion or section loss to rebar or strands
- Camber (for prestressed beams)
- Chloride contamination
- Efflorescence
- Collision damage

Some of the defects that may be encountered on steel and iron elements include:

- Corrosion and section loss
- Cracks
- Deformation
- Buckling
- Fire damage
- Paint failures
- Collision damage

Defect Qualification

Documenting of defects by the inspector must describe the seriousness of a defect. For example:

- Crack sizes - record lengths, widths, and depth
- Section loss - record the remaining section dimensions (when reporting section loss, it is important to document the section remaining rather than

trying to estimate the percentage of section loss)

- Deformation - record amount of misalignment

Defect Quantification

The inspector must also describe the quantity of a defect. For example:

- Spalling – 2’ x 3’ x 2” deep
- Scaling – 4’ high by full abutment width
- Delamination – 1’ x 6”
- Decay – 2’ x 2’ x 3” deep

Defect Location

The exact position of the defect on the element or member is required if load capacity analysis is to be performed. For example:

- Left side of web, top half, 3 feet from north bearing
- Top of top flange, from 3 feet to 6 feet west of Pier 2

The accuracy of the load capacity analysis depends on precise location information for defects:

- Bending moment – Flexure is maximum at or near midspan for simple span structures. Maximum negative moment occurs at the intermediate supports if the structure is continuous.

- Shear – Shear is maximum at or near the supports.

- Axial compression members - The capacity of the member to resist compressive forces is reduced by any deformation or change in cross section. The potential capacity reduction is not dependent on where on the member the defect is located. All segments are critical.

- Axial tension members - These members experience a reduction in capacity through loss of section or from cracking. As with the axial compressive members, tensile members are equally susceptible regardless of the location of the defect.

- Combinations - While axial members are critical at all locations, it is not always apparent which members are loaded only in an axial direction. In fact, due to the dead load of the member itself, most are not. Other factors can also contribute to bending forces that will create varying moments, shears, compression, and tension areas within a member that is primarily axial. Because of this, inspectors should identify the exact position of defects in all members using reference points, regardless of the forces acting on the member.

Locating a defect may include tying it to an established permanent reference. Avoid using references that can change over time.

Some examples of proper referencing include:

- 2210 mm (7’-3”) from fixed bearing on Beam 3 at Abutment 1
- 940 mm (3’-1”) from west corner of Abutment 2
- 760 mm (2’-6”) below bridge seat on south face of Column 1, Pier 2

Reference points to avoid:

- Expansion rocker faces
- Ground levels, especially those that may be exposed to water
- Water levels

4.3.6 Structure Files

Structure files are used to maintain detailed information on each important structure. A thorough study of the available historical information can be extremely valuable in identifying possible critical areas of structural or hydraulic components and features. Because this information may require considerable effort to assemble, a separate file should be established for each structure.

The contents of any particular file may vary depending upon the size and age of the structure, the functional classification of the road carried by the structure, and the informational needs of the agencies responsible for inspection and maintenance. A very small structure may be documented in an inventory listing or with a file that contains little more than an inventory card plus dates and comments of previous inspections. For larger structures, it is recommended that the following types of information be assembled when possible.

Construction and Design

"As built" or design plans should be included in a structure file. If plans are not available, the following types of construction information should be determined: date built; type of structure, including size, shape, and material; design capacity; and design service life. Hydraulic data should also be assembled where available, including slope of structure, elevation of inverts or footings, stream channel and water surface during normal and high flows, design storm frequency, drainage area, design discharge, date of design policy, flow conditions, limits of flood plain, type of energy dissipaters (if present), cut-off wall depth, channel alignment, and channel protection.

Repair History

Information about repairs and rehabilitation activities should be collected. The types and amount of repairs performed at a bridge or culvert site can be extremely important. Frequent roadway patching due to recurring settlement over a culvert or approach roadway for a bridge may indicate serious problems that are not readily apparent through a visual inspection of the structure itself.

Inspection History

Data from previous inspections can be particularly useful in identifying specific locations that require special attention during an inspection. Information from earlier inspections can be compared against current conditions to estimate rates of deterioration and to help judge the seriousness of the problems detected and the anticipated remaining life of the structure.

This page intentionally left blank

Table of Contents

Section 4
Bridge Inspection
Reporting System

Topic 4.4 The Inspection Report

4.4.1 Introduction

The purpose of the bridge inspection reporting system is to have trained and experienced personnel record objective observations of all elements of a bridge and to make logical deductions and conclusions from their observations.

The bridge inspection report should represent a systematic inventory of the current or existing condition of all bridge members and their possible future weaknesses. Moreover, bridge reports form the basis of quantifying the manpower, equipment, materials, and funds that are necessary to maintain the integrity of the structure.

A bridge inspection is not complete until an inspection report is finalized. The bridge inspection report must document all signs of distress and deterioration with sufficient precision so that future inspectors can readily make a comparison of condition. A complete inspection report contains several parts, as outlined in this topic. A sample bridge inspection report is presented in Appendix B. An inspection report should be prepared for special inspections, which are conducted for checking a specific item where a problem or change may be anticipated. Even if no changes are evident, a report should be made for each type of bridge inspection, even if it may only be a cursory inspection. Some bridge owners also request a special bridge inspection and report when planning a major rehabilitation.

4.4.2 Basic Components of a Comprehensive Bridge Inspection Report

Table of Contents

The table of contents should present the general headings and topics of the inspection report in an orderly manner so that individual sections of the report can be found with ease. It generally follows the title page, and individual sections are listed with their corresponding starting page number.

Location Map

A map should be included with a scale large enough to positively locate the structure. The bridge should be clearly marked and labeled, and the map should have a north arrow.

Bridge Description and History

The bridge description and history section of the report should contain all pertinent data concerning the design, construction, and use of the bridge. The type of superstructure will generally be given first, followed by the type of abutments and piers, along with their foundations. If data is available, indicate the type of foundation soil, maximum bearing pressures, and pile capacities. The type of deck is also indicated.

Design Data

The design information should include a description of the following:

- Skew angle
- Number and length of spans
- Total length
- Bridge width
- Wearing surface
- Sidewalks
- Railing
- Year constructed/reconstructed
- Number of traffic lanes
- Design live loading
- Waterway
- Other features intersected
- Clearances
- Encroachments
- Alignment

Construction Data

The construction history of the bridge should include the date it was originally built, as well as the dates and descriptions of any repairs or reconstruction projects. State what plans are available, where they are filed, and whether they are "as-built."

Service Data

State the average daily traffic (ADT) count and the average daily truck traffic (ADTT) count, along with the date of record. This information should be updated approximately every five years. Any environmental conditions which may have an effect on the bridge, such as salt spray, industrial gases, bird droppings, and ship and railroad traffic, should be noted in the report.

The history of the bridge is from a structural standpoint and should be developed from information obtained from design, construction and rehab plans, previous inspection reports, maintenance records, discussions with maintenance crews and local residents, and any other available source that offers pertinent information. Items to be included in the history narrative include:

- Year built
- Reconstruction year, if any
- Historical flood frequencies and high water marks
- Maintenance measures and repairs
- Chronological record of conditions
- Reference drawings
- Photos

Executive Summary

The executive summary is a narrative presentation summarizing the inspection and analysis findings in regard to the qualitative condition and the load capacity of the bridge, along with an overview of recommendations. The executive summary must properly identify the bridge (e.g., name, number, and location) and the date of inspection. The executive summary should also present any high priority repair items.

Inspection Procedures

The procedures and equipment used to inspect the bridge should be documented. In most instances, it is advantageous to inspect structures in the same sequence as the load path (i.e., the deck first, then the superstructure, and finally the substructure). This manual is organized and presented in that sequence.

However, many inspections cannot follow this sequence due to traffic and lane-closure restrictions. It is useful to document whatever sequence was used during the inspection. This information will be useful in planning future inspections and will also serve as a checklist to make sure that all elements and components were inspected. The following information should be included:

- Equipment required (e.g., hammers and plumb bobs)
- Access equipment (e.g., rigging, ladders, and free climbing)
- Access vehicles (e.g., inspection cranes and bucket trucks)
- Traffic restrictions (e.g., lane closures, flagmen, and hours of operation)
- Inspection methods (e.g., corings and ultrasonic)
- Personnel (e.g., by name and classification)
- Special equipment (e.g., material testing and underwater inspection)
- Deviations from "hands-on" inspection of all areas
- Time required for inspection
- Channel profiles

When structure plans are not on file and a load rating has not been calculated, it may be necessary to obtain field measurements to permit calculation of the load capacity of the structure.

Inspection Results

Narrative descriptions of the conditions should be both quantitative and qualitative, indicating the locations and the extent of the affected areas. Use forms consistent with similar inspections. Note all signs of distress, failure, or defects with sufficient precision so that a deterioration rate can be determined. This is very important for determining estimated remaining life and an optimal improvement stategy.

Note any load, speed, or traffic restrictions on the bridge. Include information about high water marks and unusual loadings. Note the environmental conditions such as temperature, rain, or snow. All work or repairs to the bridge since the last inspection should be listed. Verify or obtain new dimensions when improvement work has altered the dimensions of the structure. New streambed profiles should be taken with each inspection to detect scour, channel migration, or channel aggredation and degradation.

The seriousness and amount of all deficiencies must be clearly stated. In emergency situations, the inspector should immediately contact the inspection supervisor and the representative of the bridge owner.

Load Rating Summary

A summary of any load capacity rating analysis that has been performed should be included in the report. The summary should be presented in a table or chart. Governing load ratings should be shown for both inventory and operating levels for all types of loadings used in the analysis. The governing member for each rating should be identified. The governing member is the one that has the lowest capacity for a given type of loading.

For example, in a Girder-Floorbeam-Stringer structure, Stringer 3 in Bay 5 may have the lowest capacity for carrying HS20 trucks, compared to all other stringers, floorbeams, or girders. The HS20 inventory and operating ratings for this stringer would be reported, and it would be identified as the governing member.

Conclusions and

A good inspection report should explain in detail the type and extent of any

Recommendations

deterioration found on the bridge and should point out any deviations or modifications that are contrary to the "as-built" construction plans. The depth of the report should be consistent with the importance of the deterioration. Not all conditions of deterioration are of equal importance. For example, a crack in a prestressed concrete box beam which allows water to enter the beam is much more serious than a vertical crack in an abutment backwall or a spall in a corner of a slopewall.

The inspector, in formulating conclusions for the cause of the defect, must report the seriousness of the defect or deficiency involved. The inspector's experience and judgment are called upon when interpreting inspection results and arriving at reasonable and practical conclusions. The conclusions are the heart of the inspection report. Improper and misinformed conclusions will lead to improper recommendations. The inspector may need to play the role of a detective to conclude why, how, or when certain defects occurred. When the inspector cannot interpret the inspection findings, the advice of more experienced personnel should be sought.

The recommendations made by the inspector constitute the "focal point" of the operation of inspecting, recording, and reporting. The inspector must review previous inspection recommendations and identify any that have not been addressed, particularly if urgent. A thorough, well documented inspection is essential for making informed and practical recommendations to correct or preclude bridge defects or deficiencies.

All recommendations for maintenance work, stress analysis, postings, further inspection, and repairs should be included. The inspector must carefully consider the benefits to be derived from making repairs and the consequences if the suggested repairs are not made. The inspector should list, in order of greatest urgency, any repairs that are necessary to maintain structural integrity and public safety. Recommendations concerning repairs may be classified into two general categories:

- Urgent repairs
- Programmed repairs (i.e., those to be performed sometime later)

The inspector must decide whether a repair is urgent. Usually this is easily determined, but occasionally the experience and judgment of a Professional Engineer may be required to reach a proper decision. A large hole through the deck of a bridge obviously needs attention, and a recommendation for emergency repair is in order. By contrast, a slightly deteriorated gusset plate at a panel point of a truss may not be critical. A condition such as this would appropriately call for a recommendation for a programmed repair.

Typically, most recommendations concerning repairs submitted by the bridge inspector will be in the category of programmed repairs (i.e., repairs that will be incorporated into preprogrammed repair and maintenance schedules). Whenever recommendations call for bridge repairs, the inspector must carefully describe the type of repairs that are needed, the scope of work to be done, and an estimate of the quantity of materials that will be required.

If not already described in the executive summary, the conclusions and recommendations section of the report should summarize the following:

- Overall condition
- Major deficiencies
- Load-carrying capacity
- Recommendations for:
  - Further inspection
  - Maintenance
  - Repairs
  - Painting
  - Posting
  - Rehabilitation
  - Replacement

Some state and local agencies designate separate personnel to prepare recommendations and cost estimates.

Report Appendices

The appendices should contain any back-up information that can be used to substantiate the inspector's conclusions and recommendations. As a minimum, the appendix should include photographs, drawings and sketches, and inspection forms. It can also include copies of any field notebooks used and specialist reports (e.g., underwater, nondestructive testing (NDT), and survey), or these documents can be referenced in the report.

Photographs

Photographs will be of great assistance to anyone reviewing reports on bridge structures. It is recommended that pictures be taken of any problem areas that cannot be completely explained by a narrative description. It is better to take several photographs that may be unessential than to omit one that would preclude misinterpretation or misunderstanding of the report. At least two photographs of every structure should be taken. One of these should depict the structure from the roadway, while the other photo should be a view of the side elevation. Also, photographs should be inserted on sheets that are the same size as the report pages. Captions should be provided for each photo, and photos should be numbered so that they can be referred to in the body of the report.

Drawings and Sketches

Sketches should be used freely as needed to illustrate and clarify conditions of structural elements. Clear diagrams are very helpful at future investigations in determining the progression of defects and to help determine any changes and their magnitude. Drafting-quality plans and sketches, sufficient to indicate the layout of the bridge, should be included as an appendix.

Inspection Forms

The inspection forms should contain the actual field notes, as well as the numerical condition and appraisal ratings by the inspector. The inspection forms must be signed by the inspection team leader. A complete SI&A form should be included in the appendix. If a previous report or printout is used for inventory data, items should be field checked for accuracy.

Load Capacity Analysis

Stress analysis is frequently performed on the structure to determine the load capacity of the bridge. It should include investigation of all primary load-carrying members of the bridge. Such analysis is normally performed by engineers in the office, not by the inspector. Also, not all inspections require stress analysis.

Field Inspection Notes

The original notes taken by the inspectors in the field or photocopies thereof should be included in the appendix section of the report. The original field notes are source documents and as such should be included in the structure file.

Underwater Inspection Report

If an underwater inspection of the substructure has been performed, a separate report is usually prepared by the diver. If applicable, the diver's report should be included in the appendix.

Material Testing Results

Sometimes material testing is performed on a structure in order to determine the strength and properties of an unknown or suspect material. The testing lab's report should also be included in the appendix of the bridge inspection report.

To achieve maximum effectiveness, each report should be supplemented with sketches, photographs, or any other additional explanatory information. Reports and supplemental information must be accurate, and descriptions or explanations should be clear and concise.

4.4.3 Importance of the Inspection Report

Source of Information

The bridge inspection report is an extremely valuable document when completed properly. A new inspection report should be made each time a bridge is inspected.

A well prepared report will not only provide information on existing bridge conditions, but it also becomes an excellent reference source for future inspections, comparative analyses, and bridge study projects. Any conditions that are suspicious but unclear should be reported in a factual manner, avoiding speculation. Further action on such reports will be determined after review and consultation by more experienced personnel.

Legal Document

In preparing a report, keep in mind that bridge funding may be allocated or repairs designed based on this information. Furthermore, the inspection report is a legal record which may form an important element in some future litigation. The language used in reports should be clear and concise and, in the interest of uniformity, care should be taken to avoid ambiguity of meaning. The information contained in reports is obtained from field investigations, supplemented by reference to "as-built" or "field checked" plans. The source of all information contained in a report should be clearly stated.

The inspector should sign and date the inspection forms and condition reports as they are completed. No undocumented alterations should be made to the report once it is completed. Some inspectors retain copies of their reports for their personal files in the interest of self-protection should any litigation come about.

Critical Areas

A primary purpose of the inspection report is to provide guidance for immediate follow-up inspections or action. The report provides information which may lead to decisions to limit the use of, or to close to traffic, any bridge which the inspection has revealed to be hazardous to public safety.

Maintenance

Another purpose of the inspection report is to provide useful information about the needs and effectiveness of routine maintenance activities. An active preservation program is vital to the long-term structural integrity of a bridge. The inspection report enables bridge maintenance to be programmed more effectively through early detection of structural defects or deficiencies, thus minimizing repair costs.

Load Rating Analysis

When an inspection reveals defects or deficiencies that may effect the load capacity of the structure, it should be reviewed by an engineer to determine if a revised stress analysis is needed. Any new stress analysis is made to determine the safe load capacity for the current condition. It may then be necessary to restrict loads crossing the bridge so that its safe load capacity is not exceeded. It is important that the calculations for the revised load-carrying capacity analysis become part of the structure file.

Bridge Management

Another purpose of the inspection report is analysis by the states and the FHWA of the SI&A data. The intent of the analysis is to aid in the decisions for allocating and prioritizing funding.

4.4.4 Quality

The accuracy and uniformity of information collected and recorded is vital for the management of a state's bridges for rehabilitation, maintenance, replacement, and, most importantly, public safety. Quality cannot be taken for granted. The responsibility of ensuring quality bridge inspections rests with each state or agency. The operation of a quality review will be determined by the organization of the inspection teams. Two phrases are frequently used when discussing quality: quality control and quality assurance.

Quality Control

Quality control (QC) is the establishment and enforcement, by a supervisor, of procedures that are intended to maintain the quality of the inspection at or above a specific level. If a state's inspection program is decentralized, the individual districts are responsible for their own QC. If the inspection efforts are centralized, then the responsibility for QC is at the centralized level.

Quality Assurance

Quality assurance (QA) is the verification of the level of quality of the bridge inspection. This is accomplished by the reinspection of a sample of bridges by an independent inspection team. For decentralized state inspections, the QA program can be performed by the central staff or their agent (e.g., consultants). If the inspections are centralized within the state, then the QA program should be performed by consultants or a division separate and independent of the inspection organization.

The quality of the inspection and reports rests primarily with the inspection team leaders and team members and their knowledge and professionalism in developing a quality product. A QA/QC program is a means by which random inspections, reviews, and evaluations are performed in order to provide feedback concerning the quality and uniformity of the state's or agency's inspection program. The feedback is then used to improve the inspection program through improved inspection processes and procedures, training, and quality of the inspection report.

Sample inspection report is located in the appendix.

Table of Contents

Section 5

Inspection and Evaluation of Decks

Section 5

Inspection and Evaluation of Decks

Topic 5.1 Timber Decks

5.1.1 Introduction

Timber bridges make up approximately 7% of the bridges listed in the National Bridge Inventory (NBI). Furthermore, approximately 7% of the steel bridges, which are categorized as steel bridges in the NBI also have timber decks. Timber can be desirable for use as a bridge decking material because it is resistant to deicing agents, which typically harm concrete and steel, and it is a renewable source of material. Timber can also withstand relatively larger loads over a short period of time when compared to other bridge materials. Finally, timber is easy to fabricate in any weather condition and is lightweight.

5.1.2 Design Characteristics

Timber decks are normally referred to as decking or timber flooring, and the term is generally limited to the roadway portion which receives vehicular loads. Timber decks are usually non-composite because of the inefficient shear transfer through the attachment devices. The basic types of timber decks are:

- Plank decks
- Nailed laminated decks
- Glued-laminated deck panels
- Stressed-laminated decks
- Structural composite lumber decks

Plank Decks

Plank decks consist of timber planks laid transversely across the bridge (see Figure 5.1.1). The planks are individually attached to the bridge beams using spikes or bolt clamps, depending on the beam material. It is common for plank decks to have 50 mm (2-inch) depth timbers nailed longitudinally on top of the planks to distribute load and retain the bituminous wearing surface.

Figure 5.1.1 Plank Deck

Nailed Laminated Decks

Nailed laminated decks consist of timber planks with the wide dimensions of the planks in the vertical position and laminated by through-nailing to the adjacent planks (see Figure 5.1.2). On timber beams, each lamination is toenailed to the beam. On steel beams, clamp bolts are used as required. In either case, laminates are generally perpendicular to the roadway centerline.

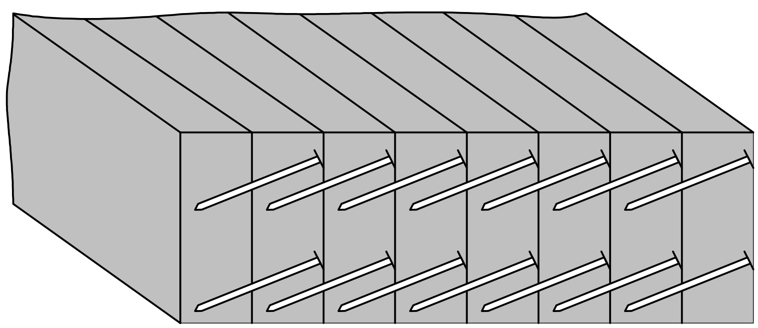


Figure 5.1.2 Section of a Nailed Laminated Deck

Glued-laminated Deck Panels

Glulam is an engineered wood product in which pieces of sawn lumber are glued together with waterproof adhesives. Glued laminated deck panels come in sizes usually 1.2 m (4 feet) wide. The panels are laid transverse to the traffic and are attached to the superstructure. In some applications, the panels are interconnected with dowels. There are several techniques used to attach glued-laminated decks to the superstructure or a floor system, including nailing, bolting, reverse bolting, clip angles and bolts, and nailers (see Figure 5.1.3).

The nailing method is generally not preferred due to the possibility of the nails being pried loose by the vehicle traffic.

Bolting the deck to the superstructure or floor system provides a greater resistance to uplift than nailing, but bolts may still be pried loose.

Reverse bolting involves fastening the bolts to the underside of the deck on either side of the stringers, thereby preventing the lateral movement of the deck. This is a rare type of connection.

Clip angles and bolts involve attaching clip angles to the stringers and then using bolts to attach the clip angles to the deck.

Nailers are planks that run along the top of steel stringers. This technique involves the bolting of the nailers to the stringers and nailing the timber planks to the nailers. This prevents the costly bolting of all planks to the steel stringers.

Figure 5.1.3 Glued-laminated Deck Panels

Stressed-laminated Decks

Stressed-laminated decks are constructed of sawn lumber glulam wood post-tensioned transversely utilizing high strength steel bars. Stressed timber decks consist of thick, laminated timber planks which usually run longitudinally in the direction of the bridge span. The timber planks vary in length and size. The laminations are squeezed together by prestressing (post-tensioning) high strength steel bars, spaced approximately 600 mm (24 inches) on center. With a hydraulic jacking system tensioning the bars, they are passed through predrilled holes in the laminations. Steel channel bulkheads or anchorage plates are then used to anchor the prestressing bars. This prestressing operation creates friction connections between the laminations, thereby enabling the laminated planks to span longer distances (see Figure 5.1.4).

Prestressed laminated decks are used on a variety of bridge superstructures, such as trusses and multi-beam bridges, and they can be used as the superstructure itself for shorter span bridges.

Figure 5.1.4 Stressed-laminated Deck

Structural Composite Lumber Decks

Structural composite lumber (SCL) decks include laminated veneer lumber (LVL) and parallel strand lumber (PSL). Laminated veneer lumber is made by gluing together thin sheets of rotary-peeled wood veneer with a waterproof adhesive. Parallel strand lumber is made by taking narrow strips of veneer and compressing and gluing them together with the wood grain parallel. SCL bridge decks are gaining popularity and are comprised of a parallel series of fully laminated LVL or PSL T-beams or a parallel series of fully laminated LVL or PSL box beams. The T-beams and box sections run parallel with the direction of traffic and are cambered to meet the needs of the specific bridge site. The box sections or T-beams are stress laminated together by either placing steel bars or prestressing strands through the top flanges (timber deck area) and/or through the outside edges of the box section top flanges. Steel channels or bearing plates are then placed on the bars or strands with double nuts. Standard strand chucks are placed on the opposite end to initiate the prestressing process. The prestressing bars or strands are generally epoxy coated to resist corrosion.

Structural composite lumber decks are capable of full preservative penetration, and asphalt overlays are typical.

Figure 5.1.5 Structural Composite Lumber Deck Using Box Sections

5.1.3 Wearing Surfaces

The wearing surface of a timber deck is constructed of either timber, bituminous materials, or concrete. Timber wearing surfaces usually run parallel with traffic and are used with transverse plank decks. Bituminous wearing surfaces can either be hot mix asphalt or a chip and seal method. Concrete wearing surfaces for timber decks are less common than timber or bituminous wearing surfaces, although some exist.

Timber

A timber wearing surface may consist of longitudinal timbers placed over the transverse decking. Runner planks or "running boards" are planks placed longitudinally only in the strips where the wheels of vehicles ride (see Figure 5.1. 6).

Figure 5.1.6 Timber Wearing Surface on a Timber Deck

Bituminous

Bituminous or asphalt wearing surfaces generally consist of a coarse aggregate. The aggregate is mixed with a binder substance that holds the aggregate together and bonds the surfacing to the deck. Asphalt is a popular bituminous wearing surface for timber decks. However, it is not commonly used on plank decks due to the fact that deflection of the planks will cause the asphalt to break apart.

Concrete

While concrete may be used as a wearing surface on timber decks, it is not frequently used for this purpose. However, new composite studies between concrete overlays and timber decks are being performed. These studies generally involve a timber deck with steel shear studs doweled into the timber deck with a concrete overlay completing the composite action.

5.1.4 Protective Systems

Protective systems are necessary to resist decay in timber bridge decks. Water repellents, preservatives, fumigants, fire retardants, and paints are some of the common timber protective materials. In order for the protective material to serve its purpose, the surface of the timber must be properly prepared.

Water Repellents

Water repellents help to prevent water absorption in timber decks, which slows decay by molds and weathering. The amount of water in wood directly affects the amount of expansion and contraction due to temperature. Water repellents are used to lower the water content of timber deck members and must be reapplied periodically. Because it needs to be applied rather frequently, it is not the best means of protecting timber structures.

Preservatives

Timber preservatives are usually applied by pressure, which forces the preservative into the timber deck member. The deeper the preservative penetration, the greater the protection from decay by fungi. Preservatives are the best way to protect against decay.

Preservatives are either oil-based or water-based. Some common oil-based preservatives are coal-tar creosote and pentachlorophenol. Chromated copper arsenate (CCA) is a very common water-based preservative.

Fumigants

Fumigants are applied to timber members in a liquid form through drilled holes. Once in the hole, the hole is plugged and the fumigant volatilizes and moves through the member as a gas, thus preserving the internal heartwood. Two common types of fumigants are chloropicrin and metham. These two fumigants are very hazardous and can only be applied by a professional. Also, the locations in which these fumigants can be applied are limited.

Fire Retardants

Fire retardants slow the spread of fire and prolong the time required to ignite the wood. The two main types of fire retardants are pressure impregnated salts and intumescent paints. These retardants insulate the wood, but adversely affect the material properties of wood.

Paint

Paints for timber decks can either be oil-based, oil-alkyd or latex-based. Oil-based paints provide the best barrier from moisture but is not very durable. Oil-alkyd paints have more durability than oil-based paints but contain lead pigments which cause various health hazards. Latex-based paints, on the other hand, are very flexible and resistant to chemicals.

5.1.5

Overview of Common Defects

A prepared bridge inspector should know what to look for prior to the inspection. The following is a list of common defects that may be encountered when inspecting timber bridge decks. Refer to Topic 2.1 for a detailed description of these common defects:

- Fungus decay
- Damage by parasites
- Deflection
- Checks
- Splits
- Shakes
- Loose connections
- Surface depressions
- Chemical attack

5.1.6 Inspection Procedures and Locations

Procedures

Visual

The inspection of timber decks for deterioration and decay is primarily a visual activity. All surfaces of the deck planks should receive a close visual inspection.

Physical

However, physical examinations must also be used for suspect areas. The most common physical inspection techniques for timber include sounding, probing, drilling, core sampling, and electrical testing. An inspection hammer should be used initially to evaluate the subsurface condition of the planks and the tightness of the fasteners. In suspect areas, probing can be used to reveal decayed planks using a pick test or penetration test (see Figure 5.1.7). A pick test involves lifting a small sliver of wood with a pick or pocketknife and observing whether or not it splinters or breaks abruptly. Sound wood splinters, while decayed wood breaks abruptly. If the deck planks are over 50 mm (2 inches) thick, suspect planks should be drilled to determine the extent of decay.

Figure 5.1.7 Inspector Probing Timber with an Ice Pick at Reflective Cracks in the Asphalt Wearing Surface

Advanced Inspection Techniques

In addition, several advanced techniques are available for timber inspection. Nondestructive methods, described in Topic 13.1.2, include:

- Pol-Tek
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 13.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Shigometer

Locations

Timber deck inspection generally includes visually interpreting the degree of decay on the top and, if visible, the bottom and sides of the deck. Also, all visible fastening devices and bearing areas should be inspected. In all instances, it is helpful if the inspector has available the previous inspection report so that the progression of any deterioration can be noted. This provides a more meaningful inspection.

The primary locations for timber deck inspection include:

- **Areas exposed to traffic** - examine for wear, weathering, and impact damage (see Figure 5.1.8)
- **Bearing and shear areas** where the timber deck contacts the supporting floor system - inspect for crushing, decay, and fastener deficiencies (see Figure 5.1.9)
- **Tension areas** between the support points - investigate for flexure damage, such as splitting, sagging, and cracks
- **Deck surface** - check for decay, particularly in areas exposed to drainage (see Figure 5.1.10)
- **Outside edges of deck** - inspect for decay
- **Nailed laminated decks** - swelling and shrinking from wetting and drying cause a gradual loosening of the nails, displacing the laminations; this permits moisture to penetrate the deck and superstructure, eventually leading to decay and deterioration of the deck
- **Prestressing anchorages** – check for corrosion, crushing, and decay
- **Fire damage**

Figure 5.1.8 Wear and Weathering on a Timber Deck

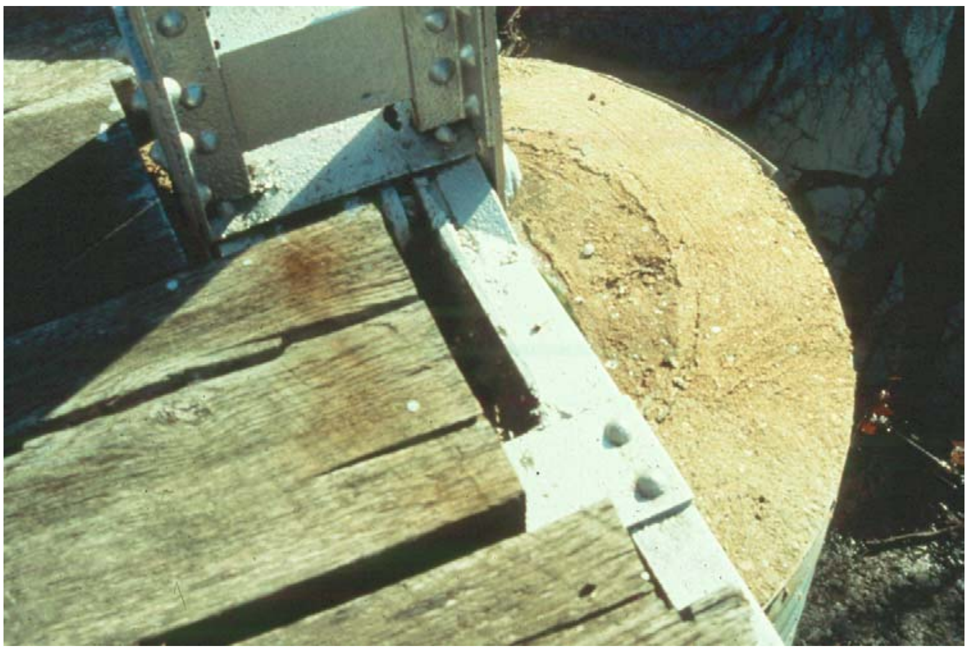


Figure 5.1.9 Bearing Area on a Timber Deck

Figure 5.1.10 Edge of Deck Exposed to Drainage, Resulting in Plant Growth

5.1.7 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of all bridge members, including timber decks. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) component rating method and AASHTO element level condition state assessment method.

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 deck. Rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 for additional details about the NBIS rating guidelines. 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 top of deck or slab and the underside. 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 timber deck, the AASHTO CoRe element is one of the following, depending on the riding surface:

Element No.	Description
031	Timber Deck
032	Timber Deck – with AC Overlay
054	Timber Slab
055	Timber Slab – with AC Overlay

The unit quantity for these elements is "each", and the entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2). Condition state 1 is the best possible rating. The inspector must know the total slab surface area in order to calculate a percent deterioration and fit into a given condition state description. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For the purposes of this manual, a deck is supported by a superstructure and a slab is supported by substructure units.

Table of Contents

Section 5
Inspection and Evaluation of Decks

Topic 5.2 Concrete Decks

5.2.1 Introduction

The most common bridge deck material is concrete. The physical properties of concrete permit casting in various shapes and sizes, providing the bridge designer and the bridge builder with a variety of construction methods. This topic discusses various aspects of concrete bridge decks and related bridge inspection issues.

5.2.2 Design Characteristics

The role of a concrete bridge deck is to provide a smooth riding surface for motorists, divert runoff water, distribute traffic and deck weight loads to the superstructure, and act compositely or non-compositely with the superstructure. Increased research and technology are providing the bridge deck designer with a variety of concrete mix designs, from lightweight concrete to fiber reinforced concrete to high performance concrete, as well as different reinforcement options, to help concrete bridge decks better perform their role.

There are four common types of concrete decks:

- Reinforced cast-in-place (CIP)
- Precast
- Precast prestressed
- Precast prestressed deck panels with CIP topping

Reinforced Cast-in-Place

Concrete decks that are cast in place on the bridge are referred to as "cast-in-place" (CIP). Forms are used to contain reinforcing bars and wet concrete so that after curing, the deck components will be in the correct position and shape. "Bar chairs" are used to support reinforcement in the proper location during casting. There are two types of forms used when placing cast-in-place concrete: removable and stay-in-place.

Removable forms are usually wood planking or plywood but can also be fiberglass reinforced plastic. These forms are removed from the deck after the concrete has cured.

Stay-in-place (SIP) forms are corrugated metal sheets permanently installed above or within the floor system. After the concrete has cured, these forms, as the name indicates, remain in place as permanent, nonworking members of the bridge (see Figure 5.2.1).

Figure 5.2.1 Stay-in-Place Forms

Precast

Precast deck panels are reinforced concrete panels that are cast and cured somewhere other than on the bridge. Precast decks are typically reinforced with conventional mild reinforcement. The slabs are transported to the bridge site, then placed on the bridge, leveled, and attached to the superstructure/floor system. Leveling is generally accomplished using leveling bolts and a grouting system.

The precast deck panels fit together using match cast keyed construction. After leveling, precast deck panels are attached to the superstructure/floor system. Mechanical clips can be used to bolt the deck panels to the stringers. An alternate method involves leaving block-out holes in the precast panels as an opening for shear connectors. The deck panels are positioned over the shear connectors, and the block-out holes are then filled with concrete or grout.

Precast Prestressed

Precast prestressed decks are also reinforced concrete slabs cast and cured away from the bridge site. However, they are reinforced with prestressing steel in addition to some mild reinforcement. The prestressing tendons or bars are tensioned prior to placing the slab (pretensioned) or after the slab is cured (post-tensioned). The tendons are held in position until the slab has sufficiently cured. This creates compressive forces in the slab, which reduce the amount of tension cracking in the cured concrete.

Precast Prestressed Deck Panels with Cast-in-Place Topping

Precast prestressed deck panels can also be used in conjunction with a cast-in-place concrete overlay. Partial depth reinforced precast panels are placed across the beams or stringers and act as forms (see Figure 5.2.2). A cast-in-place layer, which may be reinforced, is then placed which engages both the supporting members and the precast slab units. After the cast-in-place layer has cured, composite action is achieved with the precast deck panels.

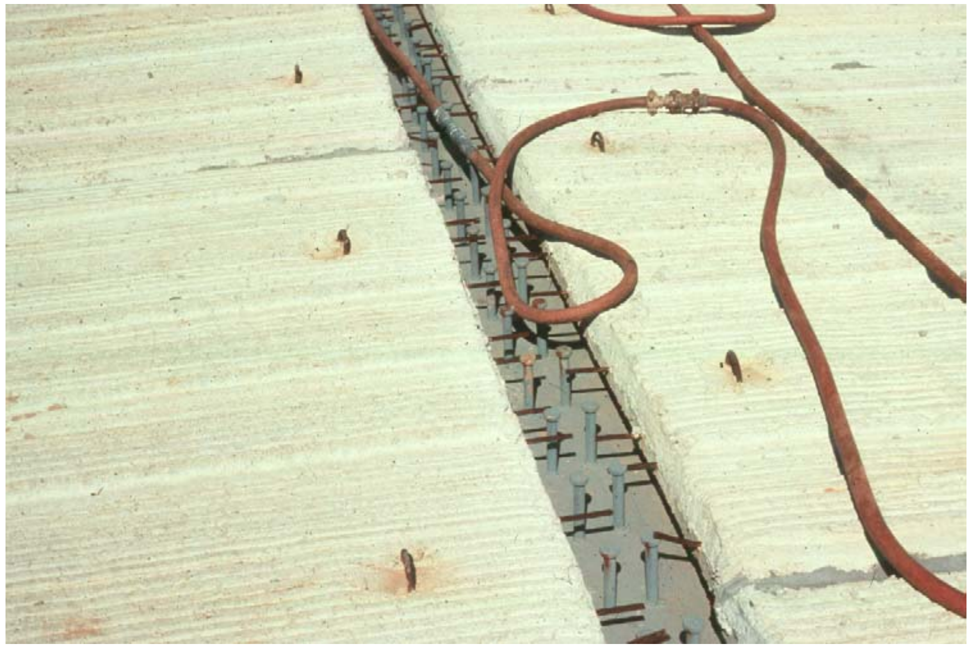


Figure 5.2.2 Precast Deck Panels (with Lifting Lugs Evident and Top Beam Flange Exposed)

In addition to the four common concrete decks, there are two new types of decks that may become more common in the future:

- Fiber reinforced polymer (FRP)
- Fiber reinforced concrete (FRC)

Fiber Reinforced Polymer

New and innovative research is being performed in the area of fiber reinforced polymer (FRP) bridge decks. Most of the FRP composite deck systems use glass reinforcing fibers set in a polyester or vinylester resin matrix. The two most common FRP deck systems use prefabricated panels comprised of pultruded tubes that are glued together with adhesive, and honeycomb or sandwich core systems that are hand-laid up or utilize vacuum assisted resin transfer molding techniques. These deck systems are factory built to the specified deck panel dimension and are then shipped to the erection site. Once at the site, the individual deck panels are bonded together with high performance adhesives. If beams support this type of deck system, a grouted haunch or fillet is required to take into account the imperfections of the beams. Composite action can be developed with FRP deck systems by cutting pockets in the deck to access welded shear studs and then grouting the pockets. Composite action in FRP deck systems is still being researched.

FRP decks require an overlay due to the low skid resistance of the materials. Latex concrete, micro-silica concrete, or dense concrete are not very compatible with FRP deck systems in the areas of stiffness, tensile strength, or compressive strength. Thin epoxy or polymer modified concrete overlays are better suited for use with FRP deck systems. Hot asphalt has been used as an overlay and has worked well over several years on some decks.

An 200 mm (8 inch) deep FRP deck weighs only 98 kg/m^2 (20 pounds per square foot (psf)) compared to 488 kg/m^2 (100 (psf)) for a conventional 200 mm (8 inch) concrete bridge deck.

Fiber Reinforced Concrete

Another new bridge deck material is fiber reinforced concrete (FRC). This type of bridge deck uses common Portland cement concrete mixes with 0.2 to 0.8 percent fiber by volume. The most common type of fiber reinforcement is polypropylene. The purpose of the fiber is to minimize shrinkage cracking of fresh concrete and increase the impact strength of cured concrete.

An FRC bridge deck can either be reinforced with conventional rebar or have no conventional steel reinforcement included in the deck. Initial research testing the ability of polypropylene fibers to block the corrosion of steel reinforcement in concrete bridge decks proved that the fibers did not significantly retard the corrosion process. Therefore, most FRC bridge decks have been designed and constructed without steel reinforcement. FRC decks without steel reinforcement have transverse steel straps welded to the top flange of steel girders and are made composite with the superstructure via shear studs welded to the top flange. The steel straps run the entire width of the deck and provide lateral restraint of the supporting girders. Since no steel reinforcement is included in the deck itself, the deck does not deteriorate due to steel reinforcement corrosion. Therefore, steel-free bridge decks give designers a viable alternative in areas where deicing salts are used.

Composite Action

A concrete deck is generally required when composite action is desired in the superstructure (refer to Topic P.1.10). Composite action is defined as dissimilar materials joined together so they behave as one structural unit. A composite bridge deck structure is one in which the deck acts together structurally with the beams to resist the applied loads. An example of composite action is a cast-in-place concrete deck joined to steel or prestressed concrete beams or a steel floor system using shear connectors (see Figures 5.2.3 and 5.2.4). A precast deck can also develop composite action through grout pockets, which engage shear connectors. Some examples of shear connectors are studs, spirals, channels, or stirrups. Shear connectors are generally welded to steel beams. In concrete beams, shear connectors are simply extended portions of shear stirrups which protrude beyond the top of the casting. Composite action does not occur until the CIP deck is placed and cured or the precast deck grout pockets have been filled and cured.

Non-Composite Member

A non-composite concrete deck is not mechanically attached to the superstructure and does not contribute to the capacity of the superstructure. A non-composite concrete deck only carries wheel loads.

Figure 5.2.3 Shear Connectors Welded to the Top Flange of a Steel Girder

Figure 5.2.4 Prestressed Concrete Beam with Shear Connectors Protruding

Steel Reinforcement

Because concrete has relatively little tensile strength, steel reinforcement is used to resist the tensile stresses in the deck. When reinforcement was first used for bridge decks, it was either round or square steel rods with a smooth finish and had a tendency to “slip” when a tension force was applied. Today, the most common reinforcement is steel deformed reinforcing bars, commonly referred to as "rebars." These bars are basically round in cross section with lugs or deformations rolled into the surface to create a mechanical bond between the bars and the concrete. Lap splices and bar development are dependent on that mechanical bond. A lap splice is the amount of overlap that is needed between two rebars to successfully have the two bars act as one. A typical lap splice length is approximately 30 bar diameters. Mechanical end anchorages or lock devices can also be used to splice rebar. Bar development is the length of embedded rebar needed to develop the design stress and varies based on material properties and bar diameter. For large bars, this length is significant. When space is limited, a mechanical hook (90° or 180° bend) is placed at the end of a bar to achieve full development.

Although concrete decks could not function efficiently without reinforcement, the corrosion of the reinforcing steel is the primary cause of deck deterioration. Since about 1970, epoxy coatings have been a common method of protecting rebars against corrosion. Less common methods of protection include galvanizing and use of stainless steel.

Primary reinforcement carries the tensile stress in a concrete deck and is located on both the top and bottom of the deck. Secondary reinforcement is temperature and shrinkage steel and is placed perpendicular to the primary reinforcement. Additional longitudinal deck reinforcement is generally placed over piers to help resist the negative moments at piers.

The inspector must be able to identify the direction of the primary reinforcement to properly evaluate any cracks in the deck. Primary reinforcement is placed perpendicular to the deck's support points. For example, the support points on a multi-beam bridge and a stringer type floor system are parallel with the direction of traffic. Therefore, the primary deck reinforcement on these deck types is perpendicular to the direction of traffic (see Figure 5.2.5). The support points on a floorbeam-only type floor system are perpendicular with the traffic flow, and the primary deck reinforcement is therefore parallel with the traffic flow. In all cases, the primary reinforcement is closer to the concrete surface.

Figure 5.2.5 Pothole Showing Deck Reinforcing Steel Perpendicular to Traffic

Primary reinforcement is generally a larger bar size than temperature and shrinkage steel. However, to improve design and construction efficiencies, concrete decks may be reinforced with the same size bar in both the top and bottom rebar mats. Reinforcement cover is generally 50 to 64 mm (2 to 2-1/2 inches) minimum for cast-in-place decks without a wearing surface, and 25 mm (1 inch) minimum for precast decks with a separate wearing surface.

5.2.3 Wearing Surfaces

Wearing surfaces are placed on top of the deck. The wearing surface protects the deck and provides a smooth riding surface. The wearing surface materials most commonly used on concrete decks are generally either special concrete mixes or asphalt concrete. Wearing surfaces are incorporated in many new deck designs and are also a common repair procedure for decks.

Concrete

There are two categories of concrete wearing surfaces: integral and overlays. An integral concrete wearing surface is cast with the deck slab, typically adding an extra 13 to 25 mm (1/2 to 1 inch) of thickness to the slab. When the wearing surface has deteriorated to the extent that rideability is affected, it is milled, leveled and replaced with an overlay.

A concrete overlay wearing surface is cast separately over the previously cast concrete deck. Some concrete wearing surfaces may have transverse grooves cut into them as a means of improving traction and preventing hydroplaning. The grooves can be tined while the concrete is still plastic or they can be diamond-sawed after the concrete has cured. There are various types of concrete overlays in use and being researched at the present time. These include:

- Low slump dense concrete (LSDC)
- Polymer/latex modified concrete (LMC)
- Internally sealed concrete
- Lightweight concrete (LWC)
- Fiber reinforced concrete (FRC)

Low slump dense concrete (LSDC) uses a dense concrete with a very low water-cement ratio (approximately 0.32). LSDC overlays were first used in the early 1960's for patches and overlays on bridges in Iowa and Kansas (hence the common term "Iowa Method"). The original overlays were 31 mm (1¼ inches) thick, but now a 50 mm (2-inch) minimum is specified. This type of overlay is generally used because it cures rapidly and has a low permeability. The low permeability resists chloride penetration, while the fast curing decreases the closure period. Low slump dense concrete overlays are placed mainly in locations where deicing salts are used. Surface cracking is a problem in areas where the freeze/thaw cycle exists. The number of applications of deicing salts also plays a role in the deterioration of LSDC overlays. Higher strength dense concrete has been used in the recent past, and results have shown that LSDC overlaid bridge decks will require resurfacing after about 25 years of service, regardless of the concrete deck deterioration caused by steel reinforcement corrosion.

Polymer/latex modified concrete overlay involves the incorporation of polymer emulsions into the fresh concrete. The polymer emulsions have been polymerized prior to being added to the mixture. This is commonly known as latex-modified concrete (LMC). LMC is conventional Portland cement concrete with the addition of approximately 15 percent latex solids by weight of the cement. The typical thickness of 31 mm (1¼ inches) is used for LMC.

The primary difference between the LSDC and the LMC overlays is that low slump concrete uses inexpensive materials but is difficult to place and requires special finishing equipment. Conversely, latex-modified concrete utilizes expensive materials but requires less manpower and is placed by conventional equipment. The performance of LMC has generally been satisfactory, although in some cases, extensive map cracking and debonding have been reported. The causes for this are likely the improper application of the curing method, application under high temperature, or shrinkage due to high slump.

Internally sealed concrete overlays consist of the incorporation of fusible polymeric particles into a concrete mix. After the concrete has cured, the additive is then fused to it. This system, in effect, seals the concrete from moisture and chemicals.

Lightweight concrete (LWC) overlays use concrete with lightweight aggregates and a higher entrained air content. This produces an overlay of approximately 1282 to 1602 kg/m^3 (80 to 100 pcf) compared to 2243 to 2403 kg/m^3 (140 to 150 pcf) for conventional concrete. This type of overlay has a reduced dead load compared to a traditional concrete overlay. Lightweight concrete is also used for cast-in-place and precast decks.

Fiber reinforced concrete (FRC) overlays using Portland cement and metallic, glass, plastic, or natural fibers are becoming a popular solution to bridge deck surface problems. This type of reinforcement strengthens the tension properties in the concrete, and tests have shown that FRC overlays can stop a deck crack from reflecting through the overlay. This type of overlay is gaining acceptance but is still in the research stage.

Asphalt

The most common overlay material for concrete decks is asphalt. Asphalt overlays generally range from 25 mm (1 inch) up to 63 mm (2½ inches), depending on the severity of the repair and the load capacity of the superstructure. When asphalt is placed on concrete, a waterproof membrane may be applied first to protect the reinforced concrete from the adverse effects of water borne deicing chemicals, which pass through the permeable asphalt concrete layer. Not all attempts at providing a waterproof membrane are successful.

5.2.4 Protective Systems

With increasing research, the uses of protective systems are increasing the life of reinforced concrete bridge decks. Most reinforced concrete bridge decks need repair years before the other components of the bridge structure. Therefore protecting the bridge deck from contamination and deterioration is gaining importance.

Sealants

Reinforced concrete deck sealants are used to stop chlorides from contaminating the steel reinforcement. These sealants are generally pore sealers or hydrophobing agents, and their performance is affected by environmental conditions, traffic wear, penetration depth of the sealer, and ultraviolet light.

Boiled linseed oil is a popular sealant that is used to cure or seal a concrete deck.

It is applied after the concrete gains the appropriate amount of strength. This material resists water and the effects of deicing agents.

Elastomeric membranes are another approach when sealing a concrete bridge deck. This type of sealant is mixed on site and cures to a seamless viscous waterproof membrane. It is generally applied prior to placing an asphalt overlay.

Epoxy Coated Reinforcement Bars

Due to the understanding of steel reinforcement corrosion and its effects on the concrete deck, an epoxy coating is often used on all steel reinforcement placed in concrete bridge decks to prevent steel corrosion. The epoxy coating is resistant to chemicals, water, and atmospheric moisture. Epoxies utilize an epoxy polymer binder that forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation, while curing entails a chemical reaction between the coating components.

Galvanized Reinforcement Bars

Another method of protecting steel reinforcement is by galvanizing the steel. This also slows down the corrosion process and lengthens the life of the reinforced concrete slab. Galvanizing is achieved by coating the bare steel reinforcement with zinc. The two unlike metals form an electrical current between them, and one metal virtually stops its corrosion process while the other's accelerates due to the electrical current. In this situation, the steel stops corroding while the zinc has accelerated corrosion.

Stainless Steel Reinforcement Bars

The corrosion process is negligible when stainless steel reinforcement is used.

Fiberglass Reinforced Polymer (FRP) bars

Fiberglass Reinforced Polymer (FRP) bars for concrete reinforcement have the advantage of resistance to corrosion. They are also lightweight, weighing about one-quarter the weight of an equivalent size steel bar.

Cathodic Protection of Reinforcement Bars

Cathodic Protection is sometimes used on decks with black bare steel reinforcement (not epoxy coated). Steel reinforcement corrosion can also be slowed down by cathodic protection. Corrosion of steel reinforcing bars in concrete occurs by an electrical process in a moist environment at the steel surface. During corrosion, a voltage difference (less than 1 volt) develops between rebars or between different areas on the same rebar. Electrons from the iron in the rebar are repelled by the negative anode area of the rebar and attracted to the positive cathode area. This electron flow constitutes an electrical current that is necessary for the corrosion process. Corrosion occurs only at the anode, where the electrons from the iron are given up.

By cathodic protection, this electrical current is reversed, slowing or stopping corrosion. By the impressed current method, an electrical DC rectifier supplies electrical current from local electrical power lines to a separate anode embedded in the concrete. The anode is usually a wire mesh embedded just under the concrete surface. Another type of anode consists of an electrically conductive coating applied to the concrete surface. The wires from the rectifier are embedded in the coating at regular intervals.

When the impressed current enters the mesh or coating anode, the voltage on the rebars is reversed, turning the entire rebar network into a giant cathode. Since natural corrosion occurs only at the anode, the rebars are protected.

The natural corrosion process is allowed to proceed by electrons leaving the iron atoms in the anode. With impressed current cathodic protection, however, the electrons are supplied from an external source, the DC rectifier. Thus, the artificial anode mesh or coating is also spared from corrosion.

Waterproofing Membrane

There are two types of bridge deck waterproofing membrane systems.

- Self-adhering membrane – is a high strength polyester reinforced membrane with a rubber/bitumen compound, which is cold applied. A layer of bituminous base and wearing course is then applied over the membrane.
- Liquid waterproofing membrane – is a two-component compound, which is simply mixed on site to produce a viscous seamless rubber/bitumen liquid that cures to an elastomeric waterproof membrane.

These systems are used to retard reflective cracking and provide waterproofing.

5.2.5 Overview of Common Defects

Common concrete deck defects are listed below. Refer to Topic 2.2 for a detailed description of these defects:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion
- Prestressed concrete deterioration

5.2.6 Inspection Procedures and Locations

Procedures

Visual

The inspection of concrete decks for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination of a deck with a hammer can be a tedious operation. In most cases, a chain drag is used. A chain drag is made of several sections of chain attached to pipe that has a handle attached to it. The inspector drags this across a deck and makes note of the resonating sounds. A chain drag can usually cover about a 915 mm (3-feet) wide section of deck at a time (see Figure 5.2.6).

Figure 5.2.6 Inspector Using a Chain Drag

If the inspector deems it necessary, core samples can be taken from the deck and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete bridge decks are caused by corrosion of the rebar. When the deterioration of a concrete deck progresses to the point of needing rehabilitation, an in-depth inspection of the deck is required to determine the extent, cause, and possible solution to the problem. Several techniques and methods are available.

Advanced Inspection Techniques

In addition, several advanced techniques are available for concrete inspection. Nondestructive methods, described in Topic 13.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Delamination detection machinery
- Electrical methods
- Electro magnetic 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 13.2.3, include:

- Core Sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Both the top and bottom surfaces of concrete decks should be inspected for cracking, scaling, spalling, corroding reinforcement, chloride contamination, delamination, and full or partial depth failures. In all instances, it is helpful if the inspector has available the previous inspection report so that the progression of any deterioration can be noted. This provides a more meaningful inspection. Refer to Topic 2.2 for a detailed description of concrete defects.

For concrete deck inspections, special attention should be given to the following locations:

- **Areas exposed to traffic** - examine for surface texture and wheel ruts due to wear. Check cross-slopes for uniformity.

- **Areas exposed to drainage** - investigate for scaling, delamination, and spalls.

- **Bearing and shear areas** where the concrete deck is supported - check for spalls and crushing.

- **Shear key joints** between precast deck panels - inspect for leaking joints, cracks, and other signs of independent action.

- **Anchorage zones** of precast slab tie rods - check for deteriorating grout pockets or loose lock-off devices. If a previous inspection report is available, this should be used by the inspector so that the progression of any deterioration can be noted.

- **Top of the slab** over the supports - examine for flexure cracks.

- **Bottom of the slab** between the supports - check for flexure cracks (see Figure 5.2.7).

- **Asphalt overlays** - if present, they should be inspected. Cracks, delaminations, and spalls are to be noted. Often water penetrates overlays and then penetrates into the structural deck. Asphalt overlays prevent visual inspection of the top surface of the deck.

- **Stay-in-place forms** - investigate for deterioration and corrosion of the forms, often indicating contamination of the concrete deck; these forms can retain moisture and chlorides which have penetrated full depth cracks in the deck (see Figure 5.2.8).

- **Cathodic protection** - during the bridge inspection, check that all visible electrical connections and wiring from the rectifier to the concrete structure are intact. If cathodic protection appears not to be working, notify maintenance personnel. Some agencies that use cathodic protection have specialized inspection/maintenance crews for these types of bridge decks.

Figure 5.2.7 Underside View of Longitudinal Deck Crack

Figure 5.2.8 Deteriorated Stay-in-Place Form

5.2.7 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete decks. 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 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 deck. Rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 for additional details about the NBIS rating guidelines. 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 - Pontis)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the top of deck and the underside. 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 concrete deck.

In an element level condition state assessment of a concrete deck, the AASHTO CoRe element is one of the following, depending on the riding surface:

Element No.	**Description**
012	Concrete Deck – Bare
013	Concrete Deck – Unprotected with AC Overlay
014	Concrete Deck – Protected with AC Overlay
018	Concrete Deck – Protected with Thin Overlay
022	Concrete Deck – Protected with Rigid Overlay
026	Concrete Deck – Protected with Coated Bars
027	Concrete Deck – Protected with Cathodic System

The unit quantity for these elements is "each", and the entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2). Condition state 1 is the best possible rating. The inspector must know the total slab surface area in order to calculate a percent deterioration and fit into a given condition state description. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For structural cracks in the surface of bare slabs, the "Deck Cracking" Smart Flag, Element No. 358, can be used and one of four condition states assigned. Do not use Smart Flag, Element No. 358, if the bridge deck/slab has any overlay because the top surface of the structural deck is not visible. For concrete defects on the underside of a slab element, the "Soffit" Smart Flag, Element No. 359, can be used and one of five condition states assigned.

This page intentionally left blank

Table of Contents

Section 5

Inspection and Evaluation of Decks

Topic 5.3 Steel Decks

5.3.1 Introduction

Steel decks are found on many older bridges and moveable bridges. Their popularity grew until concrete decks were introduced. Today, steel bridge decks have various advantages and disadvantages, depending on the application, and are mainly used for bridge deck rehabilitation or for very long spans.

5.3.2 Design Characteristics

Steel bridge decks are mainly used when weight is a major factor. The weight of a steel deck per unit area is less than that of concrete. This weight reduction of the deck means the superstructure and substructure can carry more live load. The trade-off for this weight savings is that water is permitted to pass through, which corrodes the superstructure. Steel decks are sometimes filled with concrete to prevent the water from passing through. The four basic types of steel decks are:

- Orthotropic decks
- Buckle plate decks
- Corrugated steel flooring
- Grid decks

Orthotropic Decks

An orthotropic deck consists of a flat, thin steel plate stiffened by a series of closely spaced longitudinal ribs at right angles to the floor beams. The deck acts integrally with the steel superstructure. An orthotropic deck becomes the top flange of the entire floor system. Orthotropic decks are occasionally used on large bridges (see Figure 5.3.1).

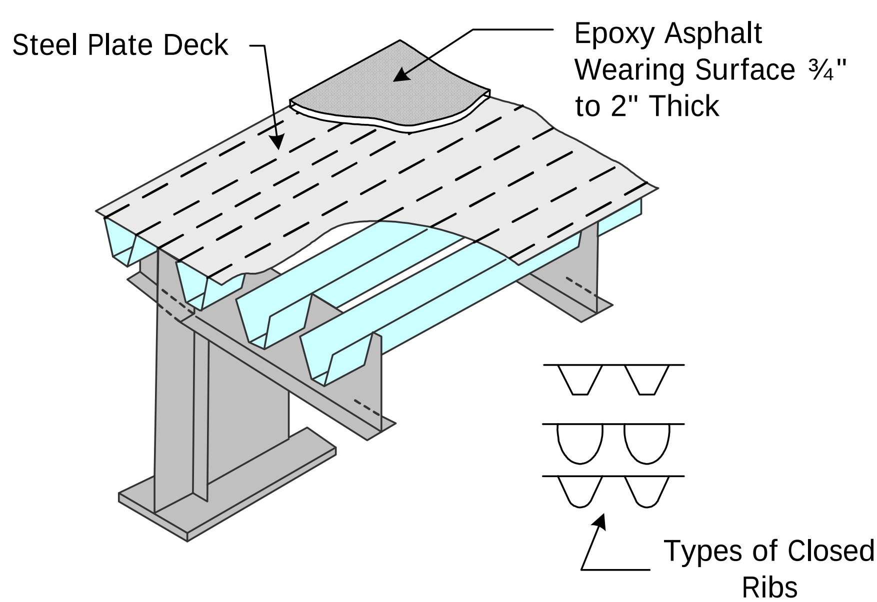


Figure 5.3.1 Orthotropic Bridge Deck

Buckle Plate Decks

Buckle plate decks are found on older bridges. They consist of steel plates attached to the floor system which support a layer of reinforced concrete (see

Figure 5.3.2). The plates are concave or "dished" with drain holes in the center. All four sides are typically riveted to the floor system. Buckle plate decks serve as part of the structural deck and as the deck form. They are obsolete, however, and are no longer used today.

Figure 5.3.2 Underside View of Buckle Plate Deck

Corrugated Steel Flooring

Corrugated steel flooring is popular because of its light weight and high strength. This deck consists of corrugated steel planks covered by a layer of asphalt (see Figure 5.3.3). The planks are set upon the stringers so that the corrugations run perpendicular to the length of the bridge. Corrugations are smaller than stay-in-place (SIP) forms, but the steel is thicker, ranging from 3 mm (0.1 inch) to 5 mm (0.18 inch). The steel planks are welded in place to steel stringers. In the case of timber stringers, the planks are attached by lag bolts. The corrugations are filled with bituminous pavement, and then a wearing surface is applied. This deck is used primarily for the rehabilitation of small bridge decks.

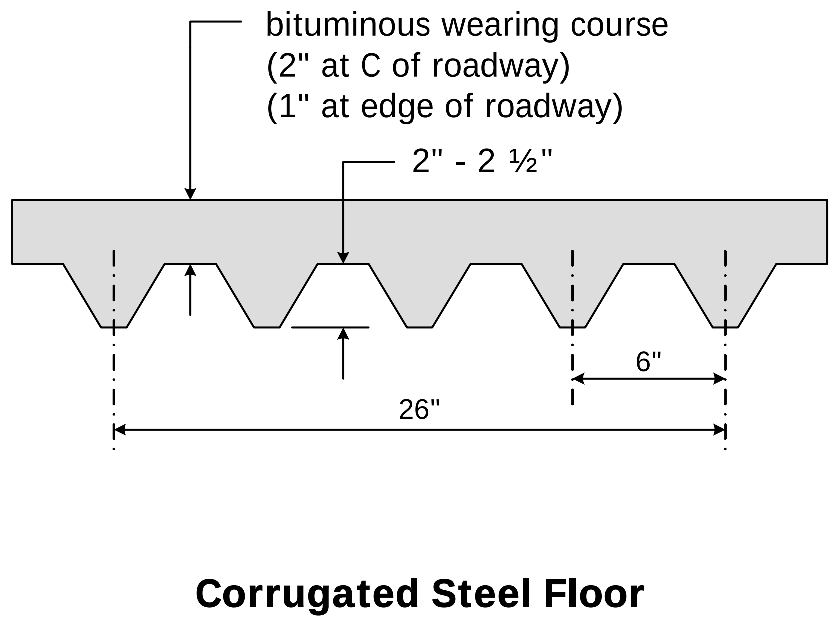


Figure 5.3.3 Sectional View of Corrugated Steel Floor

Grid Decks

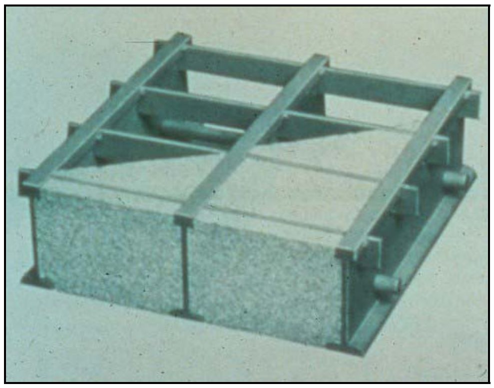
Grid decks are probably the most common type of steel deck because of their light weight and high strength. They are commonly welded units, which may be open or filled with concrete.

Open decks are lighter than concrete-filled decks, but they are vulnerable to corrosion since they are continually exposed to weather, debris, and traffic. Another disadvantage of open decks is that they allow dirt and debris to fall onto the supporting members.

Concrete-filled grid decks offer protection for the floor system against water, dirt, debris, and deicing chemicals that usually pass directly through open grid decks. They can be partially-filled or fully-filled.

Partially-filled decks are grid decks which have been partially filled with lightweight concrete. This provides a reduction in the dead load and the protection of a concrete-filled floor system. Grid decks are often found on rehabilitated bridges. Their low weight reduces the dead load on a rehabilitated bridge, and their easy installation reduces the time that the bridge must be closed for repairs.

Fully-filled decks are grid decks that have been completely filled with concrete (see Figure 5.3.4). These decks provide the maximum load carrying capacity. Form pans are welded within the grid to hold the concrete. Filled decks often contain rebars for extra strength.



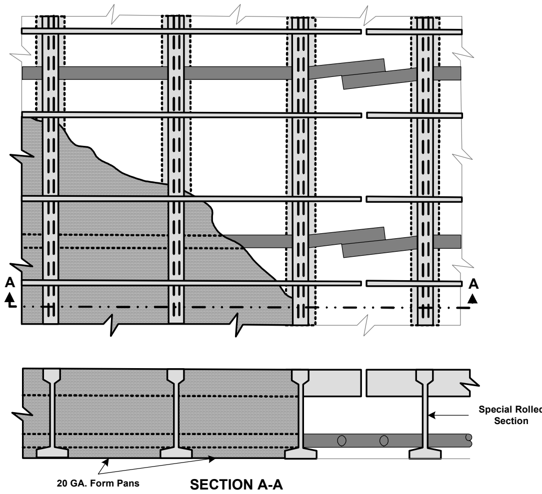


Figure 5.3.4 Schematic of Concrete Filled Grid Deck

The three types of grid decks include:

- Welded grid decks
- Riveted grate decks
- Exodermic decks

Welded Grid Decks

Welded grid decks have their components welded together. These components consist of bearing bars, cross bars, and supplementary bars (see Figure 5.3.5).

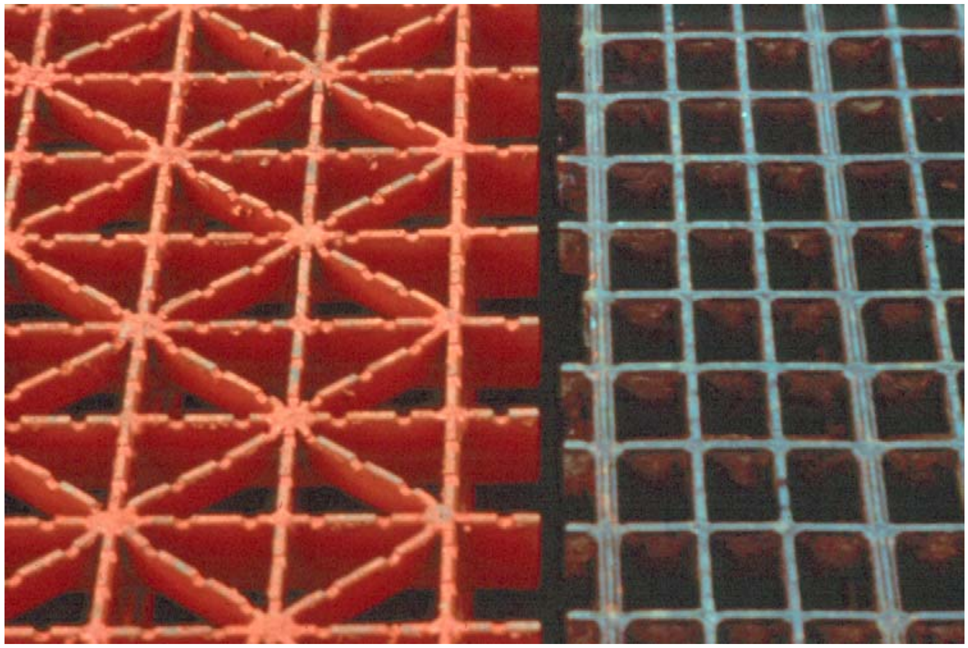


Figure 5.3.5 Various Patterns of Welded Steel Grid Decks

The bearing bars support the grating. Bearing bars are laid on top of the stringers perpendicularly and are then field-welded or bolted to the stringers. These bars are also referred to as the primary or main bars.

The cross bars are grating bars that are laid perpendicular on top of the bearing bars. They may be either shop- or field-welded to the grating system. Cross bars, also referred to as secondary bars or distribution bars, are generally serrated for improved traction.

The supplementary bars are grating bars that run parallel to the bearing bars. They are also shop- or field-welded to the grating system. Not all grating systems have supplementary bars. These bars are also referred to as tertiary bars.

Riveted Grid Decks

A riveted grid deck is made up of bearing bars, crimp bars, and intermediate bars and can either be fully or partially filled with concrete to improve the load carrying capacity of the deck (see Figure 5.3.6).

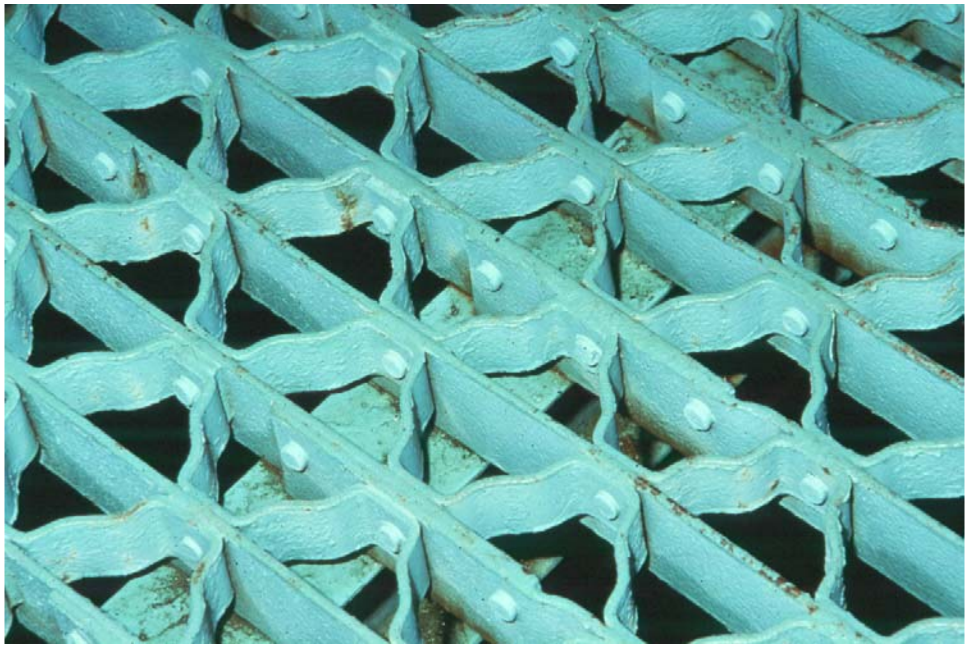


Figure 5.3.6 Riveted Grid Deck

Bearing bars run perpendicular to the stringers and are attached to the stringers by either welds or bolts. They are similar to the bearing bars in welded grates.

Crimp bars are riveted to the bearing bars to form the grating.

Intermediate bars run parallel to the bearing bars but, in order to reduce the weight of the deck, are not as long. The crimp bars are riveted to intermediate bars and may not be present on all riveted grate decks.

Exodermic Decks

Exodermic decks are a newer type of bridge deck in which a reinforced concrete slab is placed on top of, and is made composite with, a steel grid (see Figure 5.3.7). Composite action is achieved by studs that extend into the reinforced concrete slab and are welded to the grid deck below. Galvanized sheeting is used as a bottom form to keep the concrete from falling through the grid holes. Exodermic decks generally weigh 50% to 65% lighter than precast reinforced concrete decks.

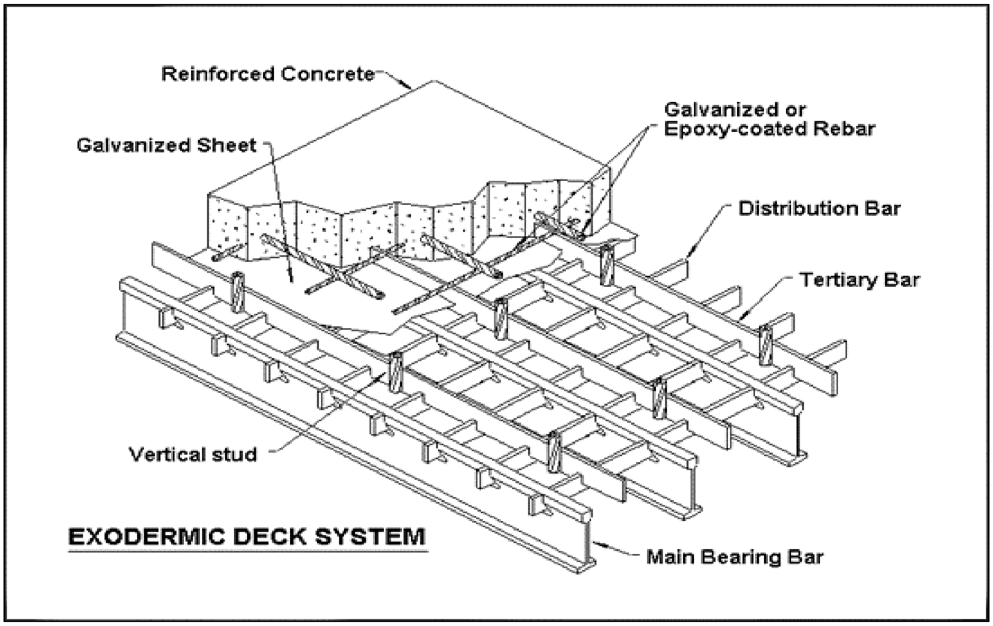


Figure 5.3.7 Schematic of Exodermic Composite Profile

5.3.3 Wearing Surfaces

Wearing surfaces protect the steel deck, provide an even riding surface, and reduce the water on the deck and superstructure. Wearing surfaces for steel decks can consist of:

- Serrated steel
- Concrete
- Asphalt

Studs can be welded to steel decks for skid resistance.

Serrated Steel

Open grid decks usually have serrated edges on the grating (see Figure 5.3.5). Designed not to wear, these serrations make up the riding surface of an open grid deck.

Concrete

Acting as the wearing surface, fully and partially filled grid decks have a layer of concrete flush with the top of the grids. This concrete wearing surface and the concrete used to fill the grids are generally poured at the same time. Different types of concrete wearing surfaces are listed and described in Topic 5.2.3. In the case of an exodermic bridge deck, the wearing surface is part of a reinforced deck made composite with an unfilled steel grid system and attached by means of vertical studs.

Asphalt

Steel plate decks, such as orthotropic decks, typically have a layer of asphalt as the wearing surface. Asphalt overlays generally range from 25 mm (1 inch) up to 63 mm (2½ inches), depending on the severity of the repair and the load capacity of the superstructure. Corrugated steel plank decks also have asphalt wearing surfaces.

An epoxy asphalt polymer concrete also is used for orthotropic bridge deck wearing surfaces. Unlike conventional asphalt mixes, epoxy asphalt polymer concrete will not melt after it has cured due to having a thermoset polymer in the mix. This polymer is different than thermoplastic polymer, which is used in conventional asphalt mixes. Therefore epoxy asphalt polymer concrete is used when strength and elastic composition are important.

5.3.4 Protective Systems

Paints

Paints provide protection from moisture, oxygen, and chlorides. Usually three coats of paint are applied. The first coat is the primer, the next is the intermediate coat, and the final coat is the topcoat. Various types of paint are used, such as oil/alkyd, vinyl, epoxy, urethane, zinc-rich primer, and latex paints.

Galvanizing

Another method of protecting steel decks is by galvanizing the steel. This also slows down the corrosion process and lengthens the life of the steel deck. This occurs by coating the bare steel with zinc. The two unlike metals form an electrical current between them and one metal virtually stops its corrosion process while the other's accelerates due to the electrical current. In this situation, the steel stops corroding, while the zinc has accelerated corrosion.

There are two methods of galvanizing steel decks (shop applied and field applied). Hot-dipping the steel deck members usually takes place at a fabrication shop prior to the initial placement of the steel deck. When sections of the deck are too large or when maintenance painting is to take place, the zinc-rich-primers can be applied in the field. The zinc paint must be mixed properly, and the surface must be prepared correctly.

Overlay

Another protective system for steel decks is the overlay material itself. The overlay covers the steel to create a barrier from corrosive agents. Overlays slow down the deterioration process for steel decks.

Epoxy Coating

Epoxy coating steel grates is another means of protecting the steel decking. This is a rare type of protective coating for steel bridge decks, but there are a limited number still in service.

5.3.5 Overview of Common Defects

Some of the common steel deck defects are listed below. Refer to Topic 2.3 to review steel defects in detail.

- Bent, damaged, or missing members
- Corrosion
- Fatigue cracks
- Other stress-related cracks

5.3.6 Inspection Procedures and Locations

Steel decks should be visually inspected for broken welds, failed fasteners, broken grids, and section loss (see Figure 5.3.8).

Procedures

Visual

The inspection of steel decks for corrosion, section loss, buckling, and cracking is primarily a visual activity. Reference Topic 2.3 for a more detailed explanation of visual inspection procedures.

Physical

Once the defects are identified visually, physical procedures must be used to verify the extent of the defect. For steel members, the main physical inspection procedures involve an inspection hammer and wire brush. Corrosion results in loss of member material. This partial loss of cross section due to corrosion is known as section loss. Section loss may be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or a D-meter, should be used to measure the remaining section of steel. The inspector must remove all corrosion products (rust scale) prior to making measurements.

The inspector should measure the bridge members to verify that the sizes recorded in the plans or inspection report are accurate. If incorrect member sizes are used, then any load rating analysis for safe load capacity of the bridge is worthless.

Advanced Inspection Techniques

In addition, several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Acoustic emissions testing
- Computer Programs
- Computer tomography
- Corrosion sensors
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 13.3.3, include:

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

Locations

The primary locations for steel deck inspection include:

- **Bearing areas** - check for cracked welds or broken fasteners, which connect the steel deck to the supporting floor system.

- **Primary bearing bars** - inspect for broken, cracked, or missing bars.

- **Tension areas** - on steel grid decks, check positive and negative moment regions of the primary bearing bars. Look for damage such as broken, cracked, or missing bars.

- **Areas exposed to drainage** - check areas where drainage can lead to corrosion.

- **Corrugated flooring** - check between the support points for section loss due to corrosion.

- **Check for slipperiness** on steel grid decks caused by excessive wear.

- **Section loss** - in areas where corrosion is evident, all scale should be removed with an inspection hammer in order to evaluate the amount of remaining material.

- **Connections** - examine for broken connections, and listen for rattles as traffic passes over the deck.

- **Filled grid decks** - inspect for grid expansion at joints and bridge ends, often caused by corrosion.

- **Corrosion** - on corrugated flooring, check between the support points for section loss due to corrosion. In areas where corrosion is evident, all scale should be removed with an inspection hammer in order to evaluate the amount of remaining material. Document the location and condition of any repair plates.

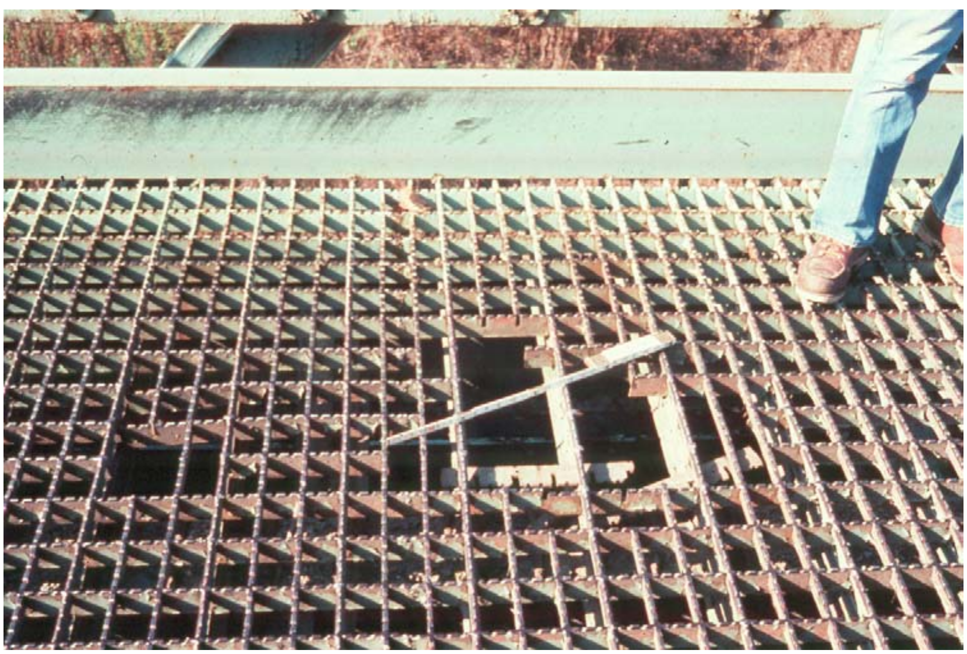


Figure 5.3.8 Broken Members of an Open Steel Grid Deck

5.3.7 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of steel decks. 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 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 deck. Rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 for additional details about the NBIS rating guidelines. 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 - Pontis)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the top of deck and the underside. 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 deck, the AASHTO CoRe element is one of the following, depending on the riding surface:

Element No.	Description
028	Steel Deck – Open Grid
029	Steel Deck – Concrete Filled Grid
030	Steel Deck – Corrugated/ Orthotropic

The unit quantity for these elements is "each", and the entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2). Condition state 1 is the best possible rating. The inspector must know the total deck surface area in order to calculate a percent deterioration and fit into a given condition state description. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For connections of steel decks showing rust packing between steel plates, the "Pack Rust" Smart Flag, Element No. 357, can be used and one of four condition states assigned.

This page intentionally left blank

Table of Contents

Section 5
Inspection and Evaluation of Decks

Topic 5.4 Deck Joints, Drainage Systems, Lighting and Signs

5.4.1 Function of Deck Joints, Drainage Systems, Lighting and Signs

Deck Joints

The deck joint is a very important part of a bridge. The primary function of deck joints is to accommodate the expansion and contraction of the deck and superstructure. In most bridges, the deck joints must accommodate this movement and prevent runoff from reaching bridge elements below the surface of the deck. In addition, the deck joint provides a smooth transition from the approach roadway to the bridge deck. The deck joint must be able to withstand all possible weather extremes in a given area. It must do all of this without compromising the ride quality of vehicles crossing the bridge.

Drainage Systems

The purpose of a drainage system is to remove water and all hazards associated with it from the structure. The drainage system should also require as little maintenance as possible and be located so that it does not cause hazards.

Lighting and Signs

Lighting serves various functions on bridge structures, depending on location and color. Highway lighting is used to increase visibility on a bridge structure. Traffic signal lighting controls traffic on a structure. Aerial obstruction lighting warns aircrafts of a hazard around and below the lights. Navigational lighting is used for the safe control of waterway traffic under a bridge structure. Finally, sign lighting ensures proper visibility for traffic signs.

Typical signs that are present on or near bridges provide regulatory (e.g., speed limits) information and advisory (e.g., clearance warnings) information. Such signs serve to inform the motorist about bridge or roadway conditions that may be hazardous.

5.4.2 Components of Deck Joints, Drainage Systems, Lighting and Signs

Deck Joints

Deck joints should not be confused with construction joints. While deck joints are used primarily to facilitate expansion and contraction of the deck and superstructure, construction joints mark the beginning or end of concrete placement sections during the construction of the bridge deck. The two major categories of deck joints are open joints and closed joints.

Open Joints

Open joints allow water and debris to pass through the joint. The two types of open joints are as follows:

- Formed joints
- Finger plate joints

Formed Joints

Formed joints are little more than a gap between the bridge deck and the abutment backwall or, in the case of a multiple span structure, between adjacent deck sections. They are usually found on very short span bridges where expansion is minimal. The formed joint is usually unprotected, but the deck slab and backwall can be armored with steel angles. Formed joints are common on short span bridges with concrete decks (see Figures 5.4.1 and 5.4.2).

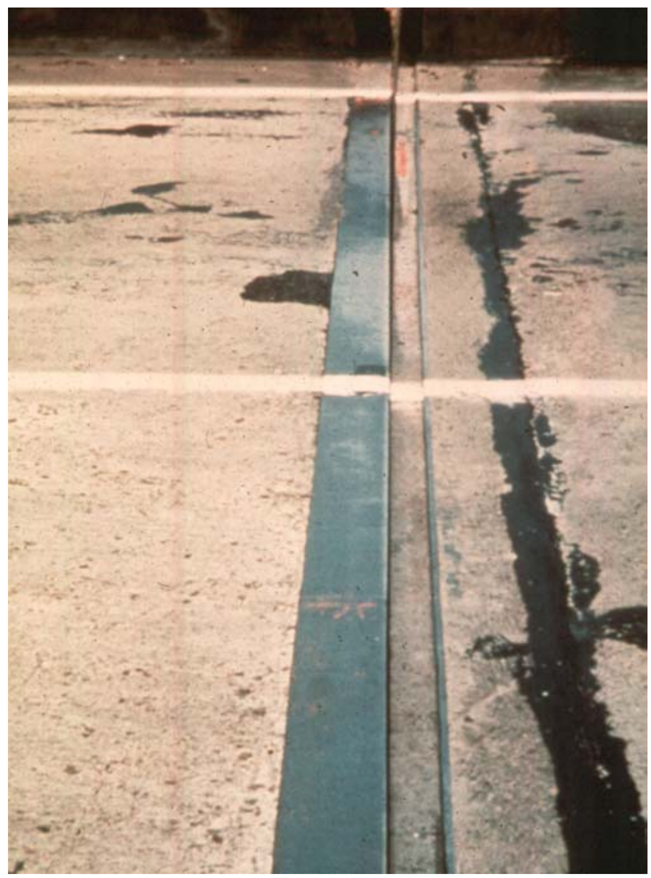


Figure 5.4.1 Formed Joint

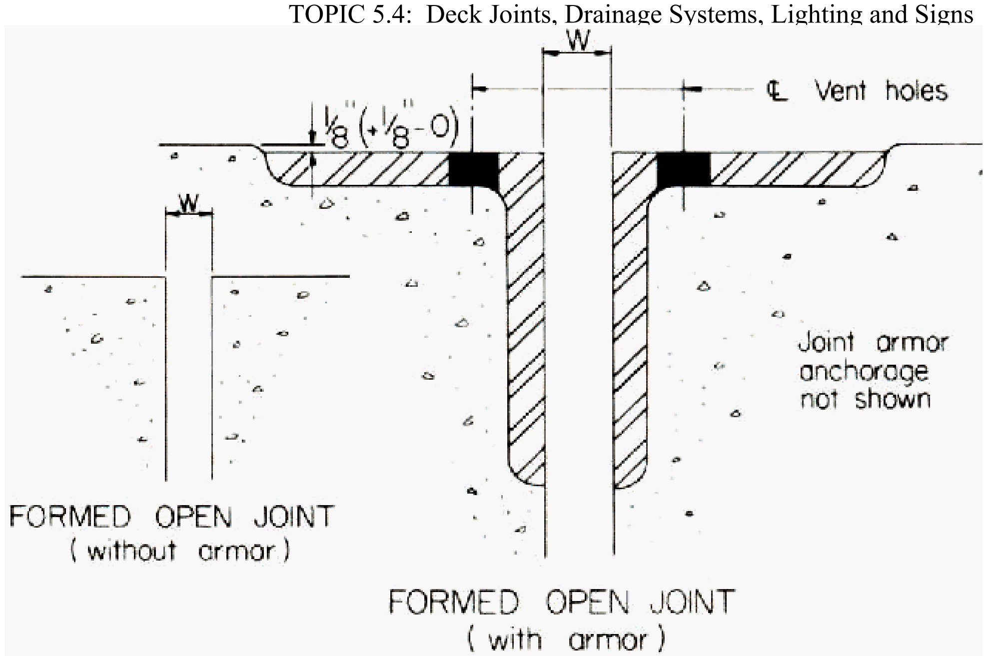


Figure 5.4.2 Cross Section of a Formed Joint

Finger Plate Joints

A finger plate joint, also known as a tooth plate joint or a tooth dam, consists of two steel plates with interlocking fingers. These joints are usually found on longer span bridges where greater expansion is required. The two types of finger plate joints are cantilever finger plate joints and supported finger plate joints.

The cantilever finger plate joint is used when relatively little expansion is required. The fingers on this joint cantilever out from the deck side plate and the abutment side plate. The supported finger plate joint is used on longer spans. The fingers on this joint have their own support system in the form of transverse beams under the joint. Some types of finger plate joints are segmental, allowing for maintenance and replacement if necessary. Finger plate joints are used to accommodate movement from 100 to over 600 mm (4 to over 24 inches) (see Figures 5.4.3 through 5.4.5).

Troughs are sometimes placed under open finger plate joints. Their purpose is to direct water that passes through the joint away from the superstructure, bearings and substructure.

Figure 5.4.3 Finger Plate Joint

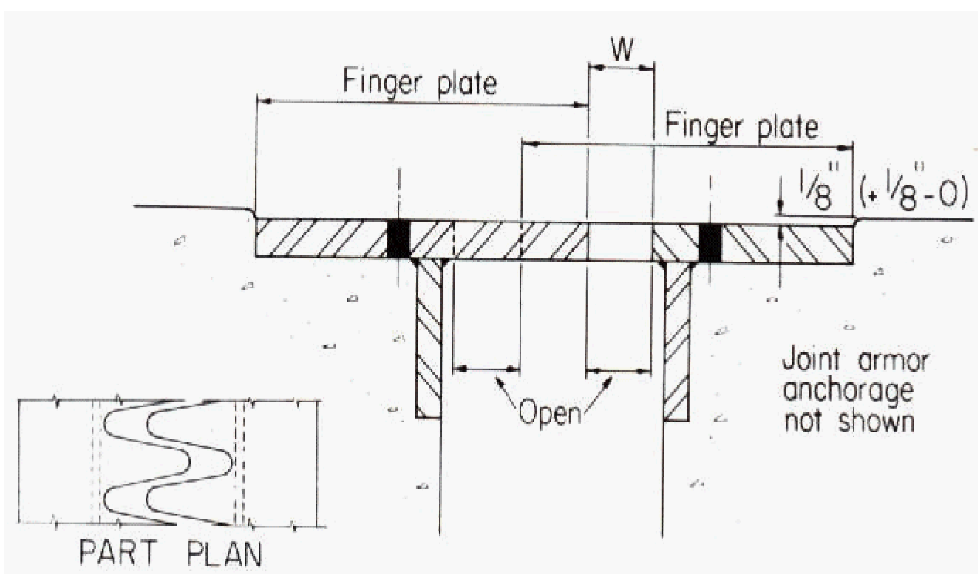


Figure 5.4.4 Cross Section of a Cantilever Finger Plate Joint

Figure 5.4.5 Supported Finger Plate Joint

Closed Joints

Closed joints are designed so that water and debris do not pass through them. This protects the superstructure and substructure members directly below the joint from the effects of water and debris buildup. There are many types of closed joints, including the following:

- Poured joint seal
- Compression seal
- Cellular seal
- Sliding plate joint
- Prefabricated elastomeric seal
- Modular elastomeric seal
- Asphaltic expansion joint

Poured Joint Seal

A poured joint seal is made up of two materials: a base and a poured sealant. The base consists of a preformed expansion joint filler. The top of this material is 25 to 50 mm (1 to 2 inches) from the top of the deck. The remaining joint space consists of the poured sealant that is separated from the base by a backer rod or a bond breaker. Since the poured joint seal can only accommodate a movement of about 6 mm (1/4 inch), it is usually found on short span structures.

Compression Seal

A compression seal consists of neoprene formed in a rectangular shape with a honeycomb cross section (see Figure 5.4.6). The honeycomb design allows the compression seal to fully recover after being distorted during bridge expansion and contraction. It is called a compression seal because it functions in a partially compressed state at all times. Compression seals can have steel angle armoring on the deck and backwall. In some cases, the deck joint is saw cut to accept the installation of the compression seal. In such cases, no armoring is provided. These seals come in a variety of sizes and are often classified by their maximum movement capacity. A large compression seal can accommodate a maximum movement of approximately 50 mm (2 inches).

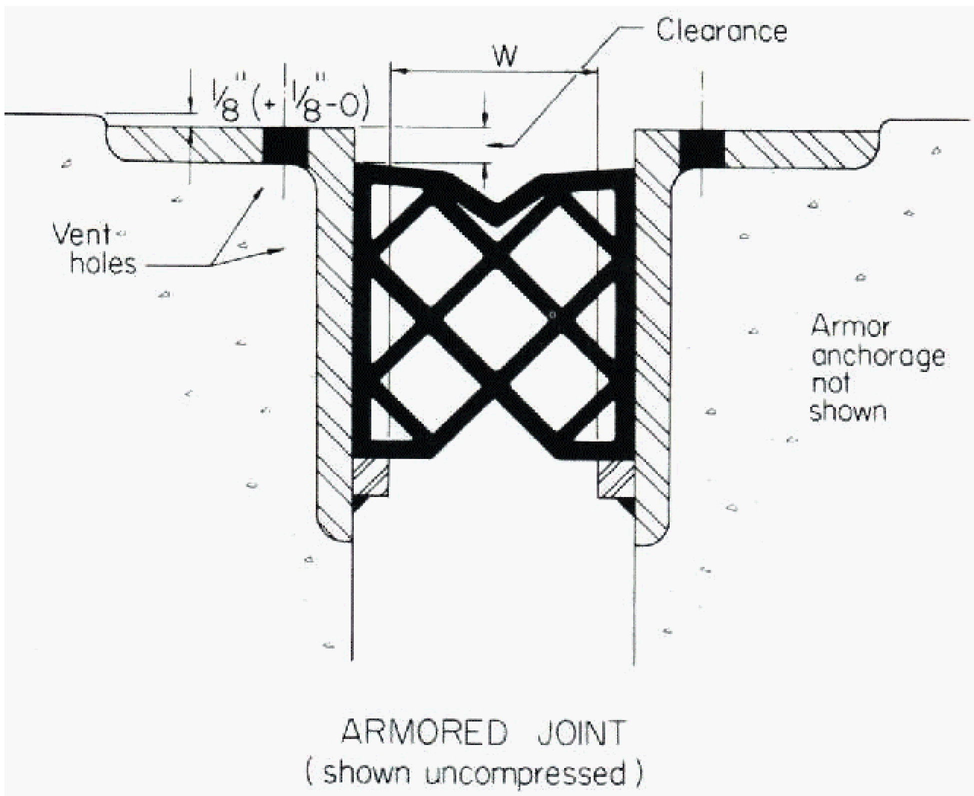


Figure 5.4.6 Cross Section of a Compression Seal with Steel Angle Armoring

Cellular Seal

The cellular seal is similar to the compression seal, and its armoring is almost identical. However, they differ in the type of material used to seal the joint. Unlike the compression seal, the cellular seal is made of a closed-cell foam that allows the joint to move in different directions without losing the seal. This foam allows for expansion and contraction both parallel and perpendicular to the joint. The parallel movement is referred to as racking and occurs during normal expansion and contraction of a curved structure or a bridge on a skew.

Sliding Plate Joint

A sliding plate joint is composed of two plates sliding on top of each other.

Although classified as a closed joint, the sliding plate joint is usually not watertight. In an attempt to seal the joint, an elastomeric sheet is sometimes used. This sheet is attached between the plates and the joint armoring. The resulting trough serves to carry water away to the sides of the deck (see Figure 5.4.7). The sliding plate joint can accommodate a maximum movement of approximately 100 mm (4 inches).

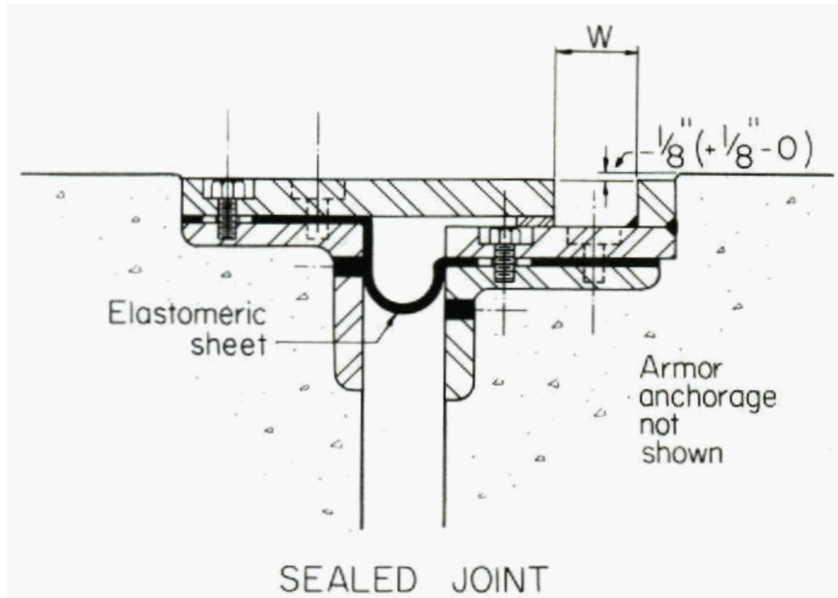


Figure 5.4.7 Cross Section of a Sliding Plate Joint

Prefabricated Elastomeric Seal

Prefabricated elastomeric seals are frequently proprietary products and include three basic types:

- Plank seal
- Sheet seal
- Strip seal

A plank seal consists of steel reinforced neoprene that supports vehicular wheel loads over the joint. This type of seal is bolted to the deck and is capable of accommodating movement ranges from 50 to 330 mm (2 to 13 inches) (see Figure 5.4.8).

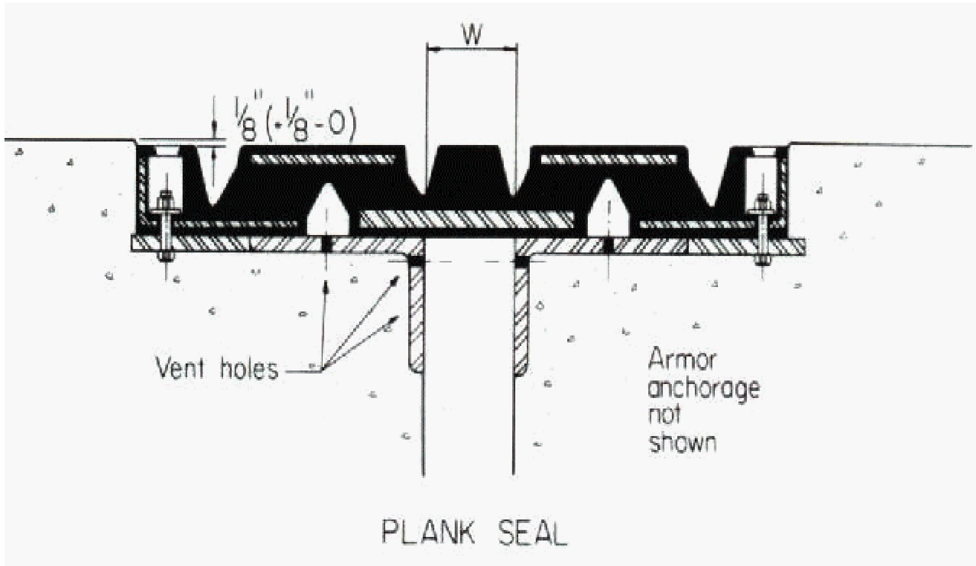


Figure 5.4.8 Plank Seal

A sheet seal consists of two blocks of steel reinforced neoprene. A thin sheet of neoprene spans the joint and connects the two blocks. This joint can accommodate a maximum movement of approximately 100 mm (4 inches) (see Figure 5.4.9).

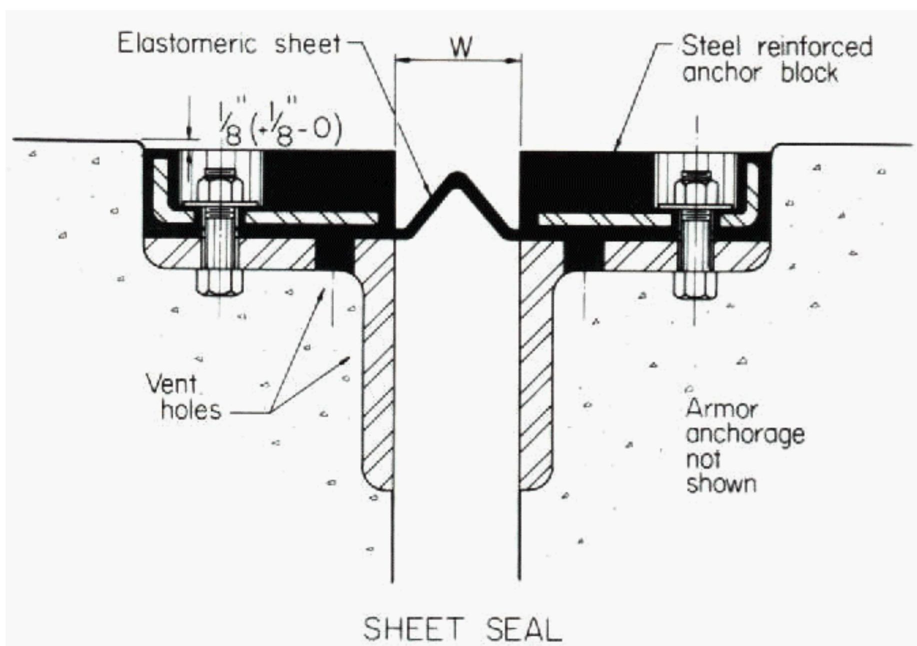


Figure 5.4.9 Sheet Seal

A strip seal consists of two slotted steel anchorages cast into the deck and backwall. A neoprene seal fits into the grooves to span the joint. This joint can accommodate a maximum movement of approximately 100 mm (4 inches) (see Figure 5.4.10).

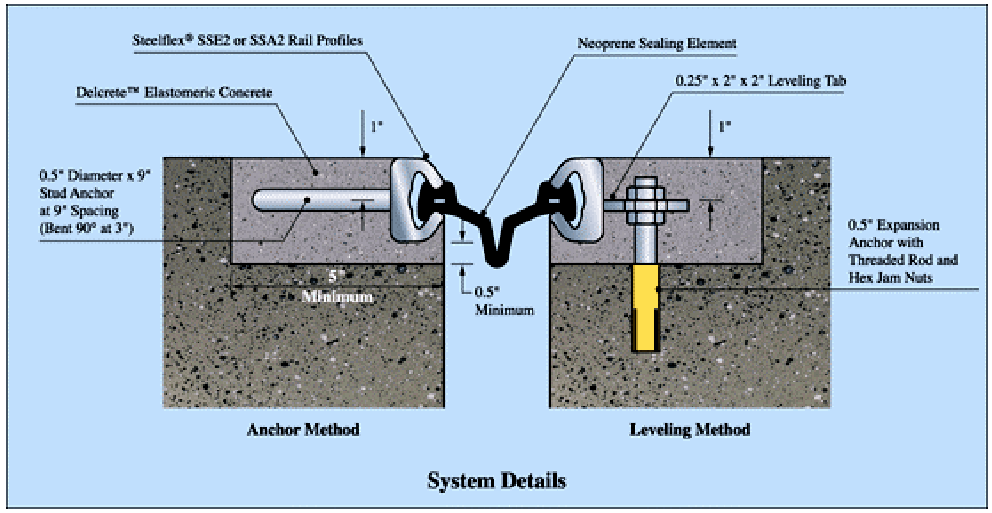


Figure 5.4.10 Strip Seal (Drawing Courtesy of the D.S. Brown Co.)

Modular Elastomeric Seal

The modular elastomeric seal is another neoprene type seal which can support vehicular wheel loads. It consists of hollow, rectangular neoprene block seals, interconnected with steel and supported by its own stringer system (see Figure 5.4.11). The normal range of operation for movement is between 100 and 600 mm (4 and 24 inches). It can, however, be fabricated to accommodate movements up to 1200 mm (48 inches).

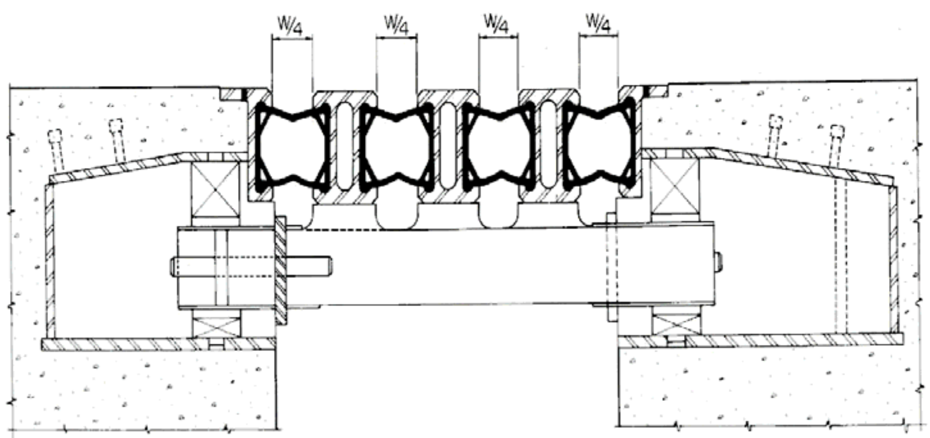


Figure 5.4.11 Schematic Cross Section of a Modular Elastomeric Seal

Asphaltic Expansion Joint

An asphaltic expansion joint is typically used on short bridges that are to be overlaid with asphalt. The joint expansion must be 50 mm (2 inches) or less. The original joint is usually a formed open joint that has deteriorated. Once the bridge joint is overlaid, the overlay material on the joint and a set distance in both directions of the joint is removed down to the original deck. A backer rod is then placed in the open joint and a sealant material is placed in the joint. Next, an aluminum or steel plate is centered over the joint to bridge the opening, and pins are put through the plate into the joint to hold it in place. A heated binder material is then poured on the plate to create a watertight seal. Layers of aggregate saturated with hot binder are then placed to the depth needed. The filled joint is then compacted. This type of joint allows for bridge decks to be overlaid without damaging existing expansion joints and is gaining popularity (see Figure 5.4.12).

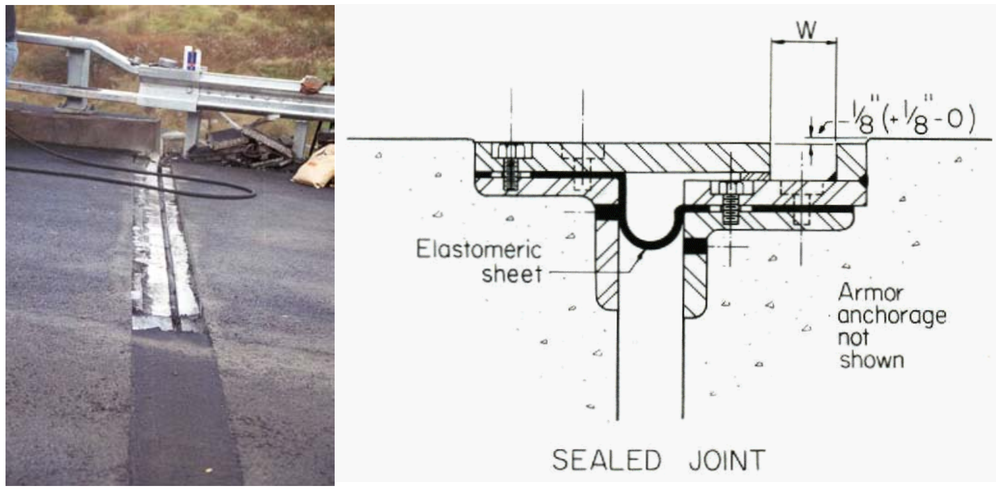


Figure 5.4.12 Asphaltic Expansion Joint

Drainage Systems

In order to perform an inspection of a deck drainage system, it is necessary to become familiar with its various elements:

- Runoff
- Bridge deck cross slope and profile
- Deck drains
- Outlet pipes
- Downspout pipes
- Cleanout plugs

Runoff

Runoff is the water and any contents that may run off the surface of the bridge deck.

Bridge Deck Cross Slope and Profile

The cross slope of the bridge deck is the first component of the drainage system that the runoff encounters. The proper cross slope and profile directs the runoff to the deck drains and eliminates or reduces ponding.

Deck Drains

The deck drain is the second component of the drainage system that runoff encounters. A deck drain is a receptacle to receive water. Deck drains may be nothing more than openings in a filled grid deck, holes in a concrete deck, or slots in the base of a parapet. Inlet boxes and scuppers are also examples of deck drains (see Figure 5.4.13).

Figure 5.4.13 Bridge Deck Inlet

Inlet boxes have a grate, which is a ribbed or perforated cover. Grates are fabricated from steel bars that are frequently oriented with the longitudinal direction of the bridge and spaced at approximately 50 mm (2 inches) on center. A bicycle safety grate has steel rods placed perpendicular to the grating bars, spaced at approximately 100 mm (4 inches) on center.

Grates keep larger debris from entering the drainage system while allowing water to pass through. They also serve to support traffic and other live loads. The drainage system may end with the deck drain.

Outlet Pipes

The outlet pipe leads water away from the drain. For bridges over roadways, the outlet pipe connects to other pipes. When the bridge is not over a roadway, the outlet pipe may simply extend a few feet down from the deck so that drainage water is not windblown onto the superstructure.

Downspout Pipes

When a bridge is located over a roadway, the deck drainage must be directed from the outlet pipe to a nearby storm sewer system or another appropriate release point. This is accomplished with a downspout pipe network (see Figure 5.4.14).

Cleanout Plugs

The cleanout plug is a removable plug in the piping system that allows access for cleaning.

Figure 5.4.14 Downspout Pipe and Cleanout Plug

Lighting

The four basic types of lighting which may be encountered on a bridge are:

- Highway lighting
- Traffic control lighting
- Aerial obstruction lighting
- Navigation lighting

Highway Lighting

The typical highway lighting standard consists of a lamp or luminaire attached to a bracket arm. Both the luminaire and bracket arm are usually made of aluminum. The bracket arm is attached to a shaft or pole made of concrete, steel, cast iron, aluminum, or, in some cases, timber. It is generally tapered toward the top of the pole.

The shaft is attached at the bottom to an anchor base. Steel and aluminum shafts are fitted inside and welded to the base. In the case of concrete, the shaft is normally cast as an integral part of the base. Sometimes the thickness of the parapet or median barrier is increased to accommodate the anchor base. This area of the barrier or parapet is called a "blister". Where the standard is exposed to vehicular traffic, a breakaway type base or guardrail may be used. Anchor bolts hold the light standard in place. These L-shaped or U-shaped bolts are normally embedded in a concrete foundation, parapet, or median barrier.

Traffic Control Lighting

Traffic control lights are used to direct traffic flow on a structure. Lights can serve a similar purpose to those found at intersections, but they can also indicate which lanes vehicular traffic is to use. These are referred to as lane control signals. Red and green overhead lights indicate the appropriate travel lanes.

Aerial Obstruction Lighting

Aerial obstruction lights are used to alert aircraft pilots that a hazard exists below and around the lights. They are red and should be visible all around and above the structure. Aerial obstruction lights are located on the topmost portion of any bridge considered by the Federal Aviation Administration (FAA) to present a hazard to aircraft. Depending on the bridge size, more than one light may be required.

Navigation Lighting

Navigation lights are used for the safe control of waterway traffic. The United States Coast Guard determines the requirements for the type, number, and placement of navigation lights on bridges. The lights are either green, red, or white and the specific application for each bridge site is unique.

Green lights usually indicate the center of a channel. These lights are placed at the bottom midspan of the superstructure. Red lights indicate the existence of an obstacle. When placed on the bottom of the superstructure, a red light indicates the limit of the channel. Lights placed to indicate a pier are placed on the pier near the waterline. Three white lights in a vertical fashion placed on the superstructure indicate the main channel.

Signs

Among the various types of signs to be encountered are signs indicating:

- Weight limit
- Vertical clearance
- Lateral clearance
- Narrow underpass
- Traffic regulatory and advisory

Weight Limit

Weight limit signs are very important since they indicate the maximum vehicle load that can safely use the bridge.

Vertical Clearance

Vertical clearance signs indicate the minimum vertical clearance for the structure. This clearance is measured at the most restrictive location within the traveling lanes.

Lateral Clearance

Lateral clearance signs indicate that the bridge width is less than the approach roadway width. Lateral clearance restrictions may be called out with a "Narrow Bridge" sign or with reflective stripe boards at the bridge.

Narrow Underpass

Narrow underpass signs indicate where the roadway narrows at an underpass or where there is a pier in the middle of the roadway. Striped hazard markings and reflective hazard markers should be placed on these abutment walls and pier edges. The approaching pavement should be appropriately marked to warn motorists of the hazard.

Traffic Regulatory and Advisory

Traffic regulatory and advisory signs indicate speed restrictions which are consistent with the bridge and roadway design. Additional traffic markers may be present to facilitate the safe and continuous flow of traffic.

5.4.3 Common Problems of Deck Joints, Drainage Systems, Lighting and Signs

Deck Joints

Common problems encountered when inspecting deck joints include the following:

- Debris and accumulation of dirt in deck joints and troughs under finger joints
- Corrosion on joints and their supports
- Damaged, torn, or missing joint seals due to snow plows, traffic, or debris buildup
- Spalled edges on joints without armor
- Spalled edges on joints due to misalignment of both sides of the joint
- Broken or misaligned fingers
- Leaking closed joint systems (or evidence of leaking)

Drainage Systems

Common problems encountered when inspecting drainage systems include the following:

- Debris buildup at inlet grate where water from the deck enters the drainage system
- Clogged or partially clogged deck drains and/or inlets
- Disconnected/clogged downspout piping
- Cracked or split pipes
- Loose or missing connections (from drain pipe below the deck to outlet pipe)
- Corrosion or section loss in metal pipes

Lighting and Signs

Common problems encountered when inspecting lighting and signs include the following:

- Lighting and signs obstructed from view due to tree growth or other signs
- Lighting and signs not present at bridge site
- Signs presently unacceptable or incorrect vertical or horizontal clearance
- Signs defaced or covered with graffiti
- Corrosion or section loss
- Loose or missing anchorages at supports
- Missing signs
- Lighting outages

5.4.4 Inspection Locations and Procedures for Deck Joints, Drainage Systems, Lighting and Signs

Deck Joints

The deck joints must allow for the expansion and contraction of the bridge deck and superstructure. The inspector must be aware of and record conditions that keep the deck joint from functioning properly.

There is not a separate item on the Structure Inventory and Appraisal (SI&A) sheet to code the serviceability of deck joints, and deck joint conditions are not considered in the rating of the bridge. However, it is important for the inspector to note their condition since deck joint problems are often related to problems elsewhere on the bridge.

The Element Level Inspection system, however, does rate deck joints. For a detailed description of deck joint condition states, see the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements and the evaluation section of this topic.

Deck joints should be inspected for:

- Dirt and debris accumulation
- Proper alignment
- Damage to seals and armored plates
- Indiscriminate overlays
- Joint supports

- Joint anchorage devices

Dirt and Debris Accumulation

Dirt and debris lodged in the joint may prevent normal expansion and contraction, causing cracking in the deck and backwall, and overstress in the bearings. In addition, as dirt and debris is continually driven into a joint, the joint material can eventually fail (see Figures 5.4.15 and 5.4.16).

Figure 5.4.15 Debris Lodged in a Sliding Plate Joint

Figure 5.4.16 Dirt in a Compression Seal Joint

Proper Alignment

Both sides of the joint should be at the same level with no vertical displacement between the two. On straight bridges, the joint opening should be parallel across the deck.

In a finger plate joint, the individual fingers should mesh together properly, and they should be in the same plane as the deck surface (see Figure 5.4.17).

Figure 5.4.17 Improper Vertical Alignment at a Finger Plate Joint

It is important that the relative movements of the joint are consistent with the temperature. During the coldest and the warmest times of the day, the air temperature and the superstructure temperature should be recorded, and the joint opening should be documented. Measurements should be taken at each curb line and the centerline of the roadway. Since heat causes expansion, the joint opening should be smallest when the temperature is greatest. The superstructure temperature can be taken by placing a surface temperature thermometer or the bulb of a standard thermometer against the superstructure member itself. The superstructure temperature is generally about 1.7 to 2.8 °C (3 to 5 ºF) lower than the air temperature.

Damage to Seals and Armored Plates

Damage from snow plows, traffic, and debris can cause the joint seals to be torn, pulled out of the anchorage, or removed altogether (see Figure 5.4.18). It can also cause damage to armored plates. Any of these conditions should be noted by the inspector. Also look for evidence of leakage through closed joints.

Figure 5.4.18 Failed Compression Seal

Indiscriminate Overlays

When new pavement is applied to a bridge, it is frequently placed over the deck joints with little or no regard for their ability to function properly. This occurs most frequently on small, local bridges. Transverse cracks in the pavement may be evidence that a joint has been covered by the indiscriminate application of new overlay, and the joint function may be severely impaired (see Figure 5.4.19).

Figure 5.4.19 Asphalt Wearing Surface over an Expansion Joint

Joint Supports

Where larger expansions and contractions must be accommodated, the joint may be fully or partially supported from beneath by transverse beams. These joint supports should be carefully inspected for proper function and for corrosion and section loss (see Figure 5.4.20).

Figure 5.4.20 Support System under a Finger Plate Joint

Joint Anchorage Devices

Deficiencies in joint anchorage devices are a common source of deck joint problems. Therefore, joint anchorage devices should be carefully inspected for proper function and for corrosion. The concrete area in which the joint anchorage device is cast should also be inspected for signs of deterioration. This area adjacent to the joint is known as the joint header.

Drainage Systems

A properly functioning drainage system removes water, and all hazards associated with it, from a structure. There is not a separate item on the SI&A Sheet to code the serviceability of drainage systems, and drainage system conditions are not considered in the rating of the bridge. However, it is important for the inspector to note their condition, since drainage system problems can eventually lead to structural problems.

The following drainage system elements should be inspected:

- Bridge deck cross slope and profile
- Grates
- Deck drains and inlets
- Drainage troughs
- Outlet pipes

Bridge Deck Cross Slope and Profile

The cross slope and profile should not prevent runoff from entering the deck drains and inlets. Adequate cross slope should be provided so that water runs off the bridge deck at a sufficient rate.

Grates

Grates should be clear of debris (e.g., plants and grass) and free to allow deck runoff to enter. Grates that are deteriorated, broken, or missing should be reported.

Deck Drains and Inlets

Deck drains and inlets must be of sufficient size and spacing to carry the runoff away from the structure effectively. Since runoff conditions can change due to development, these drainage elements should be carefully examined with each bridge inspection. Clogged deck drains lead to accelerated deck deterioration and the undesirable condition of standing water in the traffic lanes (see Figure 5.4.21).

Figure 5.4.21 Clogged Drainage Inlet

Drainage Troughs

Drainage troughs located under the joint should be carefully examined. A buildup of debris can accelerate the deterioration of the trough and allow water to drain onto structural members. If possible, use a shovel to clean as much debris as practical; report the remaining condition for appropriate maintenance work. Once cleaned, any holes found in the trough should be noted. Any evidence that indicates the trough is overflowing should also be mentioned (see Figure 5.4.22).

Figure 5.4.22 Drainage Trough with Debris Accumulation

Outlet Pipes

Outlet pipes carry runoff away from the structure. The outlet pipe may be a straight extension of the deck drain, in which case it should be long enough so that runoff is not discharged onto the structure. The outlet pipe may also be a series of pipes, called downspouting. This type of outlet pipe should be examined for split or disconnected pipes that may allow runoff to accelerate deterioration of the structure. Check the connections between the outlet pipes and substructure. If a pipe is embedded inside of a substructure unit such as a concrete pier wall, check for cracking, delamination, or other freeze-thaw damage to the substructure.

Lighting

All lights should be clearly visible. Verify that all lights are functioning and that they are not obstructed from view. Check for corrosion and collision damage to light supports. Verify that appropriate lighting is provided. Exercise caution against electrocution. The inspector should contact the maintenance department to de-energize the lighting.

Signs

Signs should be located sufficiently in advance of the structure to permit the driver adequate time to react. All signs should be clearly legible. Verify that signs have not been defaced and are not obstructed from view. Inspect for corrosion and collision damage to sign supports. Verify that appropriate signing is provided.

5.4.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of deck joints, drainage systems, lighting, and signs. 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 NBIS Rating Guidelines

Deck joints, drainage systems, lighting, and signs should not impact the deck rating, but their condition should be described in the inspection report. Deficiencies in deck joints, drainage systems, lighting, and signs should be placed on the maintenance sheet showing estimated quantities.

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 reviewed for the expansion joint. 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 expansion joints, the AASHTO CoRe element is one of the following, depending on the type of joint:

Element No.	Description
300	Strip seal expansion joint
301	Pourable joint seal
302	Compression joint seal
303	Assembly joint seal (modular)
304	Open expansion joint

Individual states have the option to change or add element numbers. In the case of expansion joints, some states have added a miscellaneous expansion joint element number.

The unit quantity for these elements is in meters or feet, and the entire element must be placed in one of the three available condition states. Condition state 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

Drainage systems, lighting, and signs have no separate element numbers. The condition of the drainage systems, lighting, and signs should, however, be noted on the inspection form.

Table of Contents

Section 5

Inspection and Evaluation of Decks

This page intentionally left blank

Topic 5.5 Safety Features

5.5.1 Introduction

For the past 25 years, highway design has included a special emphasis on providing safe roadsides for errant vehicles that may leave the roadway. Obstacles or fixed object hazards have typically been removed from within a specified roadside recovery area. Whenever this has not been feasible (for example, at bridge waterway crossings), then safety features such as highway or bridge barrier systems have been provided to screen motorists from the hazards present (see Figure 5.5.1). Such barriers sometimes constitute fixed object hazards themselves, though hopefully of less severity than the hazard they screen.

Figure 5.5.1 Bridge Safety Feature

Purpose

The barriers on bridges and their approaches are typically intended to provide vehicular containment and prevent motorist penetration into the hazard being over-passed, such as a stream or under-passing roadway or railroad. Containment of an errant vehicle is a primary consideration, but survival of vehicle occupants is of equal concern. Thus the design of bridge railing systems and bridge approach guardrail systems is intended to first provide vehicular containment and redirection, but then to also prevent rollover, to minimize snagging and the possibility of vehicle spinout, and to provide smooth vehicular redirection parallel with the barrier system. In addition, the bridge railing and bridge approach guardrail systems must do all of this within tolerable deceleration limits for seat-belted occupants.

Four Basic Components

Barrier systems at bridges are composed of four basic components:

- Bridge railing
- Transition
- Approach guardrail system
- Approach guardrail end treatment

Bridge Railing

The function of bridge railing is to contain and redirect errant vehicles on the bridge. Many rails could conceivably do this, but the safety of the driver and redirection of the vehicle must be taken into account.

Transition

A transition occurs between the approach guardrail system and bridge railing. Its purpose is to provide both a structurally secure connection to the bridge end post and also a zone of gradual stiffening and strengthening of the more flexible approach guardrail system where it is connected with the rigid bridge railing. Stiffening is essential to prevent "pocketing" or "snagging" of a colliding vehicle just before the rigid bridge railing end.

Approach Guardrail System

The approach guardrail system is intended to screen motorists from the hazardous feature beneath the bridge as they are approaching the bridge. This approach guardrail screening is often extended in advance of the bridge so as to also screen motorists from any hazardous roadside features on the approach to the bridge (see Figure 5.5.3).

Approach guardrail must have adequate length and structural qualities to safely contain and redirect an impacting vehicle within tolerable deceleration limits. Redirection should be smooth, without snagging, and should minimize any tendency for vehicle rollover or subsequent secondary collision with other vehicles. Similar to bridge railing, approach guardrail systems must satisfy agency standards, which specify acceptable heights, materials, strengths, and geometric features.

Approach Guardrail End Treatment

The approach guardrail end treatment is the special traffic friendly anchorage of the approach guardrail system (see Figures 5.5.2 and 5.5.3). It is located at the end at which vehicles are approaching the bridge. Ground anchorage is essential for adequate performance of the guardrail system. Special end treatment is necessary in order to minimize its threat to motorists as another fixed object hazard within the roadside recovery area.

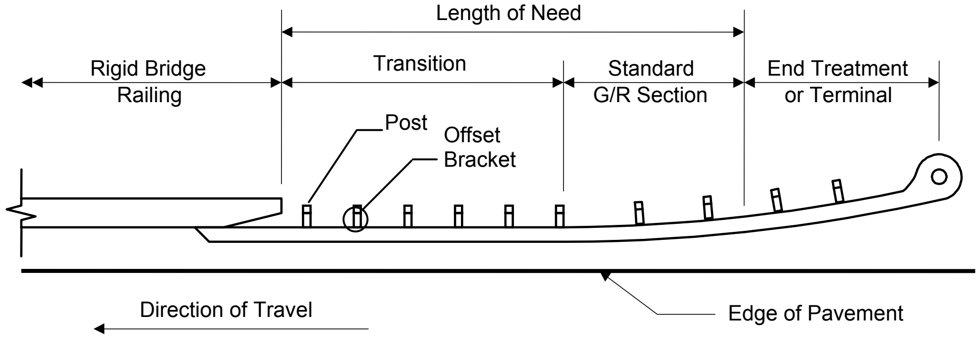


Figure 5.5.2 Approach Guardrail System

Figure 5.5.3 Approach Guardrail System

5.5.2 Evaluation

Each of the various elements of the bridge rail system is designed to meet a specific function. Based on items from an inspection checklist, the inspector can make a determination of whether or not these elements work as they should. The elements must pass the minimum standard criteria established by AASHTO.

Design Criteria

Until the mid 1980's, bridge railings were designed consistent with earlier precedent, the guidance provided in the AASHTO Standard Specifications for Highway Bridges, and professional judgment. The AASHTO Standard Specifications called for application of a 10-kip horizontally applied static load at key locations, and certain dimensional requirements were also specified. Full-scale crash testing was not required, although a design that "passed" such testing was also considered acceptable for use. Subsequent crash testing of several commonly used, statically designed bridge railings revealed unexpected failures of the systems. It was soon concluded that static design loadings were not sufficient to ensure adequate railing performance. As a result of these findings, the FHWA issued guidance in 1986 requiring that bridge railing systems must be (or must have been) successfully crash tested to be considered acceptable for use on Federal-aid projects.

Longitudinal roadside barriers, such as guardrail systems, had also been designed consistent with earlier precedent and judgment. Subsequent crash testing of these systems again revealed some unacceptable designs and prompted development of several new guardrail systems and details that were then identified as acceptable for new highway construction on Federal-aid projects.

Crash Test Criteria

Test requirements generally accepted at first were those contained in the National Cooperative Highway Research Program (NCHRP) Report 230 and in several earlier Transportation Research Board publications. In 1989, AASHTO published its "Guide Specifications for Bridge Railings," wherein not only were the required tests specified but they were categorized into three separate performance levels. A warrant selection procedure was also included for determining an appropriate performance level for a given bridge site. As the crash test criteria differed in some respects from Report 230, use of the "Guide Specification" was, and continues to be, optional.

In 1990, the FHWA identified a number of crash-tested railing systems that met the requirements of NCHRP Report 230 or one of the performance levels in the AASHTO Guide Specifications. At this point, the FHWA considered that any railing that was acceptable based on Report 230 testing could also be considered acceptable for use, at least as a PL-1 (performance level 1) as described by the AASHTO Guide Specifications. They also stated that any SL-1 (service level 1) railing developed and reported in NCHRP Report 239, "Multiple-Service-Level Highway Bridge Railing Selection Procedures," could be considered equivalent to a PL-1 railing.

In 1993, NCHRP Report 230 was superseded by NCHRP Report 350, "Recommended Procedures for the Safety Performance Evaluation of Highway Features." Its current testing criteria include provisions for six different test levels, all of which differ in some ways from the previous Report 230 tests, as well as those in the AASHTO Guide Specifications. No selection procedures or warrants for the use of a specific test level are included in Report 350, although a separate research effort is underway to establish such warrants. Adding to the conflicting guidance for selection of an appropriate bridge railing system, the 1994 AASHTO LRFD Bridge Design Specifications have been issued as an alternate to the long-standing AASHTO Standard Specifications for Highway Bridges. This most recent bridge design specification contains recommendations on railing designs and crash testing which differ from both NCHRP Report 350 and the AASHTO Guide Specifications.

Current FHWA Policy

Bridge railings to be installed on National Highway System (NHS) projects must meet the acceptance criteria contained in NCHRP Report 350 or a recognized successor to those criteria. The minimum acceptable bridge railing for high-speed highways is a Test Level 3 (TL-3) unless supported by a rational selection procedure. For locations where the posted speed limit is less than 45 mph, a TL-2 bridge railing is considered acceptable.

Railings that have been found acceptable under the crash testing and acceptance criteria of NCHRP Report 230, the AASHTO Guide Specifications for Bridge Railings, or the AASHTO LRFD Bridge Design Specifications will be considered as meeting the requirements of NCHRP Report 350, provided they are equivalent to appropriate Report 350 Test Levels. This comparison of equivalencies has been tabulated by the FHWA in their May 30, 1997 memorandum on crash testing of bridge railings, with an attached May 14, 1996 document on bridge railing design and testing.

The FHWA continues to encourage support for development of railing test level selection procedures. In the interim, until AASHTO adopts a new railing test level selection procedure, the FHWA will accept the procedures in the AASHTO Guide Specifications or, as an alternate, a rational, experience-based, cost beneficial, consistently applied procedure proposed by an individual state. Their 1996 document includes a listing of railings considered acceptable under the NCHRP Report 350 guidelines or their presumed equivalent guidelines. New crash-tested railings continue to be approved and added, and their identity and features can be obtained from the FHWA.

For non-NHS projects, the setting of criteria for establishing acceptability for bridge railings has been relegated by the FHWA to the individual states. Some states require conformity with the FHWA's NHS criteria for all bridges, on any of the highway systems. In other states, lesser performance criteria are accepted for bridges on non-NHS roads, so there may be variations between states as to safety feature acceptability.

Railing Evaluation Results/Resources

The FHWA maintains a website, http://safety.fhwa.dot.gov/programs/roadside_hardware.htm which, identifies all of the bridge and longitudinal roadside barrier systems, transitions, and end treatments which have been found to meet the various crash test requirements of NCHRP Reports 350 and 230. The website includes acceptance letters as well as links to manufacturers' websites for information on proprietary systems. Listings for several categories of safety features are accessible. New listings of bridge barriers more recently tested may be found on the longitudinal barrier list so a thorough search of all listings is advisable to identify a specific feature and its test results. The May 30, 1997 memorandum and its attached document with test level equivalencies can also be found on the website.

Additional information can also be found in the current AASHTO "Roadside Design Guide" and in the current AASHTO-AGC-ARTBA Report, "A Guide to Standardized Highway Barrier Hardware."

5.5.3 Identification and Appraisal

Identification of conforming and non-conforming bridge safety features will vary depending upon highway classification and the jurisdiction involved. With various acceptance criteria to consider and with continuing crash testing and approvals of new barriers, it is advisable to rely on the most current specific acceptance criteria for the particular state or jurisdiction within which a bridge is located. A listing of currently conforming versus non-conforming bridge safety features should be obtained for each jurisdiction prior to identification and appraisal of these features in the course of bridge inspections within that jurisdiction.

Appraisal Coding

The FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*Coding Guide*) requires an evaluation and reporting as to whether each of the four basic components satisfactorily conform to current safety design criteria for the respective component.

The condition of the safety features is not considered in the appraisal, but should be well documented in the inspection report. After determining whether the safety features at the site are acceptable, the inspector should assign an appraisal code. The FHWA *Coding Guide* contains four entries for safety features: one each for the bridge railing, approach guardrail, transition, and end treatment.

After making the determination as to whether or not safety features at the site meet currently acceptable standards, the inspector assigns an appraisal code of either 1 (meets) or 0 (does not meet) for each element of Item 36 (page 17, FHWA *Coding Guide*:

36A Bridge railing system
36B Approach guardrail transition
36C Approach guardrail system type
36D Guardrail end treatment

While there is only one safety features coding for each element, there are at least two bridge railings and four approach guardrail treatments. Some states have modified and set different coding standards. Therefore, the bridge inspector should code the worst condition for each element even though they may occur at different locations on the bridge.

Bridge Railings

Some examples of currently conforming concrete bridge railings for NHS roadways include 32" high New Jersey shape barrier, the concrete "F" shape, and the single slope concrete barrier (see Figures 5.5.4 and 5.5.5). Some steel post and beam railings mounted on a low curb, such as the Wyoming 2-tube steel railing (see Figure 5.5.6) also conforms, as do some combination safety shape barriers with a metal railing mounted on top. Safety shape barriers with a metal railing must not have a safety-walk in front of the railing. All bridge railings must pass current crash test requirements.

Figure 5.5.4 New Jersey Barrier

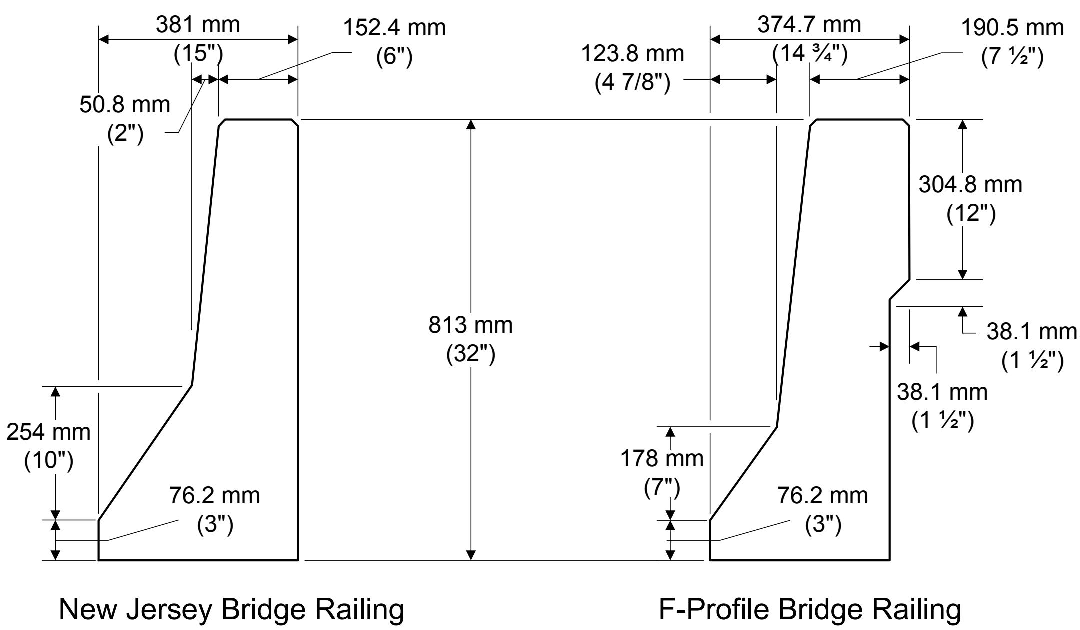


Figure 5.5.5 Comparison of New Jersey and "F" Shape

Figure 5.5.6 Wyoming 2-Tube Steel Railing

Transitions

A number of transitions between approach guardrail system and bridge railing were tested successfully using NCHRP Report 230 test criteria (see Figure 5.5.7). These are all illustrated schematically in the AASHTO "Roadside Design Guide." Though tested under NCHRP Report 230, the FHWA currently considers these as conforming transitions, at least until October 2002.

Figure 5.5.7 Thrie-beam System

Transition stiffening is usually accomplished through use of:

- Decreased post spacing
- Increased post size
- Embedment of posts in concrete bases
- Increased rail thickness, using a thicker gage rail element or by nesting two layers

Vehicle snagging is discouraged by providing an increased rail surface projection with either a broader rail face (e.g., thrie beam) or a rub rail being placed beneath the primary rail, to minimize both guardrail post and bridge endpost exposure as potential snag points.

Older transitions usually have some of the essential features but are often lacking in some. There may be guardrail anchorage to the bridge but insufficient stiffening, or perhaps some degree of stiffening but insufficient concealment of potential snag points such as the front corner of the bridge endpost or exposed guardrail posts. Cable connections to the bridge railing do not meet minimum criteria because they do not provide a smooth stiffened transition. Timber approach rail attached to the bridge rail is not an acceptable transition. No transition is provided at all when the bridge railing and approach guardrail are not structurally connected. Bridge railing and approach guardrail that do not in themselves meet minimum criteria will surely not have a transition that is adequate.

Approach Guardrail Systems

The FHWA's February 14, 2000 memo to Resource Centers summarizes the non-proprietary longitudinal barrier systems that are currently considered to meet NCHRP Report 350 guidelines. The strong post (steel or wood) W-beam guardrails with wood or approved plastic blocks are examples at Test Level 3, as are the strong post thrie-beam systems (see Figure 5.5.8). The same W-beam barriers used with a steel block are included at Test Level 2.

Figure 5.5.8 Thrie-beam System

Post and cable systems do not meet minimum criteria for bridge approach guardrail systems because they allow both snagging and pocketing of a vehicle upon impact. Timber approach guardrail does not meet minimum criteria for strength, continuity, or performance.

Approach Guardrail End Treatment

A variety of guardrail end treatments have been approved for use by the FHWA. The specific installation is dependent on various roadway features and testimony procedures as administered by the National Cooperative Highway Research Program (NCHRP). Current listings of crash tested end treatments and documentation of their performance can be found at http://safety.fhwa.dot.gov/fourthlevel/pro_res_road_nchrp350.htm. Probably the most universally effective is the buried-in-back-slope treatment where the longitudinal barrier is introduced from a buried anchorage, typically from a cut slope preceding the bridge approach guardrail installation (see Figure 5.5.9). Essential for these installations are keeping a constant rail height relative to the roadway grade and then provision of both a rub rail and an anchorage capable of developing the full strength of the W-beam rail.

Figure 5.5.9 W-Shaped Guardrail End Flared and Buried into an Embankment

Several modern proprietary end treatments that are currently in use, include:

- Sequential Kinking Terminal (SKT-350, Road Systems, Inc.)
- Extruder Terminal (ET-2000)
- Crash-cushion Attenuating Terminal (CAT-350, Trinity Industries)
- FLared Energy-Absorbing Terminal (FLEAT-350, Road Systems, Inc.) (see Figure 5.5.10).
- Slotted Rail Terminal (SRT-350) (see Figure 5.5.11)
- Improved slotted rail terminal (ROSS) – a modification of the slotted rail terminal
- Modified Eccentric Loads Terminal (MELT) (see Figure 5.5.13)

Figure 5.5.10 FLEAT-350 - FLared Energy-Absorbing Terminal

Figure 5.5.11 SRT-350 - Slotted Rail Terminal

Figure 5.5.12 TAU-II Redirective, Non-Gating Crash Cushion

Flaring the guardrail end to reduce the likelihood of a vehicular impact is only effective if there is enough space for a substantial flare from the edge of traveled way.

Burying the guardrail end has been used with and without flaring. If the guardrail end is turned down for burying, it has frequently produced rollover accidents and is not currently considered an acceptable end treatment.

If the end is concealed by flaring without turning down, and then burying at full height in a cut slope, the method has proven effective at preventing end impacts.

One of several breakaway treatments can be used. The guardrail end is modified to permit safe penetration through the system for end impacts, yet effective redirection of vehicles for impacts slightly after of the end treatment.

One of the most familiar and long-used breakaway end treatments is the BCT or breakaway cable terminal. This end treatment must also be flared to be safely effective. Flaring, along with blunting of the rail end serve to facilitate a buckling of the rail element rather than vehicle impalement. Two weakened timber posts are breakaway to minimize deceleration forces upon direct impact with either post. Cable anchorage of the rail is provided to assure adequate anchorage of the system for possible impacts after of the end treatment.

A newer version of the BCT called the MELT, for modified eccentric loader terminal, provides improved breakaway performance, especially for smaller vehicles (see Figure 5.5.13). An eccentric loader and buffer stiffening are employed to assure deflection of the rail without impaling, and a load-distributing strut between the first two breakaway timber posts enhances rail anchorage capability.

Figure 5.5.13 MELT - Modified Eccentric Loads Terminal

Other newer breakaway or energy-absorbing end treatments include the ET 2000 extruder terminal, a proven end treatment which, when impacted, slides down the rail causing diversion of the W-beam rail element through a flattening or extruding fitting (see Figure 5.5.14). As the rail is threaded through and flattened out by the extruder, impact energy is expended. The flattened rail element is peeled back out of the way as vehicle energy is transferred and gradual deceleration occurs. This end treatment is designed for use without flaring.

Figure 5.5.14 ET 2000 - Extruder Terminal

The CAT or crash-cushion attenuating terminal progressively collapses with perforated W-beam rails telescoping as the system safely decelerates an impacting vehicle (see Figure 5.5.15). Crash energy is attenuated in the process. Breakaway timber posts are employed and the treatment does not have to be flared. This makes it a feasible end treatment for guardrail introduction when there is insufficient roadside space for flaring.

Figure 5.5.15 CAT – Crash-cushion Attenuating Terminal

The SENTRE guardrail end treatment is another telescoping terminal, which utilizes posts with slip bases for breakaway and sand-filled boxes to gradually decelerate and gently guide an impacting vehicle away from the fixed rail hazard (see Figure 5.5.16). All major components are reusable which makes the system more economical for locations where more frequent impacts are expected. Flaring is possible but not required.

Figure 5.5.16 SENTRE End Treatment

The TREND dual purpose end treatment has similar features with reusable telescoping rail panels, redirecting cable, breakaway slip base posts, and replaceable sand-filled boxes (see Figure 5.5.17). However, the system is unique in that it is designed to also serve in a dual role as a transition connection to a rigid bridge railing. It is not designed to be flared.

Figure 5.5.17 TREND End Treatment

The last method for railing end treatment is shielding of the barrier with an energy-absorbing or attenuating system which dissipates impact energy as an impacting vehicle is gradually brought to a stop before reaching a rigid bridge rail endpost. Though vehicle damage may be severe, deceleration is controlled within tolerable limits to minimize occupant injury.

A variety of impact attenuators have been used, including expendable sand-filled containers, which shatter and absorb energy during impacts.

There are also more elaborate telescoping fender systems, which redirect side impacts but also telescope and attenuate crash energy through crushing of replaceable foam-filled cartridges for direct impacts. Older versions absorbed energy through expulsion of water from water-filled tubes as the device collapsed. Most parts for these more elaborate devices are reusable, making them very suitable for bridge rail end locations where frequent impacts might be expected.

In certain cases, such as at the trailing end of a one-way bridge, guardrail is not required at all.

The type of end treatment, which has sometimes been called a boxing glove, is not an acceptable end treatment.

5.5.4 Median Barriers

Median barriers are used to separate opposing traffic lanes when the average daily traffic (ADT) on the road exceeds a specified amount. They are usually found on high speed, limited access highways.

The most commonly used median barrier on bridges is the concrete median barrier. This is a double sided parapet, and it should meet the current criteria for the crash testing of bridge railing. The only acceptable end treatment for a concrete median barrier is an impact attenuator.

Double-faced steel W-beam or thrie beam railing on standard heavy posts are also used for median barriers.

Inspection of Median Barriers

Median barriers should be firmly attached to the deck, and they should be functional. Inspect for collision damage and attachment to any additional safety features. Check for deterioration and spalling on concrete median barriers, and examine for corrosion on steel railings and posts.

5.5.5 Safety Feature Inspection

The inspection of bridge safety features involves evaluation of the bridge railing system on the bridge, the guardrail system leading from the bridge, the guardrail system leading from the approach roadway to the bridge end, and whether these two systems will likely function acceptably together to safely contain and redirect errant vehicles which may collide with them.

Inspection

Criteria that must be considered during the inspection of the bridge railing are the height, material, strength, geometric features, and the likelihood of acceptable crash test performance.

Many state agencies have developed their own acceptance guidelines for bridge railings. The inspector should be familiar with agency guidelines for his or her state.

Bridge Railing

The following should be inspected on a bridge railing:

- Metal bridge railings should be firmly attached to the deck and should be functional. Check especially for corrosion and collision damage, which might render these railings ineffective (see Figure 5.5.18). Comparison of existing metal railing systems with approved crash-tested designs will establish their acceptability and crash worthiness.

Figure 5.5.18 Damaged Steel Post Bridge Railing

- Concrete bridge railing is generally cast-in-place and engages reinforcing bars to develop structural anchorage in the deck slab. Verify that the concrete is sound and that reinforcing bars are not exposed.

A commonly used bridge railing is the New Jersey parapet or safety shape.

If add-on rails are other than decorative or for pedestrians, their structural adequacy can again be verified by comparison with successfully crash tested designs.

Approach Guardrail

The following should be inspected on an approach guardrail:

- The inspector should verify that agency standards are met.
- Make note of rail element type, post size and post spacing for comparison with approved designs to verify acceptability of the guardrail system.
- Document any significant collision damage, which is evident (see Figure 5.5.19).
- Note any deterioration of guardrail elements, which could weaken the system.
- Note any areas where the railing may "pocket" during impact, snagging the vehicle and causing an abrupt deceleration or erratic rebound.
- Loose or missing bolts should also be noted.

Unless specifically designed for impact, timber approach guardrail does not meet minimum criteria for strength.

Figure 5.5.19 Approach Guardrail Collision Damage

Transition

The following should be inspected on a transition:

- Check the approach guardrail transition to the bridge railing for adequate structural anchorage to the bridge railing system.

- Check for sufficiently reduced post spacing to assure stiffening of the guardrail at the approach to the rigid bridge rail end.

- Check for smooth transition details to minimize the possibility of snagging an impacting vehicle, causing excessive deceleration.

Timber should not be used for the rails in transitions.

End Treatment

Note the type, condition, and suitability of any end treatment. Acceptable crash-tested end treatments are identified in the AASHTO Roadside Design Guide or with current FHWA issuances.

End treatments may not be required on the trailing end of a one-way bridge.

Table of Contents

Section 6
Inspection and Evaluation of Common Timber Superstructures

This page intentionally left blank

Section 6

Inspection and Evaluation of Common Timber Superstructures

Topic 6.1 Solid Sawn Timber Bridges

6.1.1 Introduction

Timber bridges are gaining a resurgence in popularity throughout the United States. There are two basic classifications in timber construction: solid sawn and glued-laminated (glulam). A solid sawn beam is a section of tree cut to the desired size at a saw mill. Solid sawn multi-beam bridges are the simplest type of timber bridge (see Figure 6.1.1).

Figure 6.1.1 Elevation View of a Solid Sawn Multi-Beam Bridge

6.1.2 Design Characteristics

Multi-beam Bridges

Solid sawn multi-beam bridges consist of multiple solid sawn beams spanning between substructure units (see Figure 6.1.2). The deck is typically comprised of transversely laid timber planks, which are supported by the beams, and longitudinally laid planks called runners, on which the vehicles ride. Sometimes a bituminous wearing surface is placed on the deck planks to provide a skid resistant riding surface for vehicles, as well as a protective surface for the planks. Beam sizes typically range from about 150 mm by 300 mm (6 inches by 12 inches) to 200 mm by 400 mm (8 inches by 16 inches), and the beams are usually spaced about 600 mm (24 inches) on center.

Figure 6.1.2 Underside View of a Solid Sawn Multi-Beam Bridge

This bridge type is generally used in older, shorter span bridges, spanning up to about 8 m (25 feet). Shorter spans are sometimes combined to form longer multiple span bridges and trestles. Many older timber trestles were built for railroads and trolley lines. Solid sawn timbers have become obsolete for most modern bridge members due to the development of high quality glulam members (see Topic 6.2).

Covered Bridges

Covered bridges are generally found along rural roads and get their name from the fact that walls and a roof protect the bridge superstructure. They are usually owned by local municipalities, although some are owned by states or private individuals. Some still carry highway traffic, but many are only open to pedestrians. While most covered bridges were built during the 1800's and early 1900's, there are a number of covered bridges being built today as historic reconstruction projects (see Figures 6.1.3 and 6.1.4).

Figure 6.1.3 Elevation View of Covered Bridge

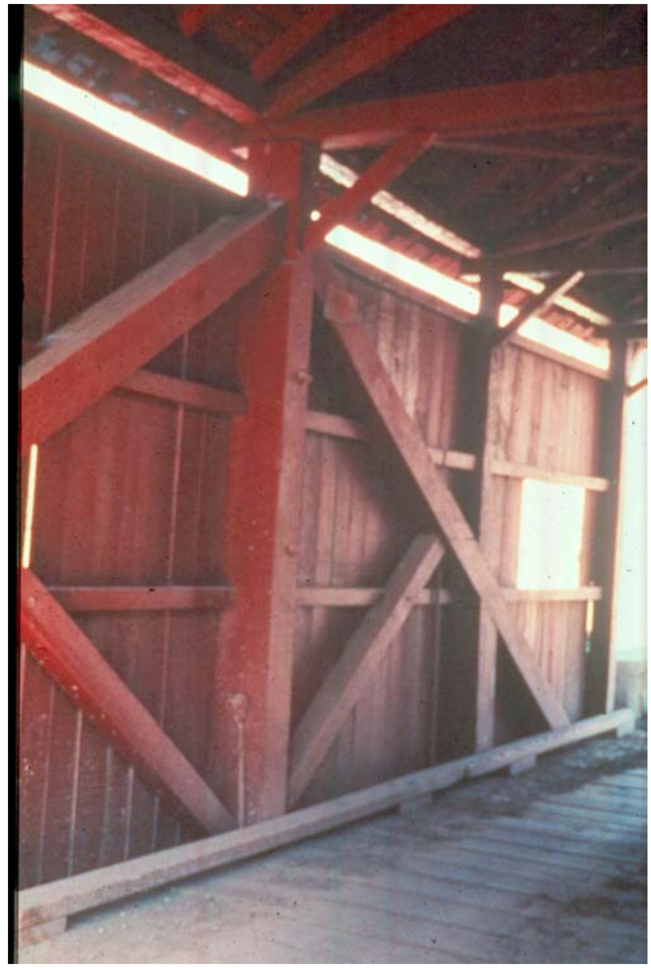


Figure 6.1.4 Inside View of Covered Bridge Showing King Post Truss Design

Trusses

The majority of covered bridges are essentially truss bridges (see Figure 6.1.5). Solid sawn timber members make up the trusses of these historic structures. The covers on the bridges prevent decay of the truss and undoubtedly are responsible for their longevity. Typical truss types for covered bridges include the king post, queen post, Town, Warren, and Howe (see Figure 6.1.6). The floor system consists of timber deck planks, stringers, and floorbeams. The span lengths of covered bridges are generally in the range of 15 to 30 m (50 to 100 feet), although many are well over 30 m (100 feet) and some span over 61 m (200 feet)).

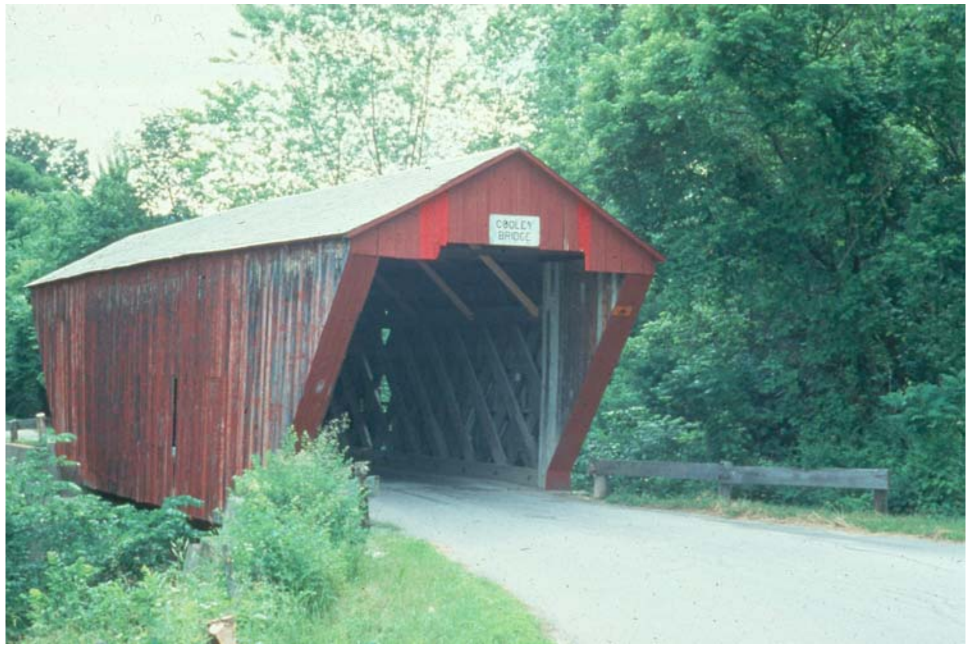


Figure 6.1.5 Town Truss Covered Bridge

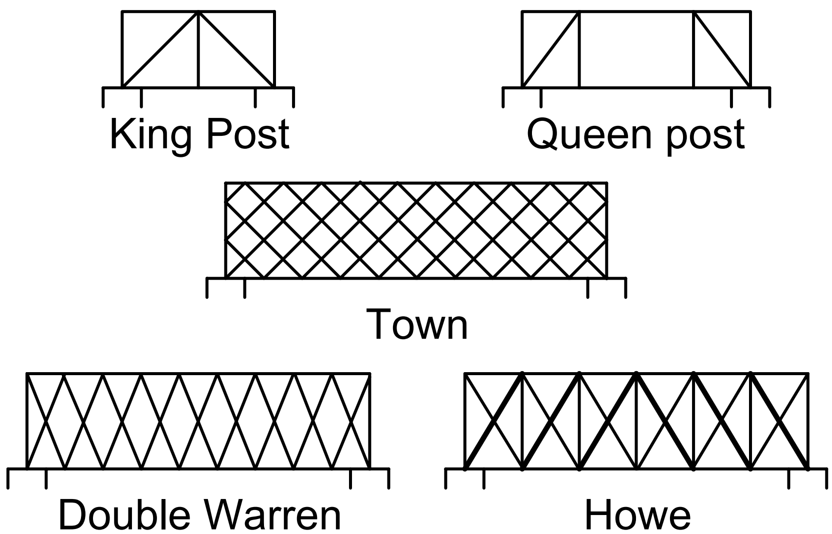


Figure 6.1.6 Common Covered Bridge Trusses

Arches

Timber arches were first used in covered bridges by Theodore Burr to strengthen the series of truss configurations normally used in covered bridges. These became known as Burr arch-trusses (see Figures 6.1.7, 6.1.8 and 6.1.9). The arch served as the main supporting element, and the king posts simply strengthened the arch. Because of their greater strength, many of these structures still exist today.

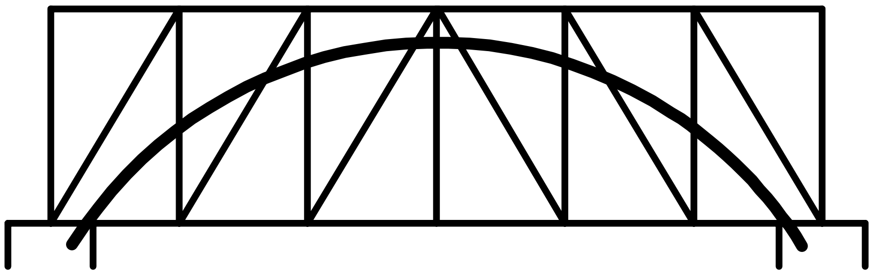


Figure 6.1.7 Schematic of Burr Arch-truss Covered Bridge

Figure 6.1.8 Burr Arch-truss Covered Bridge

Figure 6.1.9 Inside View of Covered Bridge with Burr Arch-truss Design

Primary and Secondary Members

The primary members of solid sawn multi-beam bridges are the beams, and the secondary members are the diaphragms or cross bracing if present (see Figure 6.1.2). These bridges usually have timber diaphragms or cross bracing between beams at several locations along the span.

The primary members in truss and arch structures are the truss members (chords, diagonals, and verticals), arch ribs, stringers, and floorbeams (see Figure 6.1.10). The secondary members are the diaphragms and cross bracing between stringers, the upper and lower lateral bracing, sway bracing, and the covers on the roof and sides when present.

Figure 6.1.10 Town Truss Design

6.1.3 Overview of Common Defects

Common defects that occur on solid sawn timber beams include:

- Checks, splits, and shakes
- Decay by fungi
- Damage by insects and borers
- Damage from impact/collisions
- Damage from abrasion/wear
- Damage from weathering/warping
- Damage from overstress

Other less common defects that may be encountered by the inspector include damage from chemical attack and damage from fire, which can be very destructive to timber structures. Refer to Topic 2.1 for a more detailed presentation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

6.1.4 Inspection Procedures and Locations

Procedures

Advanced Inspection Techniques

In addition, several advanced techniques are available for timber inspection.

Nondestructive methods, described in Topic 13.1.2, include:

- Pol-Tek
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 13.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Shigometer

Locations

Bearing Areas

Check the bearing areas for crushing of the beams near the bearing seat (see Figure 6.1.11). Investigate for decay and insect damage by visual inspection and sounding and/or probing at the ends of the beams where dirt, debris, and moisture tend to accumulate. Also verify the condition and operation of the bearing devices, if they are present (refer to Topic 9.1).

Figure 6.1.11 Bearing Area of Typical Solid Sawn Beam

Shear Zones

As discussed in Topic P.1, maximum shear occurs near supports. A horizontal shear force of equal magnitude accompanies the vertical shear component of this force. Because of timber's orthotropic cell structure, it has excellent resistance against vertical shear but low resistance against horizontal shear. The failure of a solid sawn timber member due to load is generally preceded by horizontal shear cracking along the grain. A horizontal shear "crack" is effectively a longitudinal split.

Investigate the area near the supports for the presence of horizontal shear cracking. The zones of maximum shear are at the ends of the beam. The presence of transverse cracks on the underside of the girders or horizontal cracks on the sides of the girders indicate the onset of shear failure. These cracks can propagate quickly toward midspan and represent lost moment capacity of up to 75%. (see Figure 6.1.12). Measure these cracks carefully for length and width width.

Figure 6.1.12 Horizontal Shear Crack in a Timber Beam

Tension Zones

Examine the zones of maximum tension for signs of structural distress. The maximum tension generally occurs at the bottom half of the middle third of the beam span. Investigate for section loss due to decay or fire, especially near mid-span and at the ends. Examine beams for excessive deflection or sagging. Tension cracks in timber break the cell structure perpendicular to the grain and are typically preceded by the appearance of horizontal shear cracks.

Solid sawn beams with sloping grain that intersects the surface in the tension zone are particularly susceptible to flexure cracking. This is because the tensile stress and horizontal shear stress combine to split the grain apart.

Areas Exposed to Drainage

Timber bridges with open plank decks are exposed throughout. Plank decks with asphalt overlays in good condition offer some protection. In these cases, deck joint areas at span ends are candidates for drainage exposure.

Investigate for signs of decay along the full length of the beam but especially where the beam is subjected to continual wetness and areas that trap moisture. These include member interfaces between deck planks and stringers, deck planks and beams, beams and bearing seats, stringers and floorbeams, floorbeams and trusses, truss member connections, arch connections, and all fastener locations. (see Figure 6.1.13).

Decay and chemical attack may be evidenced by discolored wood, brown and white rot, the formation of fruiting bodies (the result of fungal attacks, which produce disc-shaped bodies that distribute reproductive spores), "sunken" faces in the wood, or soft "punky" texture of the wood. When surface probing for expected decay is inconclusive, the next step is to drill the suspect area. If this has been done in a previous inspection, the drill hole area should be examined carefully for proper preservation treatment and dowel plug installations.

Figure 6.1.13 Decay in a Timber Beam

Areas of Insect Infestation

Insect infestation can be detected in various ways. Carpenter ants generally leave piles of sawdust; powder-post beetles leave small holes in the surface of the wood; and termites can often be readily seen. Another indication of insect infestation is hollow sounding wood. Further probing or drilling should be performed in suspect areas.

Areas Exposed to Traffic

For overhead and through structures, check for collision damage from vehicles passing below or adjacent to structural members.

Previous Repairs

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

Secondary Members

Inspect bracing members for decay and fire damage. Examine connections of bracing to beams for tightness, cracked or split members, and corroded, loose, or missing fasteners (see Figure 6.1.14).

Figure 6.1.14 Typical Timber End Diaphragm

Fasteners and Connectors

Check all fasteners (e.g., nails, screws, bolts, and deck clips) for corrosion. Also inspect for loose or missing fasteners.

6.1.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of timber bridges. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the Element Level 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 superstructure. Rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about the NBIS rating guidelines.

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. Smart Flags are also used to describe the condition of timber bridges.

In an element level condition state assessment of a solid sawn timber bridge, the AASHTO CoRe elements are:

Element No.	**Description**
111	Open Girder/Beam
117	Stringer
135	Truss/Arch
156	Floorbeam

The unit quantity for the timber superstructures is in meters or feet, and the total length must be placed in one of the available condition states. Condition state 1 is the best possible rating. See Topic 4.6 for condition state descriptions.

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.

This page intentionally left blank

Table of Contents

Section 6

Inspection and Evaluation of Common Timber Superstructures

This page intentionally left blank

Topic 6.2 Glulam Timber Bridges

6.2.1 Introduction

A glued-laminated (glulam) member is made by gluing strips of wood together to form a structural member of the desired size. An advantage of glulam members is that they allow for a higher utilization of the wood, since a lower grade of material can be used in of lower stress. Many strength reducing characteristics of wood, such as knots and checks, are minimized due to relatively small laminate dimensions. Also, the size and length of a glulam member is not limited by the size or length of a tree. Strips of wood used in glulam members are generally 20 to 40 mm (3/4 to 1-1/2 inches) thick (see Figures 6.2.1 and 6.2.2).

Figure 6.2.1 Elevation View of a Glulam Multi-beam Bridge

Figure 6.2.2 Underside View of a Glulam Multi-beam Bridge

6.2.2 Design Characteristics

Multi-beam Bridges

Glulam multi-beam bridges are very similar to solid sawn multi-beam bridges, but they generally use larger members to span greater distances. Glulam multi-beam bridges are typically simple span designs (see Figure 6.2.3). They usually support a deck consisting of glulam panels with a bituminous wearing surface. Beam sizes typically range from about 150 mm by 610 mm (6 inches by 24 inches) to 310 mm by 1525 mm (12-1/4 inches by 60 inches), and the beams are usually spaced 1.7 m to 2.0 m (5-1/2 feet to 6-1/2 feet) on center (see Figure 6.2.4).

These more modern multi-beam bridges can typically be used in spans of up to 24 m (80 feet), although some span as long as 46 m (150 feet). These too can be used to form longer multiple span structures. They are generally found on local and secondary roads, as well as in park settings.

Figure 6.2.3 Elevation View of Typical Glulam Multi-beam Bridge

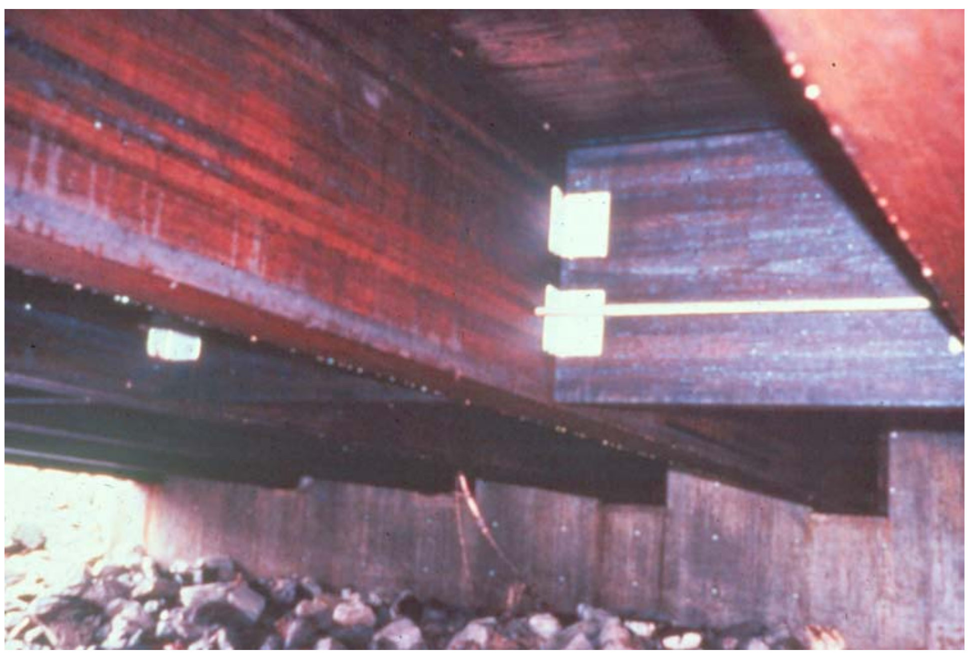


Figure 6.2.4 Underside View of Typical Glulam Multi-beam Bridge

Truss Bridges

Trusses may be of the through-type or of the deck-type. Usually the floor system consists of a timber deck supported by timber stringers and floorbeams, all of which are supported by the trusses (see Figures 6.2.5 and 6.2.6). Timber trusses are generally used for spans that are not economically feasible for timber multi-beam bridges. Timber trusses are practical for spans that range from 46 to 76 m (150 to 250 feet) (see Figure 6.2.7).

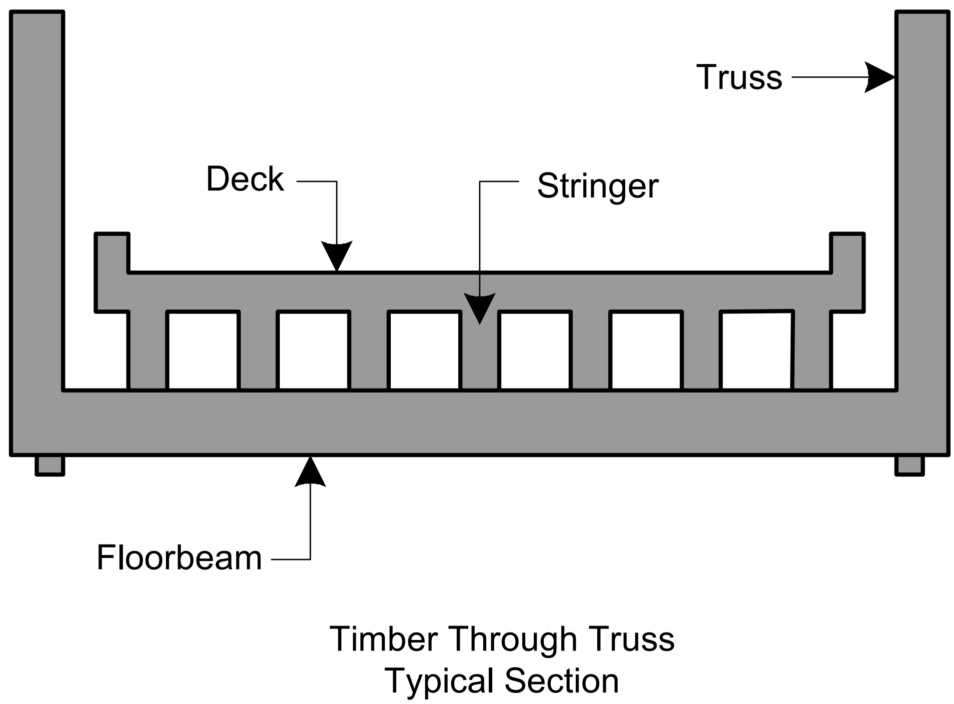


Figure 6.2.5 Timber Through Truss Typical Section

Figure 6.2.6 Bowstring Truss Pedestrian Bridge

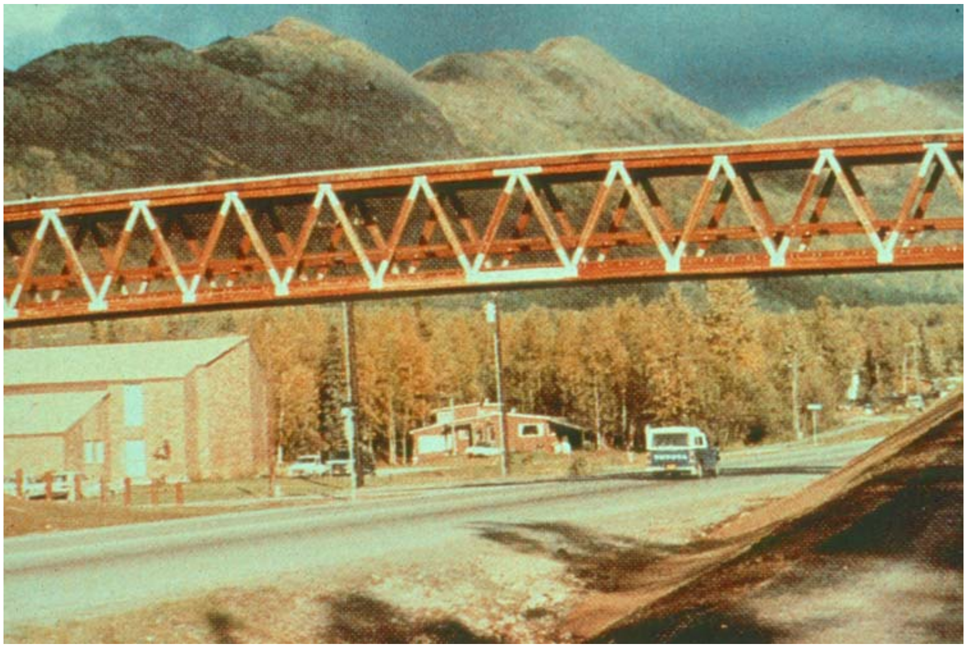


Figure 6.2.7 Parallel Chord Truss Pedestrian Bridge (Eagle River, Alaska)

Arch Bridges

Glulam arch bridges usually consist of two- or three-hinged deck arches, which support a glulam deck and floor system (see Figures 6.2.8 and 6.2.9). Glulam arches are practical for spans of up to about 91 m (300 feet). Although they are not widely used for highway bridges, they are frequently used for pedestrian overpasses and in locations such as parks where aesthetics is important.

Figure 6.2.8 Glulam Arch Bridge over Glulam Multi-beam Bridge (Keystone Wye interchange, South Dakota)

Figure 6.2.9 Glulam Arch Bridge (Colorado)

Primary and Secondary Members

The primary members of glulam multi-beam bridges are the beams, and the secondary members are the diaphragms or cross bracing (see Figures 6.2.10 and 6.2.11). Due to the larger depth of the glulam beams, diaphragms or cross bracing should always be present. Diaphragms are usually constructed of short glulam members, and cross bracing is usually constructed of steel angles.

The primary members of glulam arch and truss structures are the arch, truss, stringers, and floorbeams and spandrel bents. The secondary members are the diaphragms and cross bracing between the stringers and the lateral bracing between the arch or truss.

Figure 6.2.10 Elevation View of Typical Glulam Beam

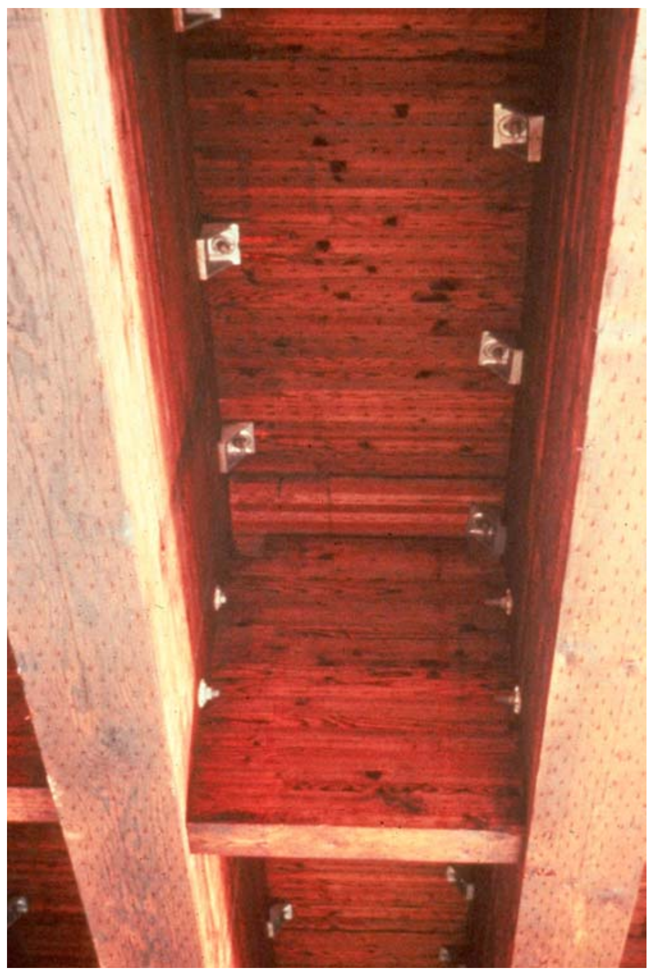


Figure 6.2.11 Typical Glulam Diaphragm

Recent technology has also produced glulam timber materials which are reinforced with fibers such as aramids, carbon, and fiberglass. These fiber reinforced glulam beams help increase the strength and mechanical properties of timber bridges.

6.2.3 Overview of Common Defects

Common defects that occur on glulam timber beams include:

- Checks, splits, and shakes
- Decay by fungi
- Damage by insects and borers
- Damage from impact/collisions
- Damage from abrasion/wear
- Damage from weathering/warping
- Damage from overstress

Other less common defects that may be encountered by the inspector include damage from chemical attack and damage from fire, which can be very destructive to timber structures. Refer to Topic 2.1 for a more detailed presentation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

6.2.4 Inspection Procedures and Locations

Since these superstructures are very similar to solid sawn superstructures, the inspection locations and procedures for glulam bridges are virtually the same as those for solid sawn bridges.

Procedures

Visual

The inspection of splits, cracks, shakes, fungus decay, deflections, crushing, delaminations, and loose connections is primarily a visual activity.

Physical

The physical examination of a glulam member can be conducted with a hammer or pick. The hammer is used to sound the members to detect hollow areas or internal decay. Picks are used to determine the condition of the surface

Advanced Inspection Techniques

In addition, several advanced techniques are available for timber inspection. Nondestructive methods, described in Topic 13.1.2, include:

- Pol-Tek
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 13.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Shigometer

Locations

Bearing Areas

Inspect the bearing areas for crushing of the beams (see Figure 6.2.12). Investigate for decay and insect damage by visual inspection, sounding, and/or probing at the ends of the beams. Also check the condition and operation of the bearing devices if they are present (refer to Topic 9.1).

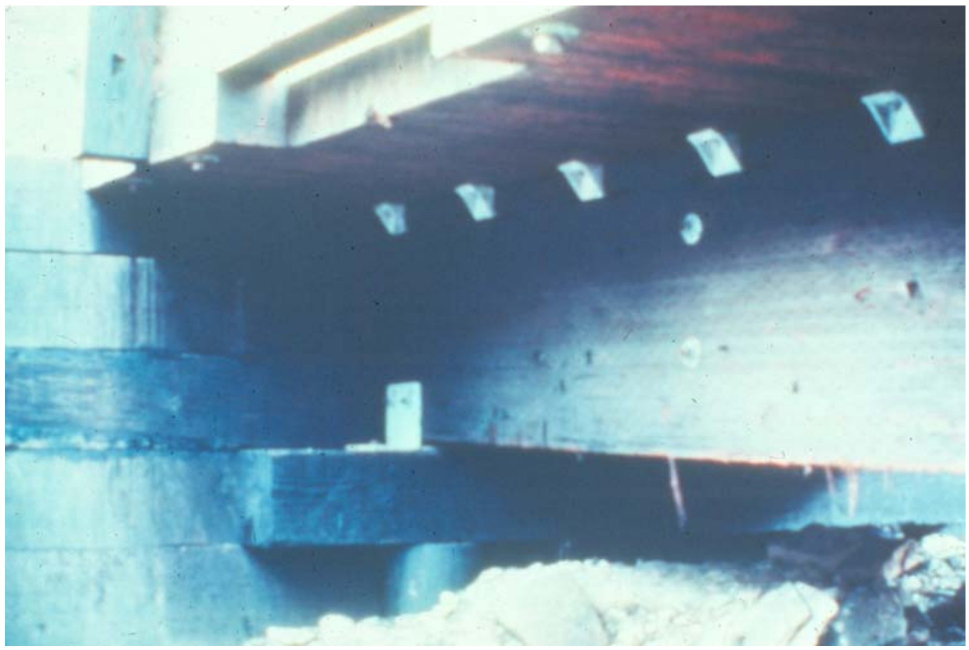


Figure 6.2.12 Bearing Area of Typical Glulam Beam

Shear Zones

Examine for horizontal shear cracks and delaminations near the ends of the beam. Delaminations (i.e., separations in the laminations) can occur due to either failure of the glue or failure at the bond between the glue and the lamination (see Figure 6.2.13). Delaminations that extend completely through the cross section of the member are the most serious since this makes the member act as two smaller members. Delaminations that are located near the center of the cross section are more serious than those that are not. Delaminations directly through a connector are also undesirable.

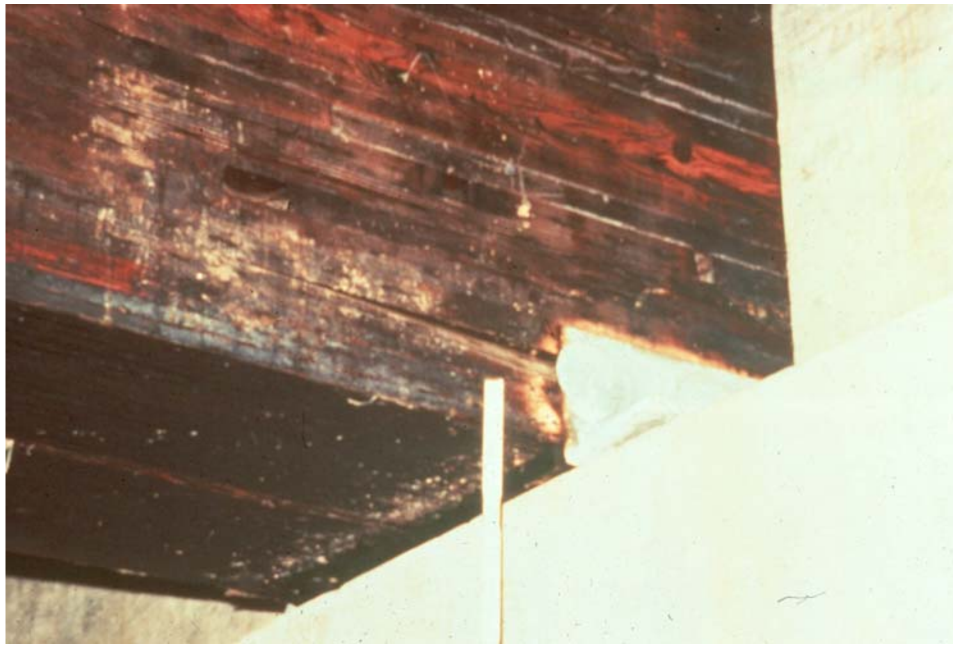


Figure 6.2.13 Close-up View of End of Glulam Bridge Showing Laminations

Tension Zones

Examine the zone of maximum tension for signs of structural distress (see Figure 6.2.14). The maximum tension generally occurs at the bottom half of the middle third of the beam span. Investigate for section loss due to decay or fire, especially near mid-span and at the ends. Inspect for excessive deflection or sagging in the beams.

Figure 6.2.14 Elevation View of Beam of Glulam Multi-beam Bridge

Areas Exposed to Drainage

Investigate for signs of decay along the full length of the member but especially where the beam is subjected to continual wetness or prolonged exposure to moisture (see Figure 6.2.15). Decay and chemical attack may be evidenced by discolored wood, brown and white rot, the formation of fruiting bodies (the result of fungal attacks, which produce disc-shaped bodies that distribute reproductive spores), "sunken" faces in the wood, or the soft “punky” texture of the wood.

Figure 6.2.15 Decay on Glulam Beam

Areas of Insect Infestation

Insect infestation can be detected in various ways. Carpenter ants generally leave piles of sawdust; powder-post beetles leave small holes in the surface of the wood; and termites can often be readily seen. Another indication of insect infestation is hollow sounding wood. Further probing or drilling should be performed in suspect areas.

Areas Exposed to Traffic

Check beams in overhead structures for collision damage from vehicles passing below.

Previous Repairs

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

Secondary Members

Examine solid sawn or glulam diaphragms for decay, fire damage, and insect damage (see Figure 6.2.16). Check steel cross bracing for corrosion, bowing, or buckling (see Figure 6.2.17). Examine connections for tightness, cracks and splits, and corroded, loose, or missing fasteners.

Figure 6.2.16 Typical Diaphragm for a Glulam Multi-beam Bridge

Fasteners and connectors

Inspect all fasteners for corrosion, tightness, and missing parts (see Figure 6.2.17).

Figure 6.2.17 Glulam Beams with Numerous Fastener Locations

6.2.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of timber bridges. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the Element Level 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 superstructure. Rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBIS rating guidelines.

The previous inspection data should be 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. Smart Flags are also used to describe the condition of timber bridges.

In an element level condition state assessment of a glulam timber bridge, the AASHTO CoRe elements are:

Element No.	Description
111	Open Girder/Beam
117	Stringer
135	Truss/Arch
156	Floorbeam

The unit quantity for the timber superstructures is in meters or feet, and the total length must be placed in one of the available condition states. 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 traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned.

This page intentionally left blank

Table of Contents

Section 6
Inspection and Evaluation of Common Timber Superstructures

This page intentionally left blank

Topic 6.3 Stress-Laminated Timber Bridges

6.3.1 Introduction

Stress-laminated timber bridges were first developed in Canada, in 1976, by the Ontario Ministry of Transportation and Communications. These bridges consist of multiple planks mechanically clamped together using metal rods to perform as one unit (see Figure 6.3.1). The compression induced frictional resistance within the timber laminations is the mechanism that makes this structural system effective.

Figure 6.3.1 Stressed-Laminated Timber Deck Bridge Carrying a 90,000-Pound Logging Truck (Source: Barry Dickson, West Virginia University)

6.3.2 Design Characteristics

Stress-Laminated Timber Deck/Slab Bridges

Sawn lumber stress-laminated deck bridges can be used for simple spans of up to 15 m (50 feet) and are capable of carrying modern highway loadings (see Figures 6.3.1 and 6.3.3). Stressed deck bridges have also been constructed using glulam members. Combining glulam technology with stress-lamination increases practical span lengths to 19 m (63 feet) (see Figure 6.3.4).

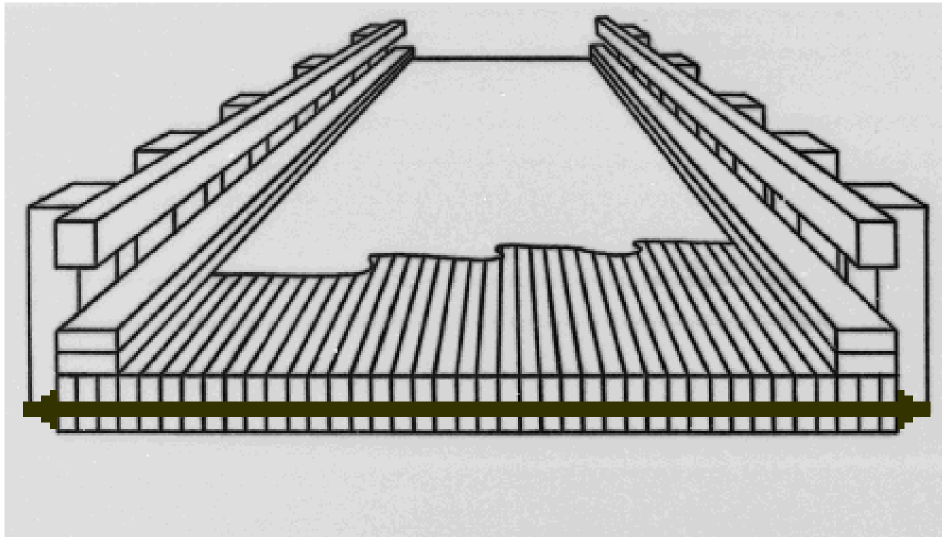


Figure 6.3.2 Typical Section of a Stress-Laminated Timber Deck Bridge

Figure 6.3.3 Solid Sawn Stress-Laminated Deck/Slab Bridge

Figure 6.3.4 Glulam Stress-Laminated Deck/Slab Bridge

Stress-Laminated Timber Tee Beam Bridges

The idea for stress-laminated timber tee beam bridges was developed at West Virginia University. These bridges consist of a stress-laminated deck and glulam beams (see Figure 6.3.6). High strength steel rods are used to join the stress-laminated deck and glulam beams together to form stress-laminated timber tee beams. The first structure of this type was built in 1988, near Charleston, West Virginia. It is about 23 m (75 feet) long and has stressing rods spaced at two feet. It has performed well so far, and stressed tee beams will likely be used in the future to achieve even longer span lengths (see Figure 6.3.7).

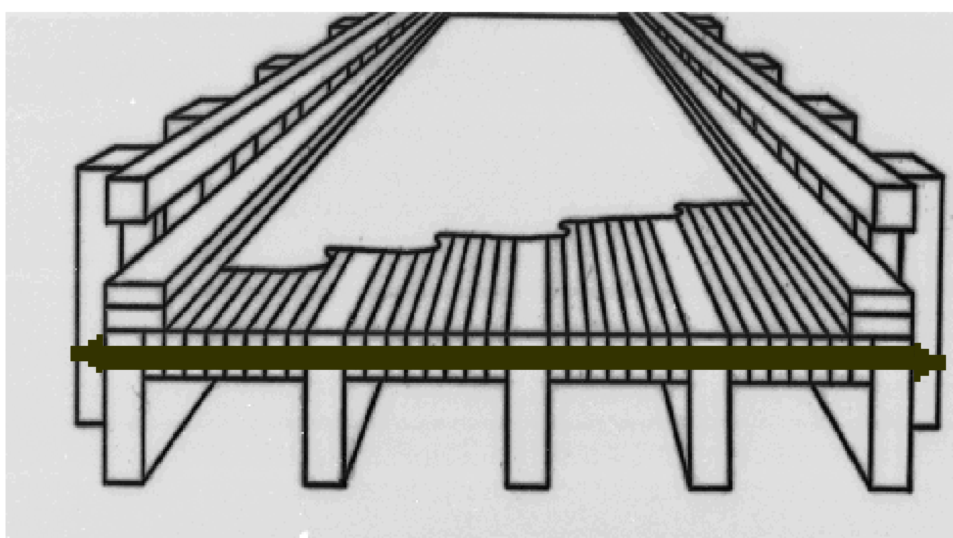


Figure 6.3.5 Typical Section of a Stress-Laminated Timber Tee Beam Bridge (Source: Barry Dickson, West Virginia University)

Figure 6.3.6 Elevation View of Stress-Laminated Timber Tee Beam Bridge (West Virginia)

Stress-Laminated Timber Box Beam Bridges

Stress-laminated timber box beam bridges represent further development of timber bridges by West Virginia University. These bridges consist of adjacent box beam panels individually comprised of stress-laminated flanges and glulam beam webs (see Figure 6.3.8). This bridge type is also known as a cellular stressed deck. Span lengths of up to 18 m (60 feet) have been designed, and there is a potential for longer spans (see Figure 6.3.9).

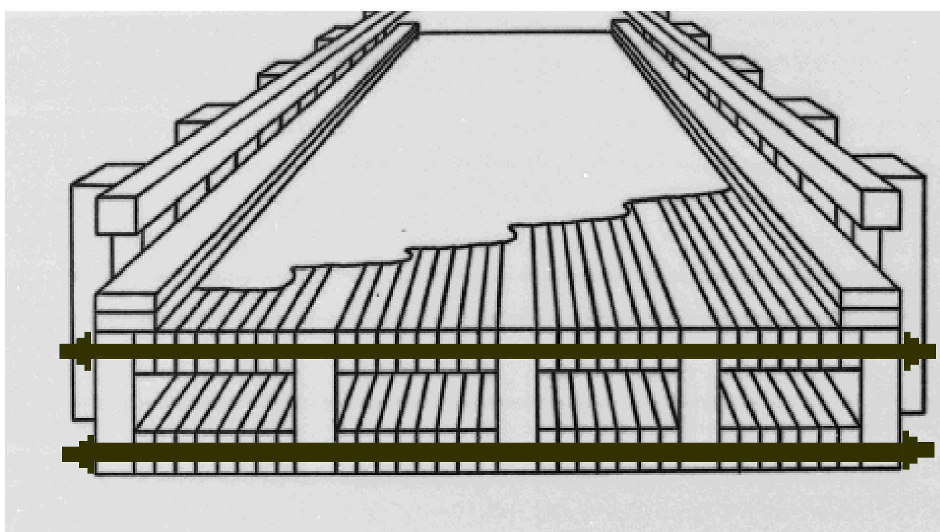


Figure 6.3.7 Typical Section of a Stress-Laminated Timber Box Beam (Source: Barry Dickson, West Virginia University)

Figure 6.3.8 Stress-Laminated Timber Box Beam Bridge Being Erected

Stress-Laminated Timber K-frame Bridges

Stressed K-frame bridges represent further development of the stressed deck bridge by the Ontario Ministry of Transportation and Communications. These bridges consist of three spans in which the stressed deck is supported at two intermediate points by stressed laminated timber struts (see Figure 6.3.5). This bridge type has been used for a bridge with a total length of 13 m (43 feet), and it has a potential for span lengths over 15 m (50 feet).

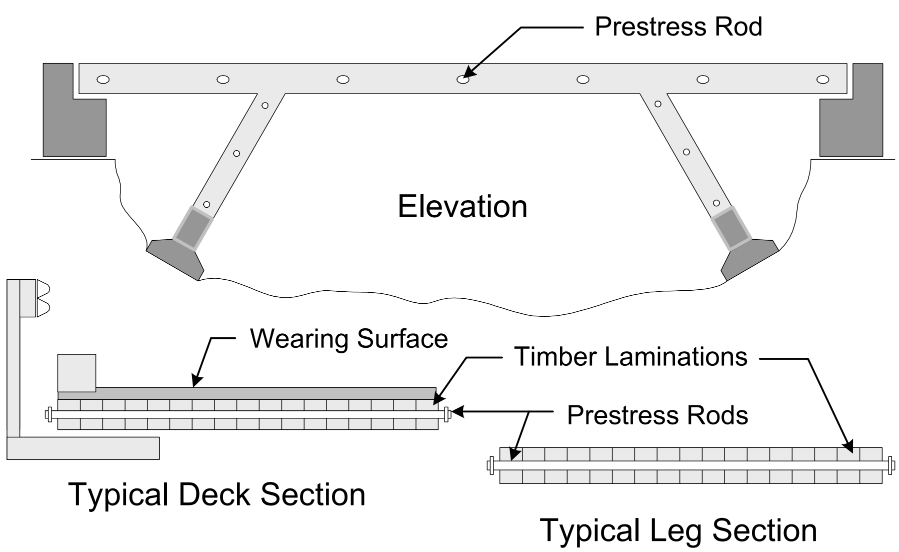


Figure 6.3.9 Stress-Laminated Timber K-frame Bridge

Primary and Secondary Members

The primary members are the decks, tee beams, and box beams. The secondary members are the diaphragms and cross bracing between beams.

6.3.3 Overview of Common Defects

Common defects that occur on stressed timber bridges include:

- Checks, splits, and shakes
- Decay by fungi
- Damage by insects and borers
- Damage from impact/collisions
- Damage from abrasion/wear
- Damage from weathering/warping
- Damage from overstress

Other less common defects that may be encountered by the inspector include damage from chemical attack and damage from fire, which can be very destructive to timber structures. Refer to Topic 2.1 for a more detailed presentation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

6.3.4 Inspection Procedures and Locations

The inspection locations and procedures for stressed timber bridges are similar to those for glulam bridges.

Procedures

Advanced Inspection Techniques

In addition, several advanced techniques are available for timber inspection. Nondestructive methods, described in Topic 13.1.2, include:

- Pol-Tek
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 13.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Shigometer

Locations

Stressing Rods

Examine the condition of the steel stressing rods, and inspect for crush and splits in the fascia members. Check for loss of prestress in the rods, which would be indicated by shifted planks in the stress-laminated timber element and excessive deflection. This may be observed when the bridge is subject to a moving live load.

Figure 6.3.10 Broken Stressing Rods

Bearing Areas

Inspect the bearing areas for crushing of the beams. Investigate for decay and insect damage by visual inspection, sounding, and/or probing at the ends of the beams. Also check the condition and operation of the bearing devices if they are present (refer to Topic 9.1, Bearings).

Shear Zones

Examine for horizontal shear cracks and delaminations near the ends of the beam. Delaminations (i.e., separations in the laminations) can occur due to either failure of the glue or failure at the bond between the glue and the lamination (see Figure 6.3.11). Delaminations that extend completely through the cross section of the member are the most serious since this makes the member act as two smaller members.

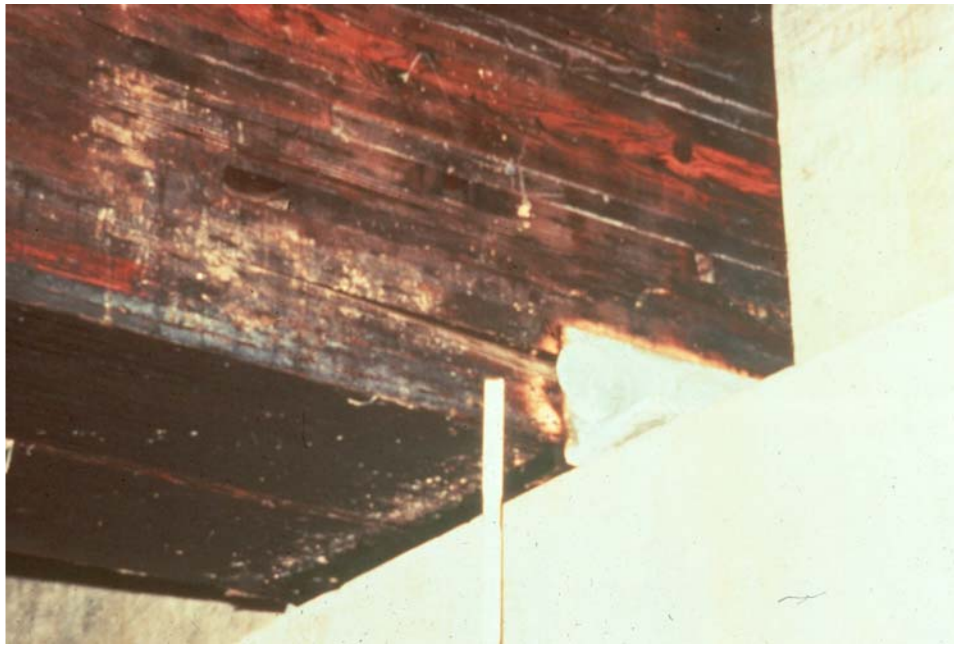


Figure 6.3.11 Close-up View of End of a Stressed Timber Bridge Showing Laminations

Tension Zones

Examine the zone of maximum tension for signs of structural distress. The maximum tension generally occurs at the bottom half of the middle third of the beam span. Investigate for section loss due to decay or fire, especially near mid-span and at the ends. Inspect for excessive deflection or sagging in the beams.

Areas Exposed to Drainage

Investigate for signs of decay along the full length of the member but especially where the beam is subjected to continual wetness or prolonged exposure to moisture. Decay and chemical attack may be evidenced by discolored wood, brown and white rot, the formation of fruiting bodies (the result of fungal attacks, which produce disc-shaped bodies that distribute reproductive spores), "sunken" faces in the wood, or the soft “punky” texture of the wood.

Areas of Insect Infestation

Insect infestation can be detected in various ways. Carpenter ants generally leave piles of sawdust; powder-post beetles leave small holes in the surface of the wood; and termites can often be readily seen. Another indication of insect infestation is hollow sounding wood. Further probing or drilling should be performed in suspect areas.

Areas Exposed to Traffic

Check beams in overhead structures for collision damage from vehicles passing below.

Previous Repairs

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

Secondary Members

Examine solid sawn or glulam diaphragms for decay, fire damage, and insect damage. Check steel cross bracing for corrosion, bowing, or buckling. Examine connections for tightness, cracks and splits, and corroded, loose, or missing fasteners.

Fasteners and Connectors

Inspect all fasteners for corrosion, tightness, and missing parts. Stressing rod hardware is the most important fastener system on a stress-laminated timber bridge.

Further development of the stressed timber bridge concept is being performed at the University of Wisconsin, West Virginia University, and Pennsylvania State University, as well as in Canada.

6.3.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of timber bridges. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the Element Level 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 superstructure. Rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBIS rating guidelines.

The previous inspection data should be 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 timber bridges.

In an element level condition state assessment of timber bridges, the AASHTO CoRe elements are:

Element No.	Description
11	Timber Deck - Bare
12	Timber Deck with A/C Overlay
54	Timber Slab - Bare
55	Timber Slab with A/C Overlay
111	Open Girder/Beam

The unit quantity for the timber superstructures is in meters (or feet), and the total length must be placed in one of the four available condition states. The unit quantity of decks and slabs is "each", and the entire element must be placed in one of the five available condition states. Some states have elected to use the total square area (m^2 or ft^2). 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 traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned.

Table of Contents

Section 7

Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Section 7

Inspection and Evaluation of Common Concrete Superstructures

Topic 7.1 Cast-in-Place Slabs

7.1.1 Introduction

The cast-in-place slab bridge is the simplest type of reinforced concrete bridge and was a common choice for construction in the early 1900's (see Figures 7.1.1 and 7.1.2). Sometimes the terms "deck" and "slab" are used interchangeably to describe the same bridge component. However, this is incorrect. A deck is supported by a superstructure unit (beams, girders, etc.), whereas a slab is a superstructure unit supported by a substructure unit (abutments, piers, bents, etc.). A deck can be loosely defined as the top surface of the bridge, which carries the traffic. A slab serves as the superstructure and the top surface that carries the traffic. Even though slabs are defined differently than decks, many of the design characteristics, wearing surfaces, protective systems, inspection procedures and locations and, evaluation, are similar. See Topic 5.2 for further details.

Figure 7.1.1 Typical Simple Span Cast-in-Place Slab Bridge

Figure 7.1.2 Typical Multi-span Cast-in-Place Slab Bridge

7.1.2 Design Characteristics

General

The slab bridge functions as a wide, shallow superstructure beam that doubles as the deck. This type of bridge generally consists of one simply supported span and is typically less than 9 m (30 feet) long. Simple and continuous multi-span slab bridges are also common. The only primary member in a cast-in-place slab bridge is the slab itself.

Steel Reinforcement

For simple spans, the slab develops only positive moment; therefore, the primary, or main tension reinforcement is located in the bottom of the slab. The reinforcement is placed longitudinally, or from support to support, parallel to the direction of traffic. For continuous spans, additional primary reinforcement is located longitudinally in the top of the slab over the piers to resist negative bending moments.

Secondary reinforcement, known as temperature and shrinkage steel, is located transversely throughout the top and bottom of the slab. In simple span slabs, secondary reinforcement is also located longitudinally in the top of the slab. In continuous span slabs, the primary reinforcement is often placed the full structure length, negating the need for longitudinal secondary reinforcement.

Nearly all slab bridges have a grid or mat of steel reinforcement in both the top and bottom of the slab that is formed by some combination of primary and secondary reinforcement (see Figure 7.1.3).

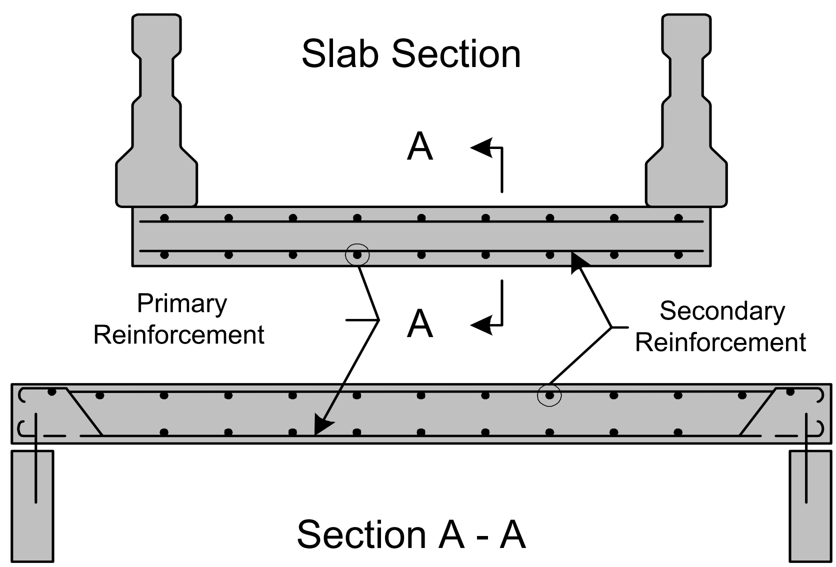


Figure 7.1.3 Steel Reinforcement in a Concrete Slab

7.1.3 Overview of Common Defects

Common defects that occur on cast-in-place slab bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear

- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.1.4 Inspection Procedures and Locations

Inspecting a cast-in-place slab bridge is similar to the procedure discussed in Topic 5.2.6, Concrete Decks, and includes the following specific procedures:

Procedures

Visual

The inspection of concrete slabs for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination of a slab with a hammer can be a tedious operation. In most cases, a chain drag is used. A chain drag is made of several sections of chain attached to a pipe that has a handle attached to it. The inspector drags this across a slab and makes note of the resonating sounds. A chain drag can usually cover about a 900 mm (3 feet) wide section of slab at a time (see Figure 5.2.8).

If the inspector deems it necessary, core samples can be taken from the slab and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete bridge slabs are caused by corrosion of the rebar. When the deterioration of a concrete slab progresses to the point of needing rehabilitation, an in-depth inspection of the slab 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, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

Examine bearing areas for spalling where friction from thermal movement and high edge or bearing pressure could cause the concrete to spall (see Figure 7.1.4).

Figure 7.1.4 Steel Rocker Bearing Supporting Haunched Slab at Pier

Shear Zones

Investigate areas near the supports for shear cracking. The presence of transverse cracks on the underside near supports or diagonal cracks on the sides of the slab indicate the onset of shear failure (see Figures 7.1.5 and 7.1.6). These cracks represent lost shear capacity and should be carefully measured.

Figure 7.1.5 Shear Cracks in the Ends of a Slab Bridge

Figure 7.1.6 Shear Zone on the Underside of a Continuous Slab Bridge Near a Pier

Tension Zones

Tension zones should be examined for flexure cracks, which would be vertical on the sides and transverse across the slab. The tension zones are at midspan along the bottom of the slab for both simple and continuous span bridges. Additional tension zones are located on top of the slab over the piers for continuous spans. Cracks greater that 2 mm (1/16 inch) wide are considered wide cracks and indicate extreme bending stresses. Check for efflorescence from cracks and discoloration of the concrete caused by rust stains from the reinforcing steel. In severe cases, the reinforcing steel may become exposed due to spalling. Document the remaining cross section of reinforcing steel since section loss will decrease live load capacity.

Check for deteriorated concrete near the tension zones, which could result in the debonding of the tension reinforcement. This would include delamination, spalls, and contaminated concrete (see Figure 7.1.7). Slab bridges which use hooks to develop the primary reinforcement are not as susceptible to debonding due to deterioration of the concrete.

Figure 7.1.7 Delamination and Efflorescence with Rust. Stains on Slab Underside in Tension Zone

Areas Exposed to Drainage

Inspect areas exposed to roadway drainage for deteriorated concrete. This includes the entire riding surface of the slab, particularly around scuppers or drains. Spalling or scaling may also be found along the curbline and fascias (see Figure 7.1.8).

Areas Exposed to Traffic

For grade crossing structures, check areas exposed to traffic for damage caused by collision. Such damage will generally consist of corner spalls and may include exposed rebars.

Skewed Bridges

Examine skewed bridges for lateral displacement and cracking of acute corners due to point loading and insufficient reinforcement.

Figure 7.1.8 Deteriorated Slab Fascia due to Roadway Deicing Agents

7.1.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are 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). For a slab bridge, these guidelines must be applied for both the deck component and the superstructure component.

The previous inspection data should be used along with current inspection findings to determine the correct rating. Typically, for this type of structure, the deck and superstructure components will have the same 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 top of slab and the underside. 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 concrete superstructure.

In an element level condition state assessment of a slab bridge, the AASHTO CoRe element is one of the following, depending on the riding surface:

Element No.	Description
38	Concrete Slab – Bare
39	Concrete Slab – Unprotected with AC Overlay

40	Concrete Slab – Protected with AC Overlay
44	Concrete Slab – Protected with Thin Overlay
48	Concrete Slab – Protected with Rigid Overlay
52	Concrete Slab – Protected with Coated Bars
53	Concrete Slab – Protected with Cathodic System

The unit quantity for these elements is "each", and the entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2). Condition state 1 is the best possible rating. The inspector must know the total slab surface area in order to calculate a percent deterioration and fit into a given condition state description. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For structural cracks in the surface of bare slabs, the "Deck Cracking" Smart Flag, Element No. 358, can be used and one of four condition states assigned. Do not use Smart Flag, Element No. 358, if the bridge deck/slab has any overlay because the top surface of the structural deck is not visible. For concrete defects on the underside of a slab element, the "Soffit" Smart Flag, Element No. 359, can be used and one of five condition states assigned.

This page intentionally left blank

Table of Contents

Section 7
Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Topic 7.2 Tee Beams

7.2.1 Introduction

The concrete tee beam, a predominant bridge type during the 1930's and 1940's, is generally a cast-in-place monolithic slab and stem system formed in the shape of the letter "T."

The cast-in-place tee beam is the most common type of tee beam. However, precast tee beam shapes are used by some highway agencies. Types of precast tee beams include bulb tee, double tee, quad tee, and rib tee.

Recent technology has also produced the inverted tee beam, a new type of precast tee beam, for short to medium span bridges. Developed in Nebraska, this prestressed concrete beam reduces the weight up to 20% compared to conventional I-beams and eliminates the need for falsework construction.

7.2.2 Design Characteristics

General

Spacing of the tee beams is generally 900 to 2400 mm (3 to 8 feet), center-to-center of beam stems. The depth of the stems is generally 450 to 600 mm (18 to 24 inches). Simple span design was most common but continuous span designs were popular in some regions. A 75 or 100 mm (3 or 4 inch) fillet at the slab-stem intersection identifies this older form of construction (see Figure 7.2.2).

Care must be taken not to describe tee beam bridges as composite. They do not meet the definition of composite, because the slab and stem are constructed of the same material. The slab portion of the beam is constructed to act integrally with the stem, providing greater stiffness and allowing increased span lengths. The tee beam bridge is used for spans between 9 and 15 m (30 and 50 feet). Simple spans are most common; however, there are some multi-span continuous tee beam bridges in use (see Figure 7.2.1).

Figure 7.2.1 Multi-span, Simply Supported Tee Beam Bridge

Figure 7.2.2 Typical Tee Beam with Fillet

The inspector should be careful not to mistake a concrete encased steel I-beam bridge for a tee beam bridge. A review of the structure file should eliminate this problem. If necessary, a dimensional evaluation will show the encased steel beams to be smaller in size.

Figure 7.2.3 Concrete Encased Steel I-beam

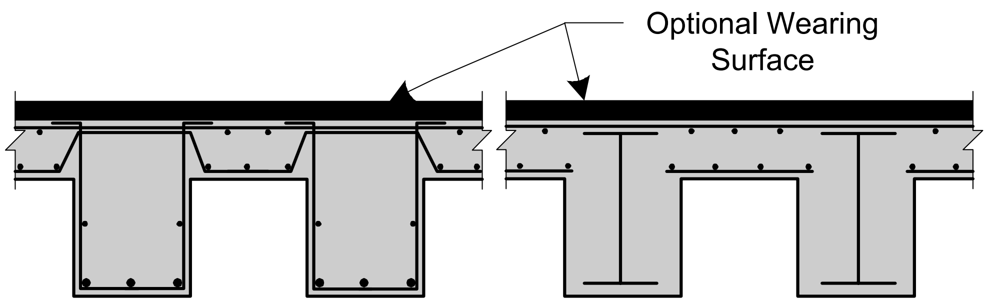


Figure 7.2.4 Comparison Between Tee Beam and Concrete Encased Steel I-beam

Primary Members and Secondary Members

The primary members of a tee beam bridge are the tee beam stem (web) and slab (flange) (see Figure 7.2.5).

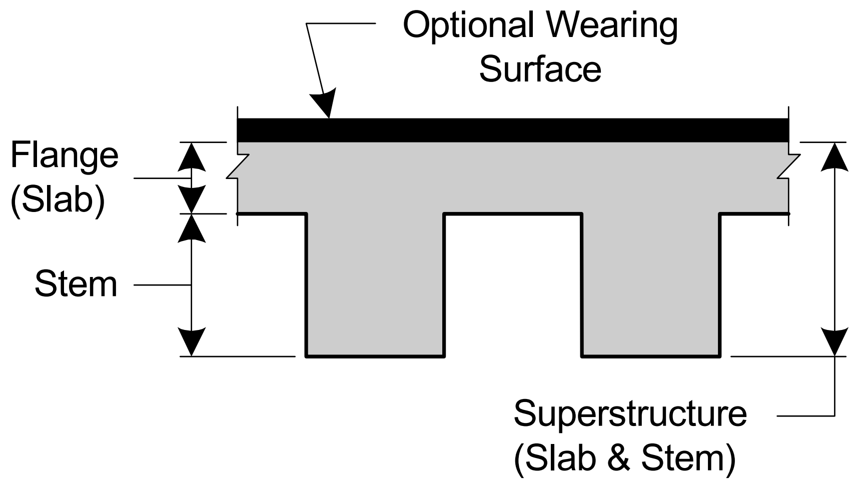


Figure 7.2.5 Tee Beam Cross Section

The only secondary members on a cast-in-place tee beam bridge are the diaphragms, which support the free edge of the beam flanges. Intermediate diaphragms may also be present in longer span bridges and are usually located at the half or third points along the span.

The diaphragms are designed as simple beams and should be inspected for flexure and shear cracks, as well as for typical concrete defects.

Figure 7.2.6 Tee Beam Diaphragms

Steel Reinforcement

The primary (tension) reinforcing steel consists of main tension reinforcement and shear reinforcement or stirrups. The main tension reinforcement is located in the bottom of the beam stem and oriented longitudinally (see Figure 7.2.7). If the concrete tee beams are continuous, there will be longitudinal reinforcement close to the top surface of the slab over the piers. The sides of the stem contain primary vertical shear reinforcement, called stirrups, and are located throughout the length of the stem at various spacings required by design. Stirrups are generally U-shaped bars and run transversely across the bottom of the stem (see Figure 7.2.7). The need for stirrups is greatest near the beam supports where shear stresses are the highest.

The secondary (temperature and shrinkage) reinforcing steel for the stem is oriented longitudinally in the sides (see Figure 7.2.7). The primary and secondary reinforcing steel for the slab portion of the beam is the same as for a standard concrete slab.

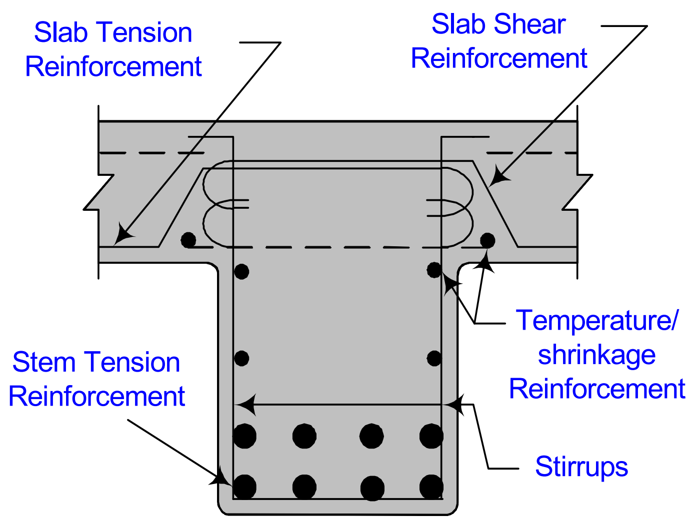


Figure 7.2.7 Steel Reinforcement in a Concrete Tee Beam

7.2.3 Overview of Common Defects

Common defects that occur on concrete tee beam bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.2.4 Inspection Procedures and Locations

Inspecting a tee beam bridge is similar to the procedure discussed in Topic 5.2.6 and includes the following specific procedures:

Procedures

Visual

The inspection of concrete tee beams for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination of a tee beam with a hammer can be a tedious operation. In most cases, a chain drag is used. A chain drag is made of several sections of chain attached to a pipe that has a handle attached to it. The inspector drags this across a slab and makes note of the resonating sounds. A chain drag can usually cover about a 900 mm (3 feet) wide section of slab at a time (see Figure 5.2.8).

If the inspector deems it necessary, core samples can be taken from the tee beam and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete tee beams are caused by corrosion of the rebar. When the deterioration of a concrete tee beam progresses to the point of needing rehabilitation, an in-depth inspection of the tee beam 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

Examine bearing areas for spalling where friction from thermal movement and high bearing pressure could cause the concrete to spall. Check for crushing of the stem near the bearing seat. Check the condition and operation of any bearing devices (see Figures 7.2.8 through 7.2.11).

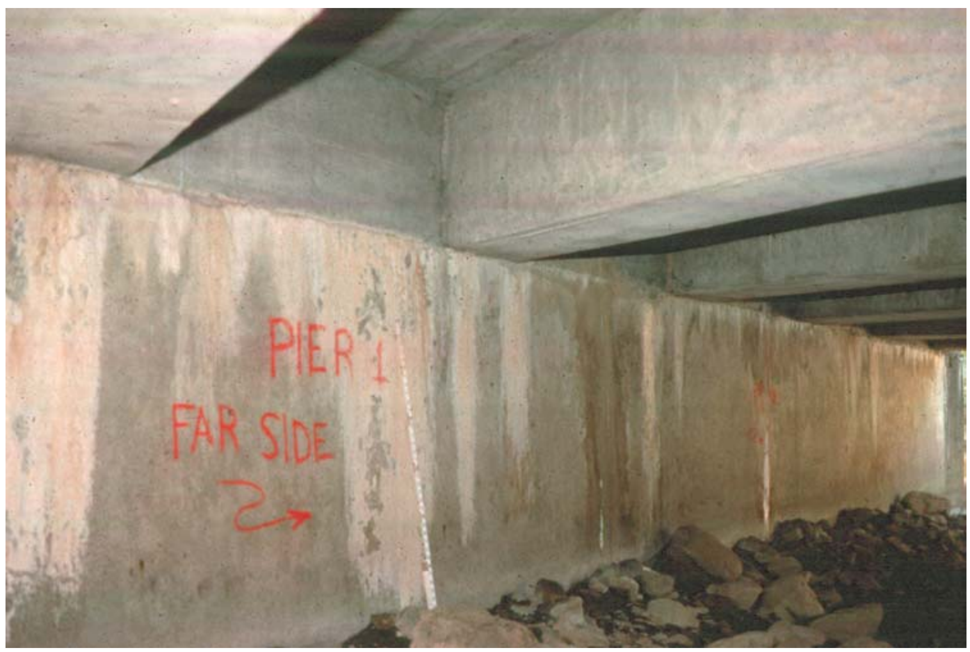


Figure 7.2.8 Bearing Area of Typical Cast-in-Place Concrete Tee Beam Bridge

Figure 7.2.9 Spalled Tee Beam End

Figure 7.2.10 Deteriorated Tee Beam Bearing Area

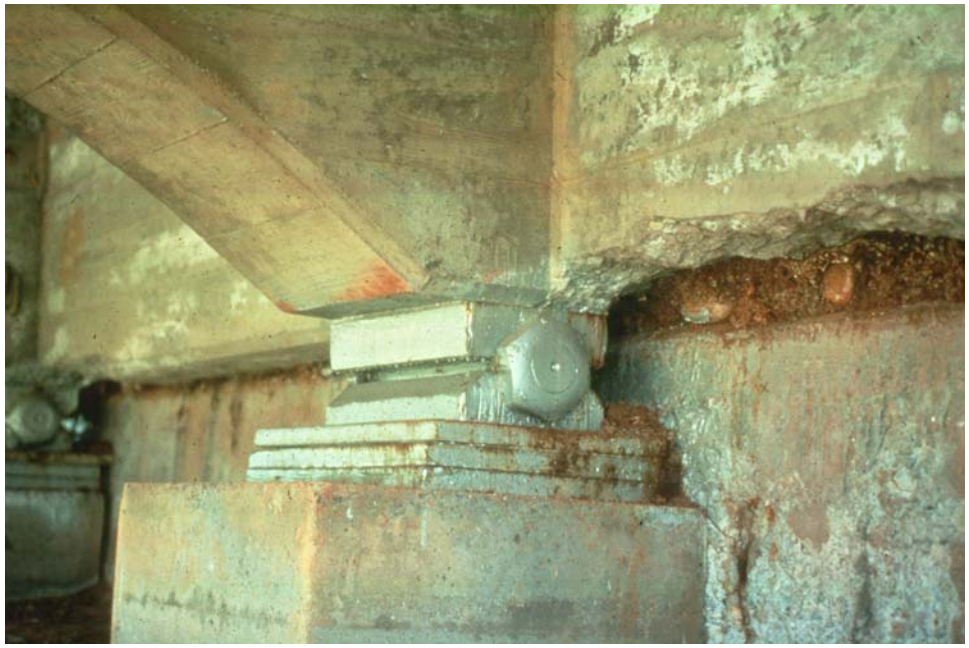


Figure 7.2.11 Steel Bearing Supporting a Cast-in-Place Concrete Tee Beam

Shear Zones

Investigate the area near the supports for the presence of shear cracking. The presence of transverse cracks on the underside of the stems or diagonal cracks on the sides of the stem indicate the onset of shear failure. These cracks represent lost shear capacity and should be carefully measured.

Figure 7.2.12 Shear Zone of Cast-in-Place Concrete Tee Beam Bridge

Tension Zones

Tension zones should be examined for flexure cracks, which would be vertical on the sides and transverse across the bottom of the stem (see Figure 7.2.13). The tension zones are at the midspan along the bottom of the stem for both simple and continuous span bridges. Additional tension zones are located on the slab over the piers for continuous spans (see Figure 7.2.14). Cracks greater that 2 mm (1/16 inch) wide are considered wide cracks and indicate extreme bending stresses. Check for efflorescence from cracks and discoloration of the concrete caused by rust stains from the reinforcing steel (see Figure 7.2.15). In severe cases, the reinforcing steel may become exposed due to spalling. Document the remaining cross section of reinforcing steel since section loss will decrease live load capacity (see Figure 7.2.16).

Figure 7.2.13 Flexure Cracks on a Tee Beam

Figure 7.2.14 Flexure Cracks in Tee Beam Slab

Check for deteriorated concrete near the tension zones, which could result in the debonding of the tension reinforcement. This would include delamination, spalls, and contaminated concrete (see Figure 7.2.15).

Figure 7.2.15 Stem of a Cast-in-Place Concrete Tee Beam with Contaminated Concrete

Figure 7.2.16 Spall on the Bottom of the Stem of a Cast-in-Place Tee Beam with Corroded Main Steel Exposed

Areas Exposed to Drainage

If the roadway surface is bare concrete, check for delamination, scaling, and spalls. The curb lines are most suspect. If the slab has an asphalt wearing surface, check for indications of deteriorated concrete such as reflective cracking and depressions (see Figure 7.2.17).

Figure 7.2.17 Asphalt Covered Tee Beam Slab

Check around scuppers or drain holes and slab fascias for deteriorated concrete (see Figure 7.2.18).

Figure 7.2.18 Deteriorated Tee Beam Stem Adjacent to Drain Hole

Check areas exposed to drainage for concrete spalling or cracking. This may occur at the ends of the stems where drainage has seeped through the slab joints (see Figure 7.2.19).

Figure 7.2.19 Deteriorated Tee Beam End Due to Drainage

Areas Exposed to Traffic

Check areas damaged by collision. Document the number of exposed and severed reinforcing bars as well as the amount of concrete and steel section loss. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss of the reinforcement bars (see Figure 7.2.20).

Figure 7.2.20 Tee Beam Bridge Over a Highway

Examine areas that have been previously repaired. Determine if the repairs are in place, and if they are functioning properly.

7.2.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guidelines systems currently in use are 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. For concrete tee beams, the slab condition influences the superstructure component rating. When the slab component rating is 4 or less, the superstructure component rating may be reduced if the recorded slab defects reduce its ability to carry applied stresses associated with superstructure moments.

Application of Condition State Assessment (Element Level Inspection - Pontis)

A narrative description with quantities is required in the first part of the inspection. Condition state summaries are then developed for the top of deck and the underside. 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 concrete superstructure.

In an element level condition state assessment of a tee beam bridge, the AASHTO CoRe element is one of the following, depending on the riding surface:

Element No.	Description
12	Concrete Deck – Bare
13	Concrete Deck– Unprotected with AC Overlay
14	Concrete Deck – Protected with AC Overlay
18	Concrete Deck – Protected with Thin Overlay
22	Concrete Deck – Protected with Rigid Overlay
26	Concrete Deck – Protected with Coated Bars
27	Concrete Deck – Protected with Cathodic System
110	Concrete Open Girder/Beam

The unit quantity for the deck elements is "each", and for the tee beam it is linear meters or feet. The entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2). The inspector must know the total deck surface area in order to calculate a percent deterioration and fit into a given condition state description. 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. Condition state 1 is the best possible rating for the tee beam. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For structural cracks in the surface of bare decks, the "Deck Cracking" Smart Flag, Element No. 358, can be used and one of four condition states assigned. Do not use Smart Flag, Element No. 358, if the bridge deck/slab has any overlay because the top surface of the structural deck is not visible. For concrete defects on the underside of a deck element, the "Soffit" Smart Flag, Element No. 359, can be used and one of five 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.

This page intentionally left blank

Table of Contents

Section 7

Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Topic 7.3 Concrete Girders

7.3.1 Introduction

Concrete girder bridges generally consist of cast-in-place monolithic decks and girder systems. Concrete girders can be used as deck girders, where the deck is cast on top of the girders (Figure 7.3.1), or as through girders, where the deck is cast between the girders. Through girders are very large in appearance and actually serve as the bridge's parapets, as well as the main supporting members (see Figure 7.3.2). Many of these bridges in service today were built in the 1940's.

Figure 7.3.1 Concrete Deck Girder Bridge

Figure 7.3.2 Concrete Through Girder Bridge

7.3.2 Design Characteristics

General

The deck slab does not contribute to the strength of the girders and serves only to distribute traffic loads to the girders. As such, the superstructure condition rating is not affected by the condition of the deck slab. If floorbeams are present, they are considered part of the superstructure.

Sometimes a concrete floorbeam and/or tee beam floor system is used between the deck girders (see Figure 7.3.3).

Figure 7.3.3 Concrete Deck Girder, Underside View

Concrete through girders are used for simple spans ranging from 9 to 18 m (30 to 60 feet) at locations with a limited under-clearance (see Figure 7.3.4). They are, however, not economical for wide roadways and are usually limited to about 7 m (24-foot) width. Girders are usually 450 to 760 mm (18 to 30 inches) wide and 1220 to 1830 mm (4 to 6 feet) deep.

Figure 7.3.4 Concrete Through Girder Elevation View

Care must be taken not to describe concrete girder bridges as composite. They do not meet the definition of composite because the concrete girders and deck consist of the same material, even though they are rigidly connected with rebars.

In a deck girder as well as a through girder structure, the live loads from the roadway surface are carried to the girders through the deck. The girders in turn carry the loads to the substructure.

Primary Members and Secondary Members

The primary members of a girder bridge are the girders, floorbeams (if present) and the deck. The secondary members consist of diaphragms or struts.

Steel Reinforcement

The primary (tension) reinforcing steel consists of main longitudinal reinforcement and shear reinforcement or stirrups. The main tension reinforcement is located in the bottom of the girder (positve moment) and on the top (negative moment). The beam also contains shear reinforcement, called stirrups, and are located throughout the girder length. Stirrups are generally U-shaped bars and run transversely across the bottom of the girder (see Figure 7.3.5). The need for stirrups is greatest near the beam supports where shear stresses are the highest.

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the sides of the girders (see Figure 7.3.5). The primary and secondary reinforcing steel for the deck portion of the beam is the same as for a standard concrete deck.

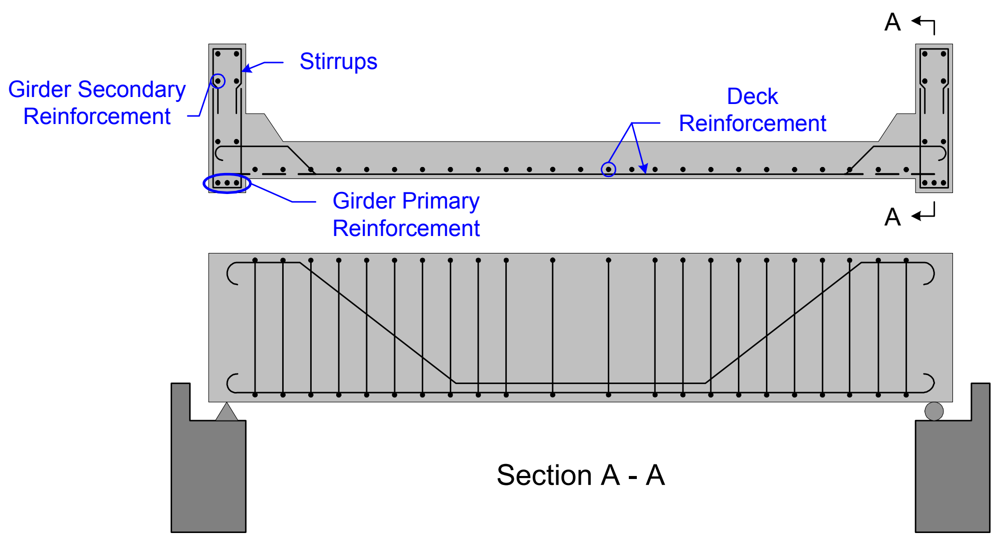


Figure 7.3.5 Steel Reinforcement in a Concrete Through Girder

7.3.3 Overview of Common Defects

Common defects that occur on concrete girder bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear

- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.3.4 Inspection Procedures and Locations

Inspecting a concrete girder bridge is similar to the procedure discussed in Topic 5.2.6, and includes the following specific procedures:

Procedures

Visual

The inspection of concrete girders 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. 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 girders and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete bridge girders are caused by corrosion of the rebar. When the deterioration of a concrete girder progresses to the point of needing rehabilitation, an in-depth inspection of the girder 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

Examine bearing areas for spalling where friction from thermal movement and high bearing pressure could cause the concrete to spall. Check for crushing of the girder near the bearing seat. Check the condition and operation of any bearing devices (see Figure 7.3.6).

Figure 7.3.6 Bearing Area of a Through Girder Bridge

Shear Zones

Investigate the area near the supports for the presence of shear cracking. The presence of transverse cracks on the underside of the girders or diagonal cracks on the sides of the girders indicate the onset of shear failure. These cracks indicate extreme shear stresses and should be carefully measured.

Tension Zones

Tension zones should be examined for flexure cracks, which would be vertical on the sides and transverse across the bottom of the deck. The tension zones are at the midspan of the through girders along the bottom of the girder and possibly the deck for both simple and continuous span bridges (see Figure 7.3.7). Additional tension zones are located on the girders over the piers for continuous spans. Cracks greater that 2 mm (1/16 inch) wide are considered wide cracks and indicate extreme bending stresses.

Check for deteriorated concrete near the tension zones, which could result in the debonding of the tension reinforcement. This would include delamination, spalls, and contaminated concrete.

Check for efflorescence from cracks and discoloration of the concrete caused by rust stains from the reinforcing steel. In severe cases, the reinforcing steel and any lap splices may become exposed due to spalling (see Figure 7.3.8). Document the remaining crosssection of reinforcing steel since section loss will decrease live load capacity.

Figure 7.3.7 Typical Elevation View of a Through Girder Bridge

Figure 7.3.8 Exposed Reinforcement in a Through Girder (under hammer)

Areas Exposed to Drainage

Inspect areas exposed to drainage. These areas will usually be at any joints or around the scuppers. Look for contamination due to deicing agents on the interior face of through girders (see Figure 7.3.9). Check around drain holes for deterioration of girder concrete.

Figure 7.3.9 Close-up of a Girder with Heavy Scaling due to Deicing Agents

Areas Exposed to Traffic

Check areas damaged by collision. Document the number of exposed and severed reinforcing bars as well as the amount of concrete and steel section loss. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss of the reinforcement bars.

Areas Previously Repaired

Examine areas that have been previously repaired. Determine if the repairs are in place and if they are functioning properly.

7.3.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the Element Level 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 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. Element Level Smart Flags are also used to describe the condition of the concrete superstructure.

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

Element No.	Description
110	Concrete Open Girder/beam

The unit quantity for the girder/beam is meters or feet, and the total length of all girders must be placed in one of the four available condition states. 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 traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned.

This page intentionally left blank

Table of Contents

Section 7
Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Topic 7.4 Concrete Channel Beams

7.4.1 Introduction

In appearance, the channel beam bridge resembles the tee beam bridge because the stems of the adjacent channel beams extend down to form a single stem (see Figure 7.4.1). In addition to the appearance of the finished concrete, the channel beam is different than the tee beam by the presence of a full-length seam or joint along the bottom of the stem (see Figure 7.4.2).

Figure 7.4.1 Underside View of Precast Channel Beam Bridge

Figure 7.4.2 General Underside View of Channel Beam Bridge

7.4.2 Design Characteristics

General

Channel beams are generally precast and consist of a mildly reinforced slab cast monolithically with two stems about 900 to 1200 mm (3 to 4 feet) apart (see Figure 7.4.3). Channel beams can also be cast-in-place with a curved underbeam soffit constructed over U-shaped beam forms (see Figure 7.4.4).

Precast channel beams may be conventionally reinforced or may be prestressed.

Figure 7.4.3 General View of a Precast Channel Beam Bridge

Figure 7.4.4 Underside View of a Cast-in-Place Channel Beam Bridge

Primary and Secondary Members

The primary members of channel beam bridges are the channel beams. Channel beams are usually found on spans up to 15 m (50 ft). As already mentioned, channel beams are usually precast, but they are sometimes cast-in-place on removable pan forms. The secondary members of channel beam bridges are the diaphragms.

Steel Reinforcement

Reinforcement cover for older channel beam bridges is often less than today's cover requirements. Air entrained concrete was not specified in channel beams fabricated in the 1940's and early 1950's, and concrete was often poorly consolidated.

The primary reinforcing steel consists of stem tension reinforcement and shear reinforcement or stirrups. The tension reinforcement is located in the bottom of the channel stem and oriented longitudinally. The tension steel reinforcement in current channel beams consists of either mild reinforcing bars or prestressing strands. The sides of the stems are reinforced with stirrups. The stirrups are located vertically in the sides of the channel stems at various spacings throughout the length and closer near the beam supports. The need for stirrups is greatest near the beam supports where the shear stresses are the highest.

The primary reinforcing steel for the slab portion of the beam is located in the bottom of the slab and is placed transversely, or perpendicular to the channel stems (see Figure 7.4.5).

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the sides of deep channel stems and longitudinally in the slab. The primary and secondary reinforcing steel for the slab portion of the beam is the same as for a standard concrete slab (see Figure 7.4.5).

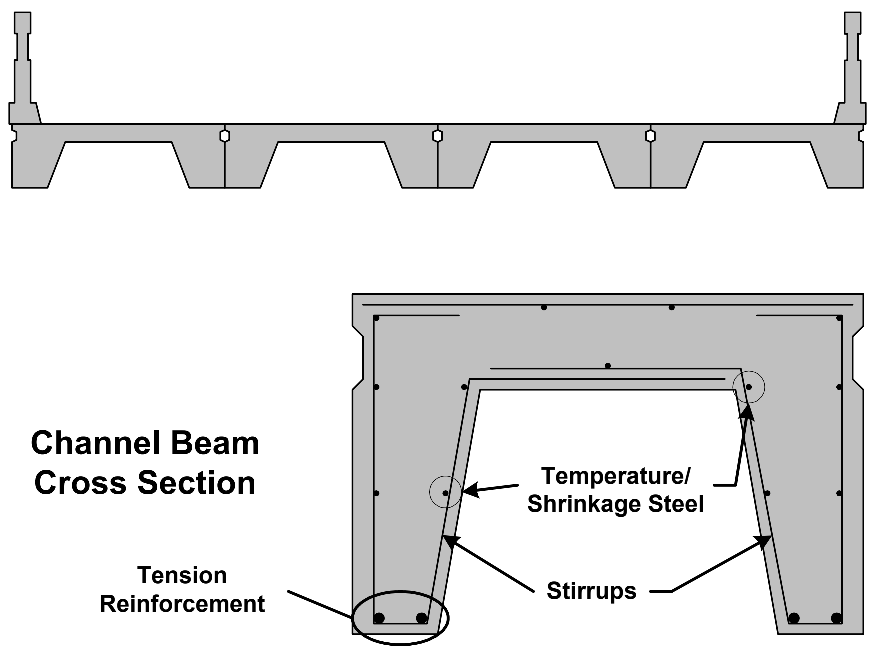


Figure 7.4.5 Cross section of a Typical Channel Beam

7.4.3 Overview of Common Defects

Common defects that occur on concrete channel beam bridges include:

- Cracking
- Scaling
- Delamination
- Spalling

- Efflorescence
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.4.4 Inspection Procedures and Locations

Inspecting a channel beam bridge is similar to the procedure discussed in Topic 5.2.6, and includes the following specific procedures:

Procedures

Visual

The inspection of concrete slabs and stems for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination with a hammer can be a tedious operation. In most cases, a chain drag is used. A chain drag is made of several sections of chain attached to a pipe that has a handle attached to it. The inspector drags this across a slab and makes note of the resonating sounds. A chain drag can usually cover about a 900 mm (3-feet) wide section of slab at a time (see Figure 5.2.8).

If the inspector deems it necessary, core samples can be taken from the slab and stems and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete bridges are caused by corrosion of the rebar. When the deterioration of a concrete channel beam progresses to the point of needing rehabilitation, an in-depth inspection of the channel beam 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

Examine bearing areas for spalling where friction from thermal movement and high bearing pressure could cause the concrete to spall. Check for crushing of the stem near the bearing seat. Check the condition and operation of any bearing devices.

Shear Zones

Investigate the area near the supports for the presence of shear cracking. The presence of transverse cracks on the underside of the stem or diagonal cracks on the sides of the stems indicate the onset of shear failure. These cracks indicate extreme shear stresses and should be carefully measured.

High Moment Regions

Tension zones should be examined for flexure cracks, which would be vertical on the sides and transverse across the bottom of the stem. The tension zones are at the midspan along the bottom of the stem for both simple and continuous span bridges. Additional tension zones are located on the slab over the piers for continuous spans. Flexure cracks in the slab will be found on the underside in a longitudinal direction.

Check for deteriorated concrete near the tension zones, which could result in the debonding of the tension reinforcement. This would include delamination, spalls, and contaminated concrete. These could occur on both the concrete stems and the slab.

Check for efflorescence from cracks and discoloration of the concrete caused by rust stains from the reinforcing steel. In severe cases, the reinforcing steel may become exposed due to spalling. Document the remaining cross section of reinforcing steel since section loss will decrease live load capacity. Check for evidence of sagging or camber loss (see Figure 7.4.6).

Figure 7.4.6 Excessive Deflection at Midspan

Areas Exposed to Drainage or Traffic

Inspect the seam or joint between two adjacent beams for leakage. Leakage generally indicates a broken shear key between the channel beams (see Figure 7.4.7). If signs of leakage are present between beams, the superstructure should be observed closely for differential beam deflection under live load (see Figure 7.4.8). Also, check beam ends for concrete deterioration due to leaking joints.

Examine areas exposed to drainage. Look for spalls and contamination at the ends and edges of the channel beams, scuppers, drain holes, and the curb line.

Check the tie-bolts for tightness and corrosion (see Figures 7.4.9 and 7.4.10).

Check the diaphragms for cracks which may occur from twisting or excessive deflection of the beams (see Figure 7.4.11).

Figure 7.4.7 Joint Leakage Between Channel Beams

Figure 7.4.8 Top of Slab View of Precast Channel Beam Bridge

Figure 7.4.9 Stem Tie-bolts

Figure 7.4.10 Close-up of Stem Tie-bolt

Figure 7.4.11 Close-up of Diaphragm

Check areas damaged by collision. Document the number of exposed and severed reinforcing bars, as well as the amount of concrete and steel section loss. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss of the reinforcement bars.

Damaged Areas

Examine areas that have been previously repaired. Determine if the repairs are in place, and if they are functioning properly.

7.4.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the element level 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 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 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. Element level Smart Flags are also used to describe the condition of the concrete superstructure.

In an element level condition state assessment of a concrete channel beam bridge, the AASHTO CoRe element is:

Element No.	**Description**
110	Concrete Open Girder/beam
109	Prestressed Concrete Open Girder/beam

The unit quantity for the girder/beam is meters or feet, and the total length of all girders must be placed in one of the four available condition states. 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 traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned.

This page intentionally left blank

Table of Contents

Section 7

Inspection and Evaluation of Common Concrete Superstructures

Topic 7.5 Concrete Arches and Arch Culverts

7.5.1 Introduction

A true arch has an elliptical shape and functions in a state of pure axial compression. It can be thought of as a long curved column. This makes the true arch an ideal form for the use of concrete. Unfortunately, the true arch form is often compromised to adjust for a specific bridge site. Because of this compromise, modern concrete arch bridges resist a load combination of axial compression, bending moment, and shear.

7.5.2 Design Characteristics

The basic design concept in arch construction utilizes a "building block" approach. Arch elements, although connected, are stacked or "bearing" on top of one another. The elements at the bottom of the pile receive the largest compressive loads due to the weight of the elements above. Arch spans are always considered "simple span" designs because of the basic arch function.

General

Open Spandrel Arch

The open spandrel concrete arch is considered a deck arch since the roadway is above the arches. The area between the arches and the roadway is called the spandrel.

Open spandrel concrete arches receive traffic loads through spandrel bents that support a slab or tee beam floor system (see Figure 7.5.1). This type of arch is generally for 61 m (200 feet) and longer spans.

Figure 7.5.1 Open Spandrel Arch Bridge

Closed Spandrel Arch

Closed spandrel arches are deck arches. The spandrel area (i.e., the area between the arch and the roadway) is occupied by fill retained by vertical walls. The arch member is called a ring or barrel and is continuous between spandrel walls.

Closed spandrel arches receive traffic loads through the fill material which is contained by spandrel walls (see Figure 7.5.2). This type of arch is efficient in short span applications.

Figure 7.5.2 Multi-span Closed Spandrel Arch Bridge

A closed spandrel arch with no fill material has a hollow vault between the spandrel walls. This type of arch has a floor system similar to the open spandrel arch and should be inspected accordingly.

Through Arch

A concrete through arch is constructed having the crown of the arch above the deck and the arch foundations below the deck. Hangers or cables suspend the deck from the arch. Concrete through arches are very rare (see Figure 7.5.3). These types of arches are sometimes referred to as “Rainbow Arches”.

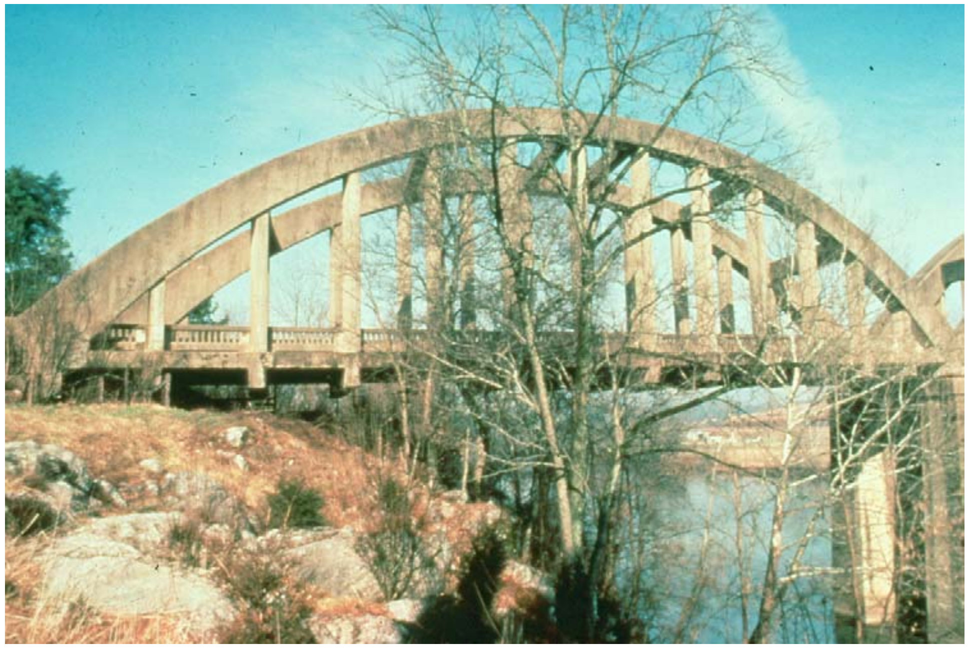


Figure 7.5.3 Concrete Through Arch Bridge

Precast Arch

Precast concrete arches are gaining popularity and can be integral or segmental. The integral arches typically have an elliptical barrel with vertical integral sides (see Figure 7.5.4). Segmental arches are oval or elliptical and can have several hinges along the arch (see Figure 7.5.5). The hinges allow for rotation and eliminate the moment at the hinge location. Both integral and segmental precast arch sections are bolted or post-tensioned together perpendicular to the arch.

Figure 7.5.4 Precast Concrete Arch with Integral Vertical Legs

Figure 7.5.5 Precast Segmental Concrete Arch

Large segmental precast arches that are post-tensioned have the ability to span great distances. This type of arch is constructed from the arch foundations to the crown using segmental hollow sections. The segmental sections are post-tensioned together along the arch through post-tensioning ducts placed around the perimeter of the segmental section. This type of design can be strong enough so that spandrel columns are not needed to support the deck (see Figure 7.5.6). For this type of design, the deck and supporting members bear on the top or crown of the arch.

High quality control can be obtained for precast arches. Sections are precast in a casting yard which allows manufacturers to properly monitor the concrete placement and curing. Reinforcement clearances and placement is also better controlled in a casting yard. Precast sections are typically tested prior to gaining acceptance for use. This ensures that the product can withstand the required loads that are applied.

Figure 7.5.6 Precast Post-tensioned Concrete Arch without Spandrel Columns

Arch Culvert

Although concrete arch culverts look like and experience most of the same defects as concrete arch bridges, concrete arch culverts are separate from concrete arch bridges due to hydraulic, structural, maintenance, traffic safety, construction, durability, and inspection differences. . An arch culvert is a curved shaped culvert that works primarily in compression. A variation of the arch culvert is the tied arch culvert. It is basically the same as the arch culvert, but it has an integral floor serving as a tie between the ends of the arch. Concrete arch culverts can be cast-in-place or precast. Unlike arch bridges, arch culverts are designed to flow full peak flows. Also, the embankment material surrounding an arch culvert is more important than the embankment material around an arch bridge. These differences plus others require special attention to be given to culverts (see Figure 7.5.7).

Figure 7.5.7 Concrete Arch Culvert

Primary Members and Secondary Members

Open Spandrel Arch

The reinforced concrete open spandrel arch consists of one or more arch ribs. The arch members are the primary load-carrying elements of the superstructure. The arch and the following members supported by the arch are also considered superstructure elements:

- Spandrel bents - support floor system
- Spandrel bent cap - transverse beam member of the spandrel bent
- Spandrel columns - vertical members of the spandrel bent which support the spandrel bent cap
- Spandrel beams - fascia beams of the floor system
- Floor system - a slab or tee beam arrangement supported by the spandrel bent caps and the substructure elements

The secondary members of an open spandrel arch bridge are the arch struts, which are transverse beam elements connecting the arch ribs. Arch struts provide stability against lateral forces (see Figure 7.5.8).

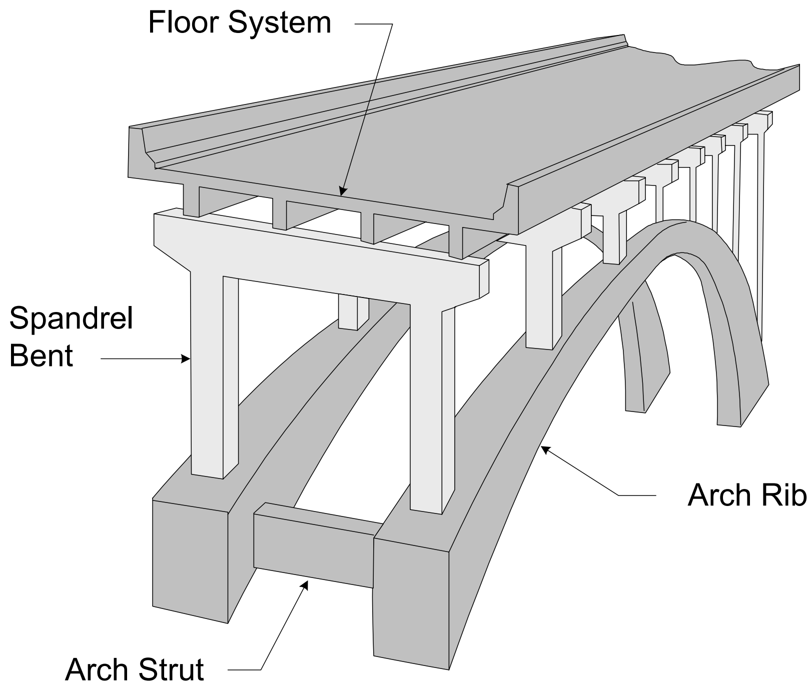


Figure 7.5.8 Primary and Secondary Members of an Open Spandrel Arch

Closed Spandrel Arch

For a closed spandrel arch, the primary members are the arch rings and spandrel walls. The arch rings support fill material, roadway, and traffic, while the spandrel walls retain fill material and support the bridge parapets.

The arch and members supported by the arch are superstructure elements. The arch itself is the primary load-carrying element of the superstructure (see Figure 7.5.9).

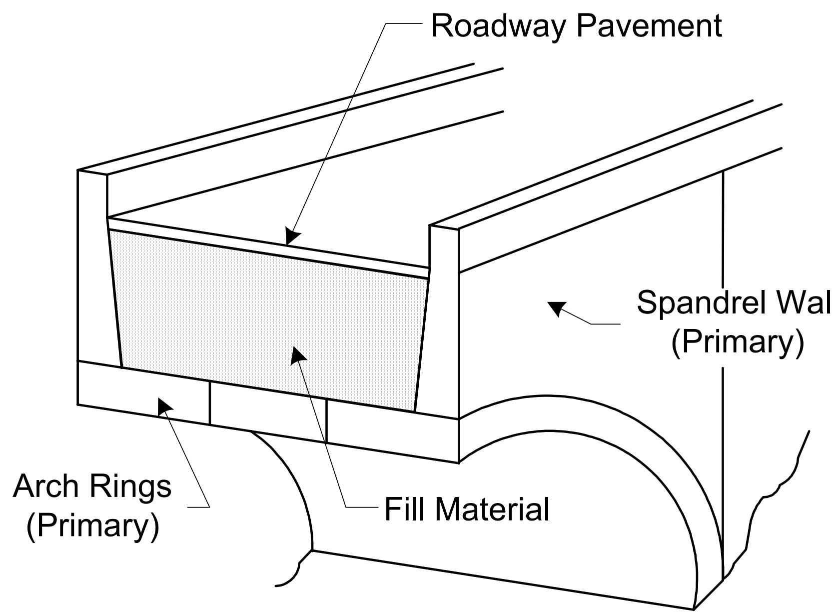


Figure 7.5.9 Primary Members of a Closed Spandrel Arch

Concrete Arch Culverts

The primary member in a concrete arch culvert is the culvert barrel. The barrel supports fill material and any live loads crossing the structure.

Steel Reinforcement

For the proper inspection and evaluation of concrete arch bridges and culverts, the inspector must be familiar with the location and purpose of steel reinforcement.

Open Spandrel Arch

The primary reinforcing steel in an open spandrel arch follows the shape of the arch from support to support. Since the arch is a compression member, reinforcement is similar to column reinforcement. The surfaces of the arch rib are reinforced with equal amounts of longitudinal steel held in place with lateral ties. This longitudinal or column reinforcement can act as compression reinforcement when the arch must resist moment due to axial load eccentricity or lateral loads. Spandrel columns are also compression members and are reinforced similar to the arch rib (see Figure 7.5.10).

In spandrel bent caps, the primary reinforcement is tension and shear steel. This is provided using "Z" shaped bars since the cap behaves like a fixed end beam (see Figure 7.5.11).

The floor system is designed and reinforced similar to other concrete beams (e.g. tee beams).

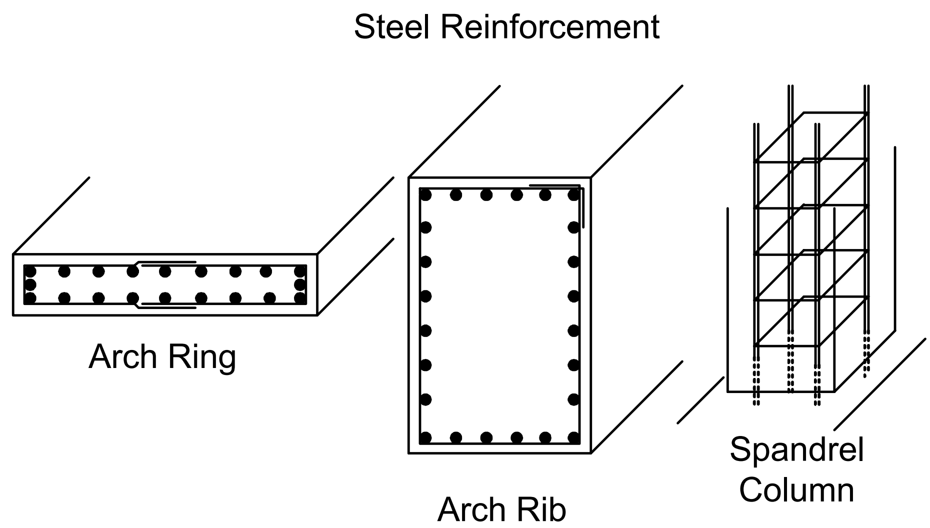


Figure 7.5.10 Open Spandrel Arch Reinforcement

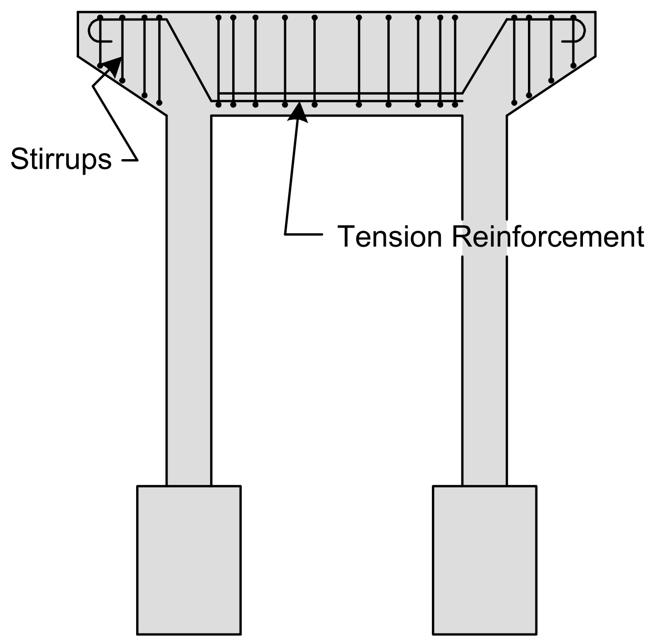


Figure 7.5.11 Spandrel Bent Cap Reinforcement

Closed Spandrel Arch

The primary reinforcing steel in the arch ring follows the shape of the arch from support to support and consists of a mat of reinforcing steel on both the top and bottom surfaces of the arch. The inspector will be unable to inspect the top surface of the arch due to the backfill.

The spandrel walls are designed to retain the backfill material. The primary tension steel for the wall is usually at the back, or unexposed, face of the wall, hidden from view. The front, or outside, face of the wall is reinforced in both directions with temperature and shrinkage steel (see Figure 7.5.12).

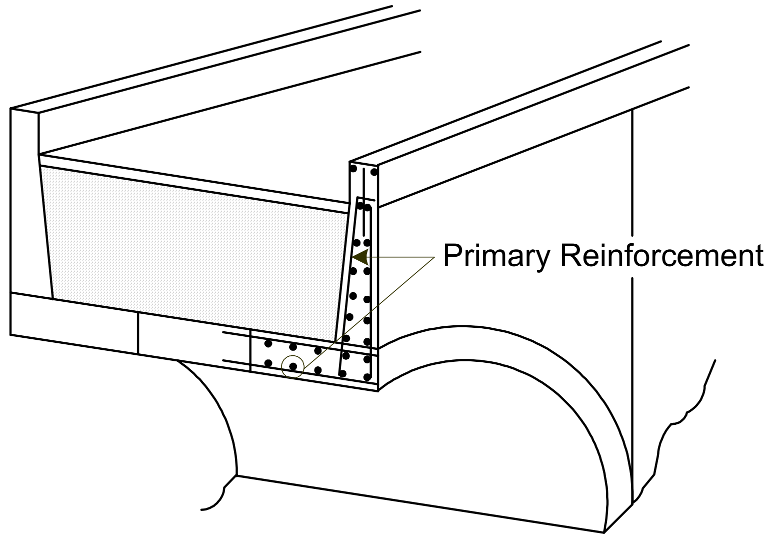


Figure 7.5.12 Reinforcement in a Closed Spandrel Arch

Arch Culvert

Reinforcement for arch culverts follows the shape of the arch from support to support. A mat of reinforcing steel is used on the top and bottom surfaces of the arch.

Other Reinforcement

Temperature and shrinkage reinforcement is used in the floor system for open spandrel arches.

A grid of temperature and shrinkage reinforcement is used in spandrel walls for closed spandrel arches.

7.5.3 Overview of Common Defects

Common defects that occur on concrete arches and culverts include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Collision damage
- Abrasion
- Overload damage
- Reinforcing/prestressing 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.

7.5.4 Inspection Procedures and Locations

Procedures

Visual

The inspection of concrete arches and culverts 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 concrete and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete bridges are caused by corrosion of the rebar. 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength

- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Open and Closed Spandrel Arches

Bearing Areas

- The arch/skewback interface has the greatest bearing load magnitude. Inspect for loss of cross section of the reinforcement bars at the spalls. Examine the arch for longitudinal cracks. These indicate an overstress condition.

- The arch/spandrel column interface has the second greatest bearing load magnitude. Examine for reinforcement cross-section loss at the spalls. Check for horizontal cracks in the columns within several meters from the arch. These indicate excessive bending in the column, which is caused by overloads and differential arch rib deflection.

- The spandrel column/cap interface has the third greatest bearing load magnitude. Inspect for loss of section due to spalling. Examine the column for diagonal cracks which begin at the inside corner and propagate upward. These indicate differential arch rib deflections (see Figure 7.5.13).

- The floor system/bent cap interface has the smallest bearing load magnitude. Examine bearing areas as described in the slab, tee beam and girder sections.

- Examine the arch ring for unsound concrete. Look for rust stains, cracks, discoloration, crushing, and deterioration of the concrete. The interface between the spandrel wall and the arch should be carefully inspected for spalls that could reduce the bearing area. Investigate the arch for transverse cracks, which indicate an overstress condition.

Figure 7.5.13 Spandrel Column Cap Interface

Shear Zones

➢ Check for shear cracks at the ends of the spandrel bent caps. When arch ribs are connected with struts, examine the arches near the connection for diagonal cracks due to torsional shear. These cracks indicate excessive differential deflection in the arch ribs. Also investigate the floor system for shear cracks.

Tension Zones

➢ Inspect the tension areas of the spandrel bent caps and columns (i.e., mid-span at the bottom and ends at the top) (see Figure 7.5.14). Also check the tension areas in the floor system.

➢ Check for transverse cracks in the arch which indicate an overstress condition. Transverse cracks are oriented perpendicular to the arch member.

➢ Inspect the spandrel walls for sound concrete. Look for cracks, movement, and general deterioration of the concrete (see Figure 7.5.15).

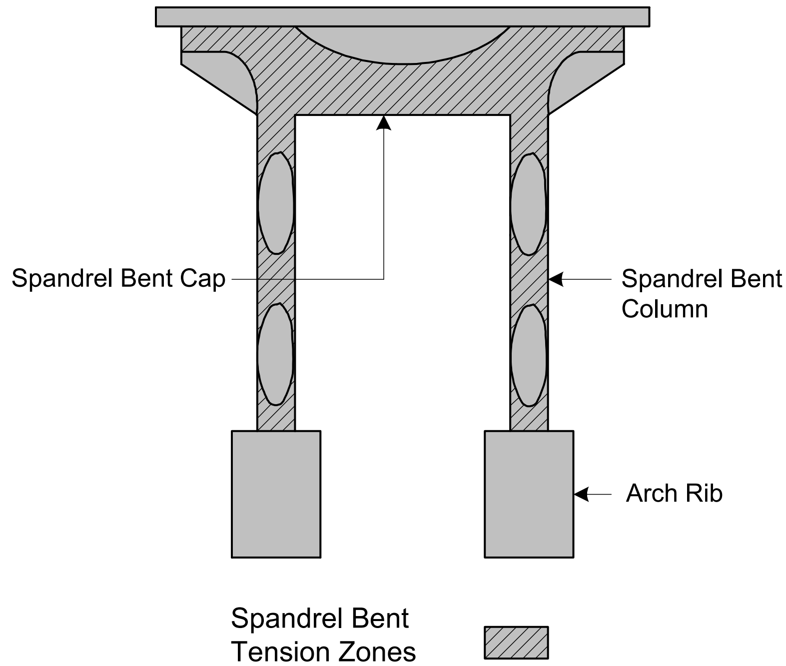


Figure 7.5.14 Spandrel Bent Tension Zone

Figure 7.5.15 Deteriorated Arch/Spandrel Wall Interface

Compression Zones

➢ Investigate the compression areas throughout the arches and spandrel columns (not only at the bearing areas). Transverse or lateral cracks indicate excessive surface stresses caused by buckling forces and bending moment (see Figure 7.5.16).

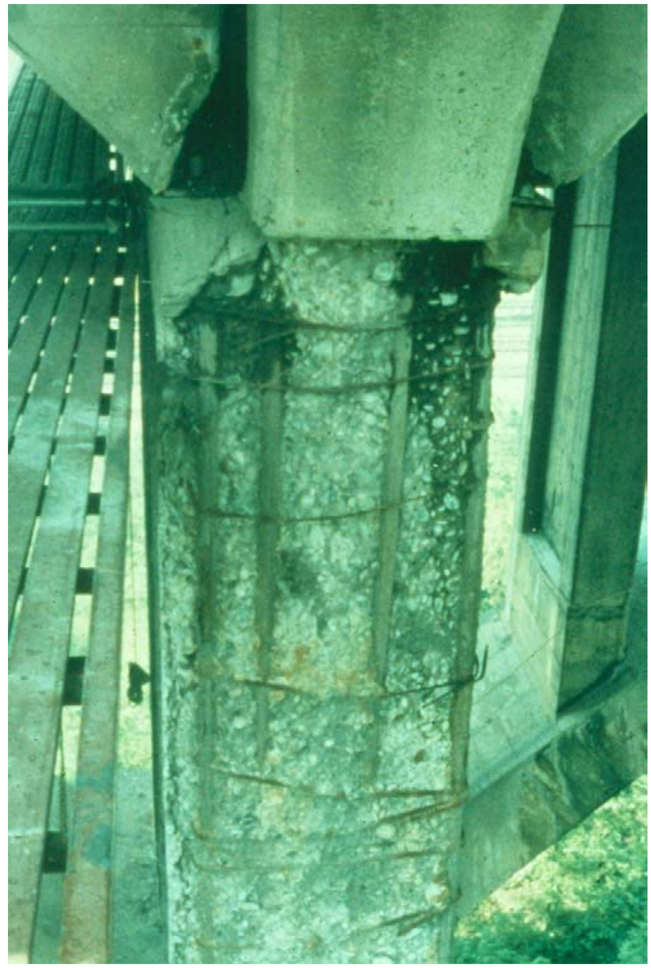


Figure 7.5.16 Severe Scaling and Spalling on a Spandrel Column

Areas Exposed to Drainage

- For an open spandrel arch, check the areas exposed to drainage and roadway runoff. Elements beneath the floor system are prone to scaling, spalling, and chloride contamination (see Figure 7.5.17).

- For a closed spandrel arch, make sure that weep holes are working properly.

- For a closed spandrel arch, check that surface water drains properly and does not penetrate the fill material.

Figure 7.5.17 Scaling and Contamination on an Arch Rib Due to a Failed Drainage System

Areas Exposed to Traffic

- Check areas damaged by collision. Document the number of exposed and severed reinforcing bars as well as the amount of concrete and steel section loss. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss of the reinforcing bars.

Previous Repairs

- Examine thoroughly any repairs that have been previously made. Determine if repaired areas are functioning properly. Effective repairs and patching are usually limited to protection of exposed reinforcement.

Concrete Arch Culverts

For a concrete arch culvert, the following locations should be inspected:

- Inspect the culvert barrel for rust stains, cracks, discoloration, crushing, and other deterioration of the concrete.

- Inspect the culvert barrel for spalls, delaminations, and rebar section loss.

- Check weep holes for partial or full blockage.

- Check approach conditions for dips, sags, cracks, pavement patches or other settlement indicators.

- Examine headwalls and wingwalls for undermining and settlement. Cracking, tipping, or separation of the barrel from the headwall are indications of undermining and settlement. Erosion is also a concern with headwalls and wingwalls.

- When dealing with precast concrete culverts, inspect for joint defects, leaking joints, cracked joints, and separated joints.

- Refer to Topics 11.1 through 11.3 for waterway inspection procedures and locations.

7.5.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the Element Level 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 superstructure. Rating codes range from 9 to 0, where 9 is the best rating possible. The previous inspection data should be used along with current inspection findings to determine the correct rating (see Topic 4.2).

For concrete arch culverts, the NBIS rating guidelines yield a 1-digit code on the Federal (SI&A) sheet that indicates the overall condition of the culvert. The culvert item not only evaluates the structural condition of the culvert, but also encompasses the alignment, settlement in the approach roadway and embankment, joints, scour, and headwalls and wingwalls. Integral wingwalls are included in the evaluation up to the first construction or expansion joint. Like concrete arches, the 1-digit code that best describes the culvert's overall condition is chosen, and the rating codes range from 9 to 0, where 9 is the highest possible 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 concrete arch or concrete arch 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. Element Level Smart Flags are also used to describe the condition of the concrete superstructure.

In an element level condition state assessment of a concrete arch or arch culvert, the AASHTO CoRe element is one or more of the following:

Element No.	**Description**
	Open Spandrel Arch
109	Open Girder/Beam (P/S Concrete)
110	Open Girder/Beam (Reinforced Concrete)
154	Floorbeam (P/S Concrete)
155	Floorbeam (Reinforced Concrete)
115	Stringer (stringer floorbeam system) (P/S Concrete)
116	Stringer (stringer floorbeam system) (Reinforced Concrete)
143	Arch (P/S Concrete)
144	Arch (Reinforced Concrete)
204	Column or Pile Extension (P/S Concrete)
205	Column or Pile Extension (Reinforced Concrete)
233	Cap (P/S Concrete)
234	Cap (Reinforced Concrete)
	Closed Spandrel Arch
143	Arch (P/S Concrete)
144	Arch (Reinforced Concrete)
	Arch Culvert
241	Arch Culvert (Precast, Prestressed, or Reinforced Concrete)

The quantities, when dealing with concrete arches or culverts, are all in meters or feet except for Elements 204 and 205, which are given in units of each. The above elements for concrete arches and culverts consist of three to five condition state descriptions to choose from for each element. All elements must be placed in one of the available condition states assigned for each element. Condition state 1 is the best possible rating. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

Smart Flag element numbers available for use in the case of open and closed spandrel arches are 360 – settlement, 361 – scour, and 362 – traffic impact. Concrete arch culverts can use Smart Flag element numbers 361 or 362 if needed. One of three condition state descriptions is chosen for each Smart Flag element that is used.

Table of Contents

Section 7

Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Topic 7.6 Concrete Rigid Frames

7.6.1 Introduction

A concrete rigid frame structure is a bridge type in which the superstructure and substructure components are constructed as a single unit. Rigid frame action is characterized by the ability to transfer moments at the knee, the intersection between the frame legs and the frame beams or slab. Reinforced concrete rigid frame bridges and culverts are cast-in-place monolithic units.

7.6.2 Design Characteristics

General

The rigid frame bridge can either be single span or multi-span. Single span frame bridges span up to 15 m (50 feet) and are generally a slab beam design (see Figure 7.6.1). The basic single span frame shape is most easily described as an inverted "U".

Figure 7.6.1 Three Span Concrete Rigid Frame Bridge

Multi-span frame bridges are used for spans over 15 m (50 feet) with slab or rectangular beam designs (see Figure 7.6.2). Other common multi-span frame shapes include the basic rectangle, the slant leg or K-frame, and Delta frames (see Figure 7.6.3).

Figure 7.6.2 Typical Multi-span Rectangular Concrete Rigid Frame Bridge

Figure 7.6.3 Typical Concrete K-frame Bridge

Rigid frame structures are utilized both at grade and under fill, such as in concrete frame culverts (see Figure 7.6.4).

Figure 7.6.4 Typical Concrete Frame Culvert

Primary Members

For single span frames, the primary member is considered to be the slab portion above the "legs" of the frame (see Figure 7.6.5).

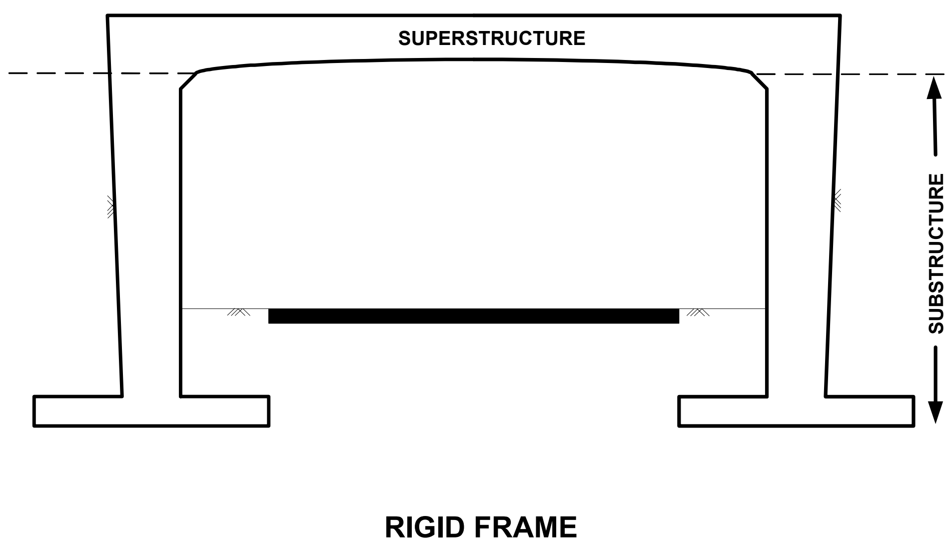


Figure 7.6.5 Elevation of a Single Span Slab Beam Frame

For multi-span frames, the primary members include the frame legs (the slanted beam portions which replace the piers) and the frame beams (the horizontal portion which is supported by the frame legs and abutments) (see Figure 7.6.6).

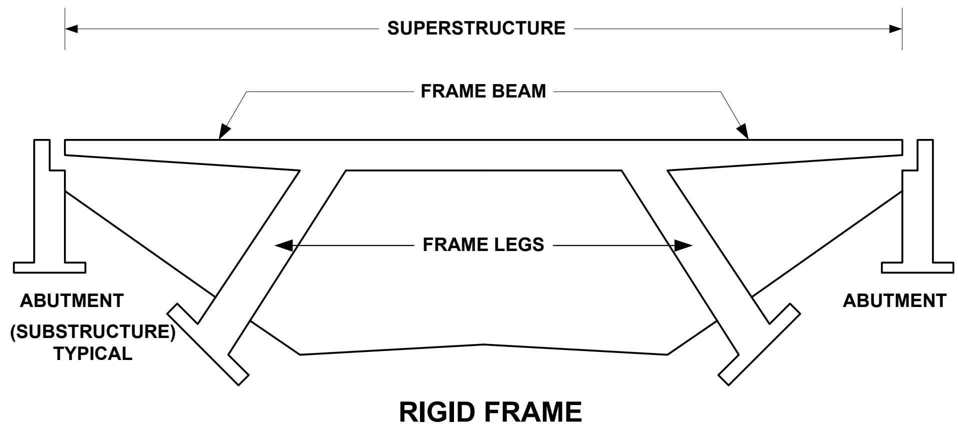


Figure 7.6.6 Elevation of a K-frame

Steel Reinforcement

Rigid frame structures develop positive and negative moment throughout due to the interaction of the frame legs and frame beams (see Figure 7.6.7). In slab beam frames, the primary reinforcement is tension steel.

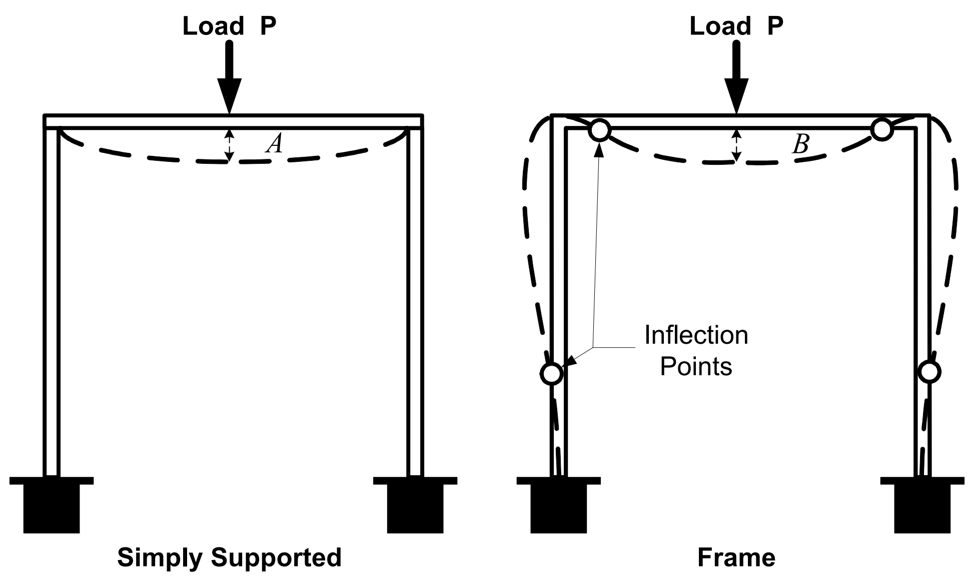
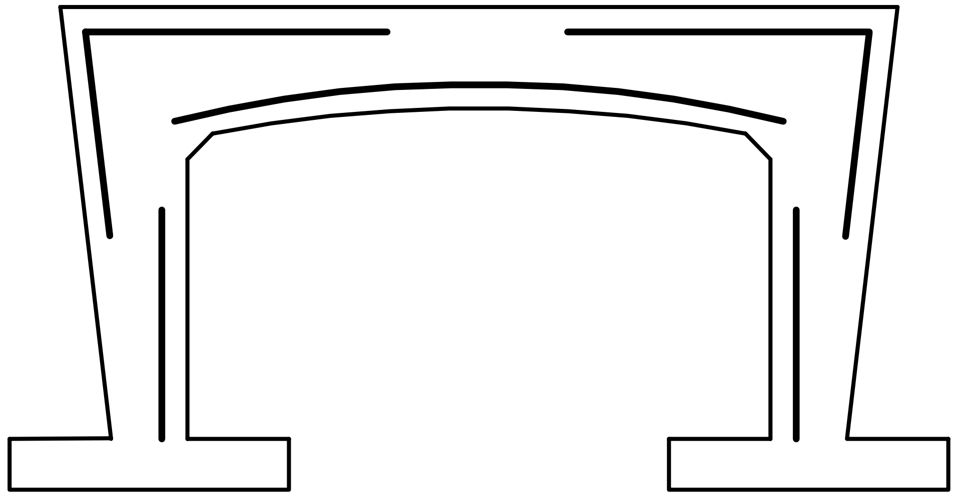


Figure 7.6.7 Deflected Frame Shape

Primary Reinforcement

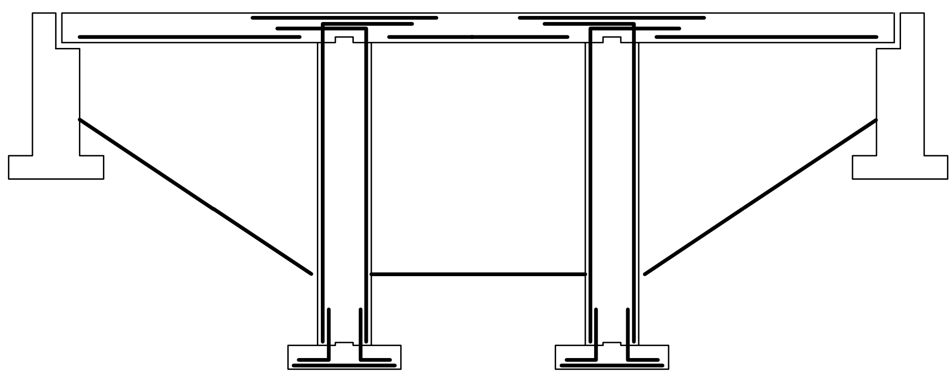
For gravity and traffic loads on single span slab frames, the tension steel is placed longitudinally in the bottom of the frame slab, vertically in the front face of the frame legs, and longitudinally and vertically in the outside corners of the frame (see Figure 7.6.8).



PRIMARY REINFORCEMENT

Figure 7.6.8 Tension Reinforcement in a Single Span Slab Beam Frame

For multi-span slab frames, the tension steel is placed longitudinally in the top and bottom of the frame slab and vertically in both faces of the frame legs (see Figure 7.6.9).



PRIMARY REINFORCEMENT

Figure 7.6.9 Tension Reinforcement in a Multi-span Beam Frame

In the beam portion of rectangular beam frames, the primary reinforcement is tension and shear steel, similar to continuous beam reinforcement. In the frame legs, the primary reinforcement is tension and shear steel near the top and compression steel with column ties for the remaining length (see Figure 7.6.10).

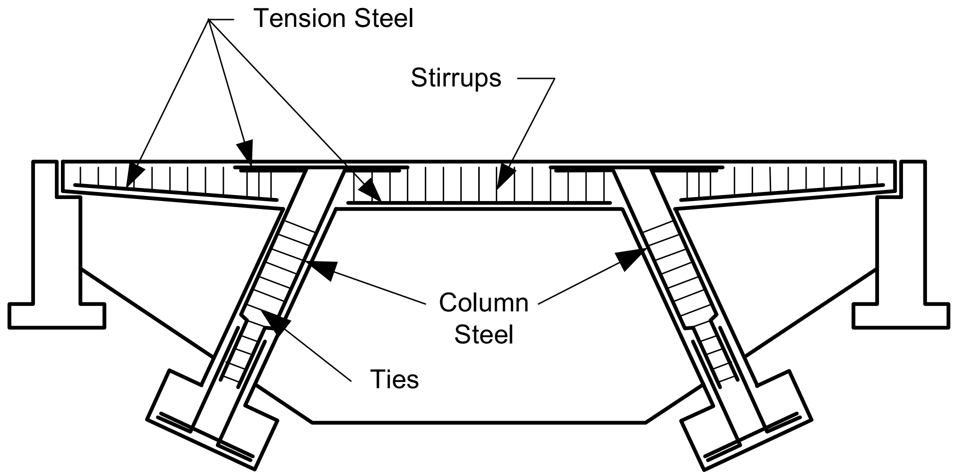


Figure 7.6.10 Tension, Shear, and Column Reinforcement in a Typical K-frame

Secondary Reinforcement

Temperature and shrinkage reinforcement is also included in both sides of the slab frames and in the beam portion of rectangular beam frames. Secondary reinforcement is perpendicular to the tension reinforcement.

7.6.3 Overview of Common Defects

Common defects that occur on concrete rigid frame bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.6.4 Inspection Procedures and Locations

Inspecting a rigid frame bridge is similar to the procedures discussed in Topic 5.2.6 and includes the following specific procedures:

Procedures

Visual

The inspection of concrete rigid frames for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination of a concrete rigid frames with a hammer can be a tedious operation. In most cases, a chain drag is used. A chain drag is made of several sections of chain attached to a pipe that has a handle attached to it. The inspector drags this across a deck and makes note of the resonating sounds. A chain drag can usually cover about a 900 mm (3-feet) wide section of deck at a time (see Figure 5.2.8).

If the inspector deems it necessary, core samples can be taken from the rigid frame and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete rigid frames are caused by corrosion of the rebar. When the deterioration of a concrete rigid frame progresses to the point of needing rehabilitation, an in-depth inspection of the rigid frame 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

Examine the bearing areas for spalling. Check the condition of the bearings, if present.

Shear Zones

Inspect the joint zones where the frame legs meet the frame beams. Look for shear cracks in the frame beams (beginning at the frame legs and propagating toward the adjacent span), in the frame legs (beginning at the top and propagating downward), and in the ends of the frame beams at the end spans (see Figure 7.6.11).

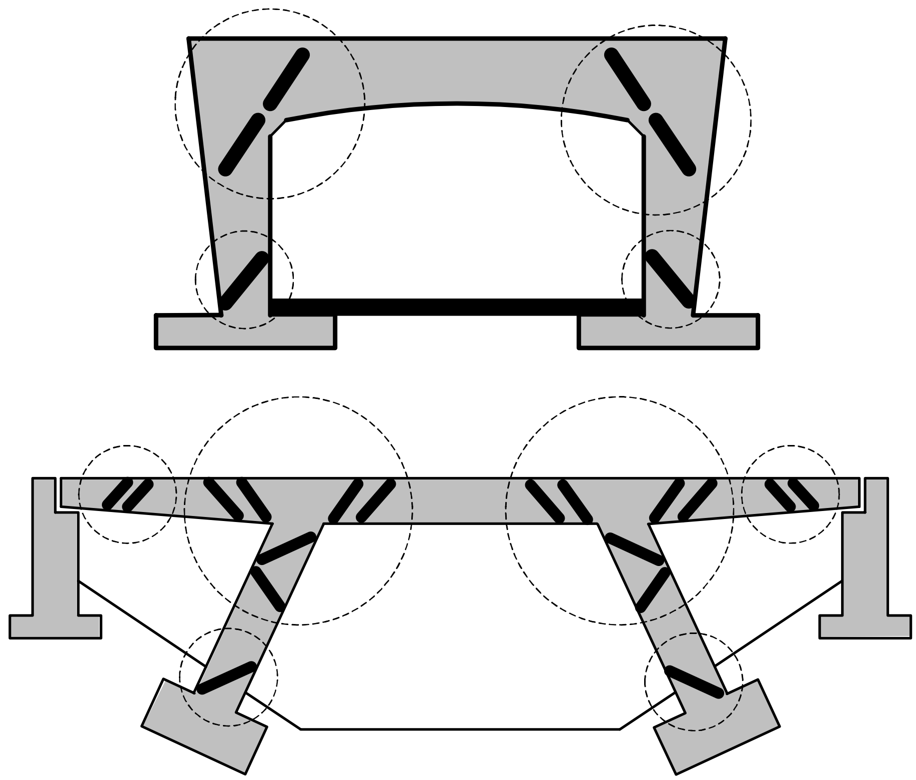


Figure 7.6.11 Shear Zones in Single Span and Multi-span Frames

Tension Zones

Investigate the tension areas for flexure cracks, rust stains, efflorescence, exposed and corroded reinforcement, and deteriorated concrete which would cause debonding of the tension reinforcement. The tension areas are located at the bottom of the frame beam at mid-span, the base of each frame leg (usually buried), and the inside faces of the frame legs at mid-height of single span slab frames (see Figures7.6.12 and 7.6.13).

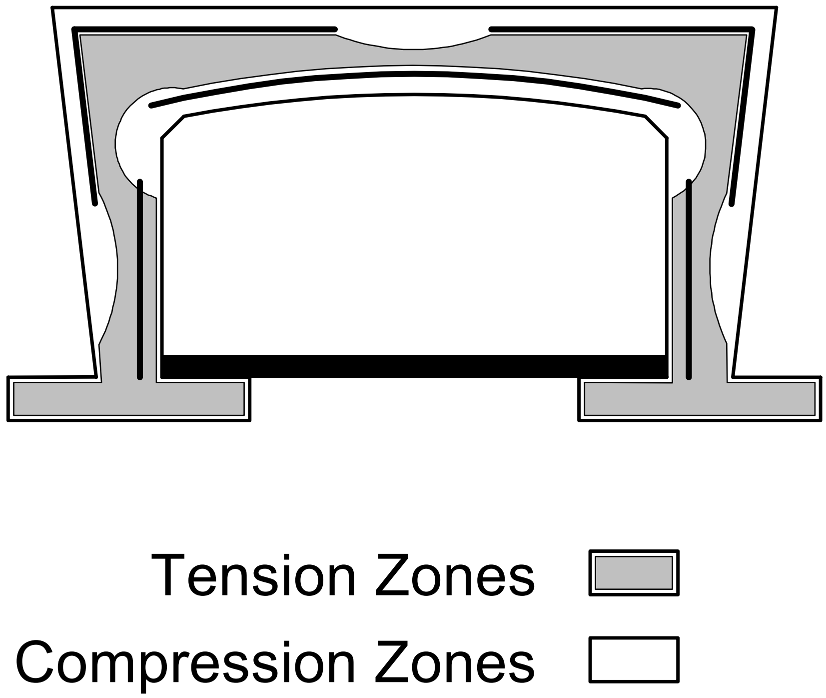


Figure 7.6.12 Tension Zones in a Single Span Beam Frame

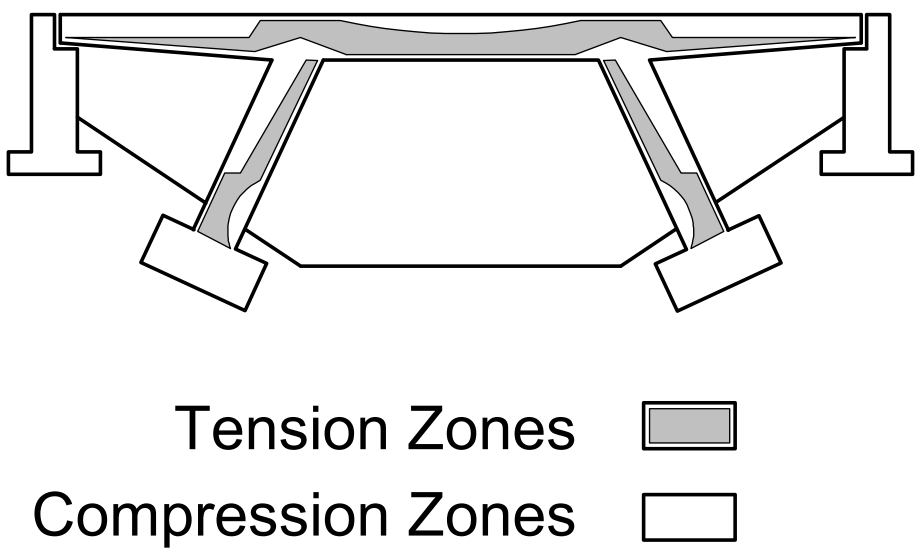


Figure 7.6.13 Tension Zones in a Multi-span Frame

Compression Zones

Investigate the compression areas for spalling, scaling, and exposed reinforcement. The legs of a frame act primarily as columns with a moment applied at the top (see Figure 7.6.14). Check the entire length of the frame legs for horizontal cracks, which indicate buckling.

Figure 7.6.14 K-frame Leg

Areas Exposed to Drainage

Examine the areas exposed to drainage for deteriorated and contaminated concrete. Check the roadway surface of the slab beam frames for delamination and spalls (see Figure 7.6.15). Special attention should be given to the tension zones and water tables.

Figure 7.6.15 Roadway of a Rigid Frame Bridge with Asphalt Wearing Surface

Check longitudinal joint areas of adjacent slab beam frames for leakage and concrete deterioration (see Figure 7.6.16). Check around scuppers and drain holes for deteriorated concrete. Check frame beam ends for deterioration due to leaking deck joints.

Figure 7.6.16 Longitudinal Joint Between Slab Beam Frames

Footings - When an invert slab is not used and the footings are exposed, they should be inspected for undermining and scour. A probing rod or bar should be used to check for voids and scoured areas that may have filled with sediment.

For additional inspection procedures and locations unique to culvert waterways, see Topic 7.12.

7.6.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the element level 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 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 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. Element level Smart Flags are also used to describe the condition of the concrete superstructure.

There is no specific element level condition state assessment of a concrete rigid frame bridges. The following AASHTO CoRe elements may be used to best describe a concrete rigid frame:

Element No.	**Description**
038	Concrete Slab - Bare
052	Concrete Slab – Protected with Coated Bars
053	Concrete Slab – Protected with Cathodic System
110	Concrete Open Girder/beam
205	Column or Pile Extension – Reinforced Concrete
210	Pier Wall – Reinforced Concrete
215	Abutment – Reinforced Concrete
241	Reinforced Concrete Culvert

The unit quantity for slab and columns is each. The unit quantity for the girder/beam and pier wall is meters or feet, and the total length of must be placed in one of the available condition states. 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 traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned.

Table of Contents

Section 7

Inspection and Evaluation of Common Concrete Superstructures

Topic 7.7 Precast and Prestressed Slabs

7.7.1 Introduction

Precast and prestressed slabs have gained popularity since the 1950's (refer to Topic 2.2). This type of design acts as a deck and superstructure combined (see Figure 7.7.1). Individual members are placed side by side and connected together so they act as one. This type of design is effective, due to the slab's shallow depth, when vertical clearances are lacking. Wearing surfaces are generally applied to the top of precast and prestressed slabs and are either concrete or bituminous.

Figure 7.7.1 Typical Prestressed Slab Beam Bridge

Although precast and prestressed slabs are different from concrete decks, the design characteristics, wearing surfaces, protection systems, common defects, inspection procedures and locations, evaluation, and motorist safety concerns are similar to concrete decks. Refer to Topic 5.2 for additional information about concrete decks.

7.7.2 Design Characteristics

Fabrication Method

The precast voided slab bridge is the modern replacement of the cast-in-place slab. This type of bridge superstructure is similar to the cast-in-place slab in appearance only. It is comprised of individual precast slab beams fabricated with circular voids. The voids afford economy of material and reduce dead load (see Figure 7.7.2). Precast slab bridges with very short spans may not contain voids.

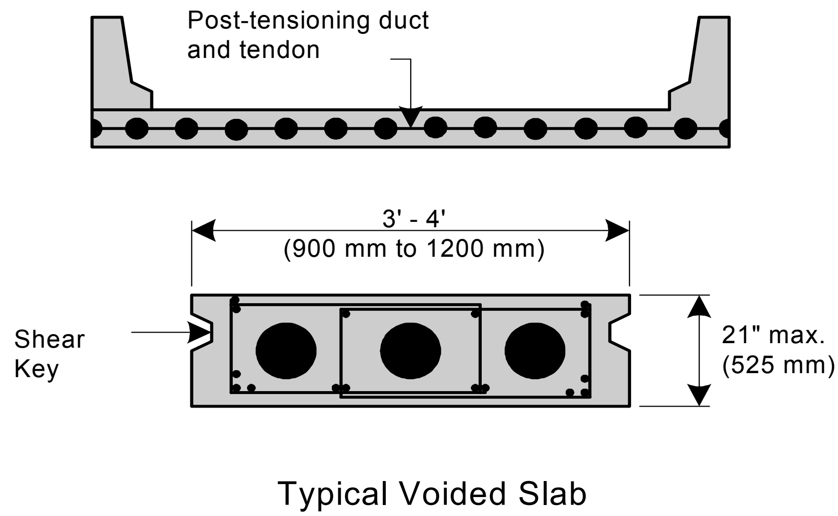


Figure 7.7.2 Cross Section of a Typical Voided Slab

Monolithic Behavior

Precast slab units are practical for spans of 6 to 15 m (20 to 50 feet). The slabs can be single or multiple simple spans. The units are typically 914 to 1219 mm (36 or 48 inches) wide and have a depth of 381, 457, or 533 mm (15, 18, or 21 inches). These special precast units are generally comprised of 28 to 56 MPa (4,000 up to 8,000 psi) prestressed concrete, and reinforced with 1860 MPa (270 ksi) pre- or post-tensioned steel tendons. Adjacent slab units are post-tensioned together with tie rods and grouted at the shear keys. This enables the slab units to act monolithically. Drain holes are placed strategically in the bottom of the slab to allow accumulated moisture to escape.

Identifying Voided Slabs

Physical dimensions alone are not enough to distinguish a slab unit from a box beam. Design or construction plans need to be reviewed. A box beam has one rectangular void, bounded by a top slab, bottom slab, and two webs. A voided slab section has two or three circular voids through it. It is also possible to find precast solid slab units.

Primary Members

The primary members of a precast voided slab bridge are the individual slab units. The slab units make up the superstructure and the deck and are commonly protected by an asphalt or concrete overlay.

Steel Reinforcement

Primary Reinforcement

The primary reinforcement consists of main tension steel and shear reinforcement or stirrups.

Prestressing strands placed near the bottom of the slab make up the main tension steel. Draped strands are often located in the webs. Depending on the age of the structure, the strand size will be 6, 10, 11, or 13 mm (1/4, 3/8, 7/16, or 1/2 inch) diameter. Strands are normally spaced 50 mm (2 inches) on center (see Figure 7.7.3). Shear reinforcement consists of U-shaped stirrups located throughout the slab at various spacings required by design.

Other Reinforcement

Other reinforcement is provided to control temperature and shrinkage cracking. This reinforcement is placed longitudinal in the beam.

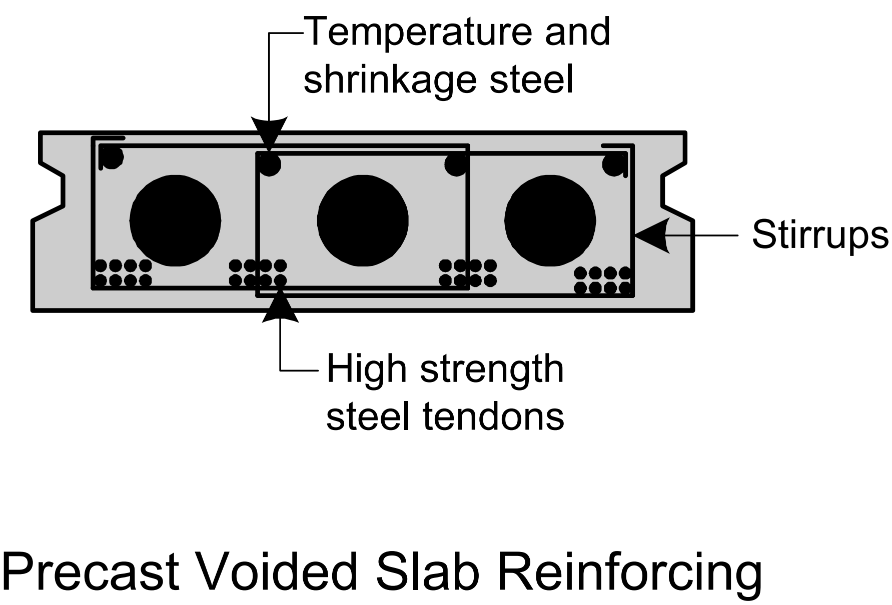


Figure 7.7.3 Prestressed Slab Beam Bridge Reinforcement

7.7.3 Overview of Common Defects

Common defects that occur on precast and prestressed slab bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear
- 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.

7.7.4 Inspection Procedures and Locations

Inspect a precast voided slab bridge similar to as described in Topic 5.2.6 and using the following procedures.

Procedures

Visual

The inspection of concrete slabs for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination of a slab with a hammer can be a tedious operation. In most cases a chain drag is used. A chain drag is made of several sections of chain attached to pipe that has a handle attached to it. The inspector drags this across a deck and makes note of the resonating sounds. A chain drag can usually cover about a 900-mm (3-feet) wide section of deck at a time (see Figure 5.2.8).

If the inspector deems it necessary, core samples can be taken from the slab and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete slabs are caused by corrosion of the rebar. When the deterioration of a concrete slab progresses to the point of needing rehabilitation, an in-depth inspection of the slab 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes

- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

- Examine the bearing areas for spalling concrete. End spalling can eventually lead to the loss of bond in the prestressing tendons. Bearing areas should also be checked for defects or deterioration due to leaking joints or poor quality control.

- Check bearing areas for spalls or vertical cracks. Spalls and cracks may be caused by corrosion of steel due to water leakage or restriction of thermal movement due to a faulty bearing mechanism.

Shear Zones

- Inspect near the supports for diagonal or shear cracks.

- Inspect between the slab sections for leakage and for reflective cracking in the traffic surface (see Figure 7.7.4). These problems indicate failed shear keys and that the slab units are no longer tied together. Observe if there is differential slab beam deflection under live load.

Figure 7.7.4 Leaking Joint between Adjacent Slab Units

Tension Zones

- Check the bottom of the slab sections for flexure cracks due to positive moments. Since prestressed concrete is under high compressive forces, no cracks should be visible. Cracks can be a serious problem since they indicate overloading or loss of prestress. Cracks that may be present will be difficult to detect with the naked eye. To improve detection, a common practice is to wet the slab surface with water using a spray bottle. Capillary action will draw water into a crack, thus producing a visible line when the surrounding surface water evaporates. All cracks should be measured with an optical crack gauge.

- Examine the top of the slab sections (if exposed) near the ends for tensile cracks due to prestress eccentricity. This indicates excessive prestress force. If the top of the slab has a wearing surface applied, check for cracks in the wearing surface. Cracks in the wearing surface may be an indication that the slab is overstressed or that water is getting to the slab.

- Investigate for evidence of sagging, which indicates a loss of prestress. Use a string line or site down the bottom edge of the fascia slab.

- Inspect the slabs for exposed strands. Prestressed strands will corrode rapidly and fail abruptly. Therefore, any exposure is significant (see Figure 7.7.5).

- Check for longitudinal cracking in skewed slab members (see Topic 7.10.4).

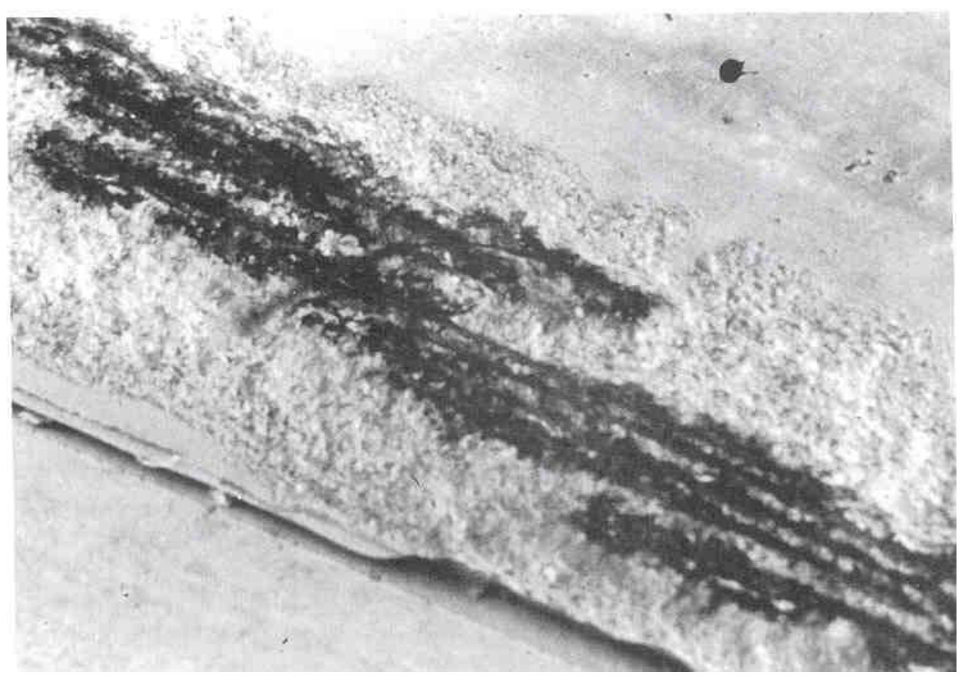


Figure 7.7.5 Exposed Strands in a Precast Slab Beam

Areas Exposed to Drainage

- Investigate areas exposed to drainage for deteriorated and contaminated concrete.

Areas over Traffic

- When precast voided slab superstructures are used for a grade crossing, check the areas over the traveling lanes for collision damage. This is generally not a problem due to the clearance afforded by the relatively shallow units.

Previous Repairs

- Examine thoroughly any repairs that have been previously made. Determine if repaired areas are sound and functioning properly. Effective repairs and patching are usually limited to protection of exposed tendons and reinforcement.

General

- Check the camber of the slab units. Loss of positive camber indicates loss of prestress in the tendons.

- Check the condition of the lateral post-tensioning grout pockets. Cracked grout or rust stains may indicate a failure of the post-tensioning tendon.

7.7.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guidelines systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the element level 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 superstructure. Rating codes range from 9 to 0, where 9 is the best rating possible (see Topic 4.2). For a precast or prestressed slab bridge, these guidelines must be applied for both the deck component and the superstructure component.

The previous inspection data should be used along with current inspection findings to determine the correct rating. Typically, for this type of structure, the deck and superstructure components will have the same 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 top of slab and the underside. 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 concrete superstructure.

In an element level condition state assessment of a precast or prestressed slab bridge, the AASHTO CoRe element is one of the following depending on the riding surface:

Element No.	**Description**
38	Concrete Slab – Bare
39	Concrete Slab – Unprotected with AC Overlay
40	Concrete Slab – Protected with AC Overlay
44	Concrete Slab – Protected with Thin Overlay
48	Concrete Slab – Protected with Rigid Overlay
52	Concrete Slab – Protected with Coated Bars
53	Concrete Slab – Protected with Cathodic System
104	P/S Closed Web/Box Girder

The unit quantity for these elements is "each" for decks, and the entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2) for decks. The unit quantity for girders is "meter" or "linear foot", and the total length of all girders combined must be rated in one of the available condition states for girders. Element 104 is the closest choice in the National Highway Institute (NHI) element list for precast/prestressed slabs. States may decide to choose their own element number for precast/prestressed slabs because the NHI does not have a specific element number for prestressed slabs. Condition state 1 is the best possible element level rating. The inspector must know the total slab surface area in order to calculate a percent deterioration and fit it into a given condition state description. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For structural cracks in the surface of bare slabs, the "Deck Cracking" Smart Flag, Element No. 358, can be used and one of four condition states assigned. Do not use Smart Flag, Element No. 358, if the bridge deck/slab has any overlay because the top surface of the structural deck is not visible. For concrete defects on the underside of a slab element, the "Soffit" Smart Flag, Element No. 359, can be used and one of five condition states assigned.

Table of Contents

Section 7
Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Topic 7.8 Prestressed Double Tees

7.8.1 Introduction

A prestressed double tee beam, like the name implies, resembles two capital letter T's that are side by side (see Figure 7.8.1). The horizontal section is called the deck or flange, and the two vertical leg sections are called the webs or stems. This type of bridge beam is mostly used in short spans or in situations where short, obsolete bridges are to be replaced.

Figure 7.8.1 Typical Prestressed Double Tee Beam

7.8.2 Design Characteristics

General

Prestressed concrete double tee beams are a monolithic deck and stem design that allows the deck to act integrally with the superstructure. The integral design provides a stiffer member, while the material-saving shape reduces the dead load (see Figure 7.8.2).

This type of construction was originally used for buildings and is quite common in parking garages. They have been adapted for use in highway structures.

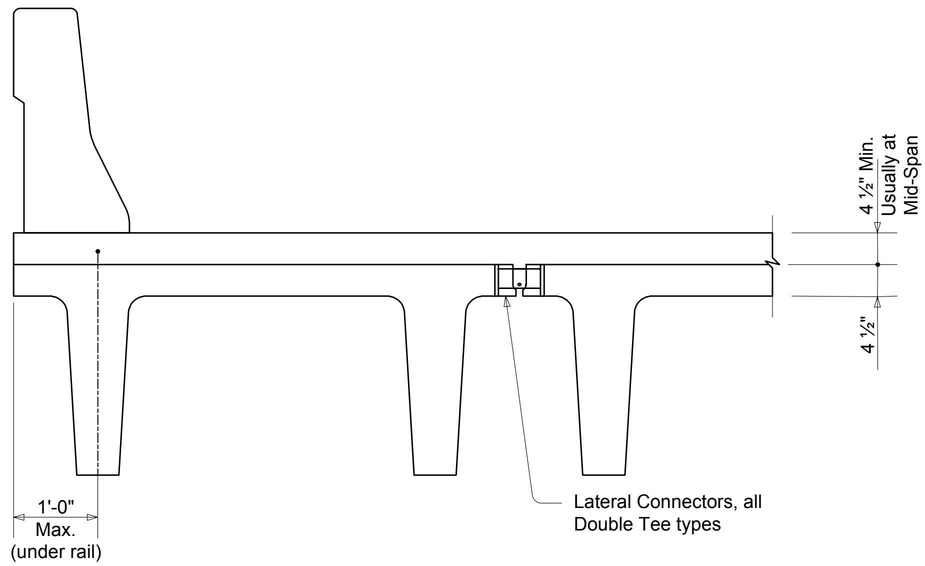


Figure 7.8.2 Prestressed Double Tee Beam Typical Section

Prestressed double tees have a typical stem depth of 305 to 864 mm (12 to 34 inches). The average flange width is 2.4 to 3.1 m (8 to 10 feet), with a typical span length of approximately 7.6 to 16.8 m (25 to 55 ft). Prestressed double tees can be used in spans approximately 24.4 m (80 ft) long with stem depths up to 1.5 m (5 feet) and flange widths up to 3.7 m (12 feet). Prestressed double tee bridges are typically simple spans, but continuous spans have also been constructed. Continuity is achieved from span to span by forming the open section between beam ends, placing the required reinforcement, and casting concrete in the void area. Once the concrete reaches its design strength, the spans are considered to be continuous for live load.
In some prestressed double tee designs, the depth of the stems at the beam end is dapped, or reduced (see Figure 7.8.3). This occurs so that the beam end can sit flush on the bearing seat.

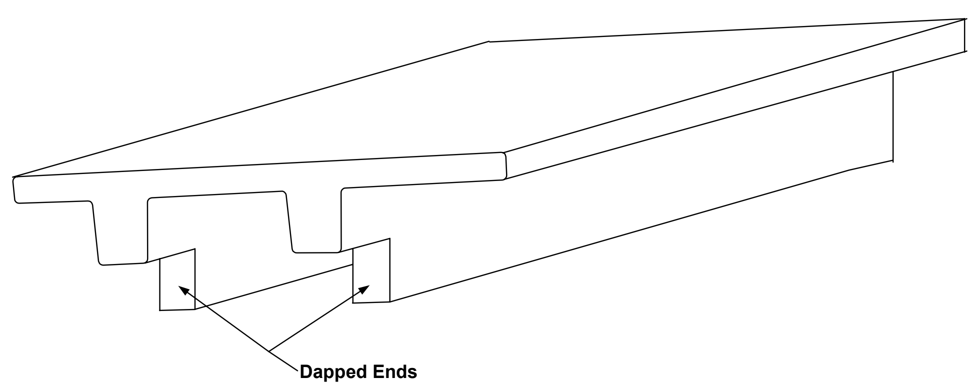


Figure 7.8.3 Dapped End of a Prestressed Double Tee Beam

The top of the flange or deck section of prestressed double tees can act as the wearing surface or be overlaid. Bituminous asphalt and concrete are typical examples of wearing surfaces that may be applied. See Topic 5.2.3 for a detailed description of the different types of concrete deck wearing surfaces.

Primary Members and Secondary Members

The primary members of a prestressed double tee beam are the stems and the deck.

The secondary members of a prestressed double tee bridge are the transverse diaphragms. The diaphragms are located at the span ends. They connect adjacent stems and prevent lateral movement. In the case of longer spans, intermediate diaphragms may also be placed to compensate for torsional forces. The diaphragms can be constructed of reinforced concrete or steel.

Steel Reinforcement

The primary tension steel reinforcement consists of prestressing strands and mild shear reinforcement (see Figure 7.8.4). The prestressing strands are placed longitudinally in each stem at the required spacing and clearance. When the double tees are to be continuous over two or more spans, conduits may be draped through the stems of each span to allow for post-tensioning. The shear reinforcement in a prestressed double tee beam consists of vertical U-shaped stirrups that extend from the stem into the flange. The shear reinforcement or stirrups are spaced along the length of the stem at a spacing required by design. The primary reinforcement for the deck or flange section of a prestressed double tee beam follows the reinforcement pattern of a typical concrete deck (see Topic 5.2.2). In some wider applications, the deck or flange portions of adjacent prestressed double tee beams may be transversely post-tensioned together through post-tensioning ducts. Transverse post-tensioning decreases the amount of damage that can occur to individual flange sides due to individual deflection and helps the double tee beams deflect as one structure.

The secondary or temperature and shrinkage reinforcement is placed longitudinally on each side of each stem and is tied to the vertical shear stirrups. In some newer designs, welded-wire-fabric is used as the shear and secondary reinforcement. The vertical bars in the welded-wire-fabric act as the shear reinforcement and the longitudinal bars perform as the secondary reinforcement. Tests have shown that temperature and shrinkage cracking can be reduced when welded-wire-fabric is used.

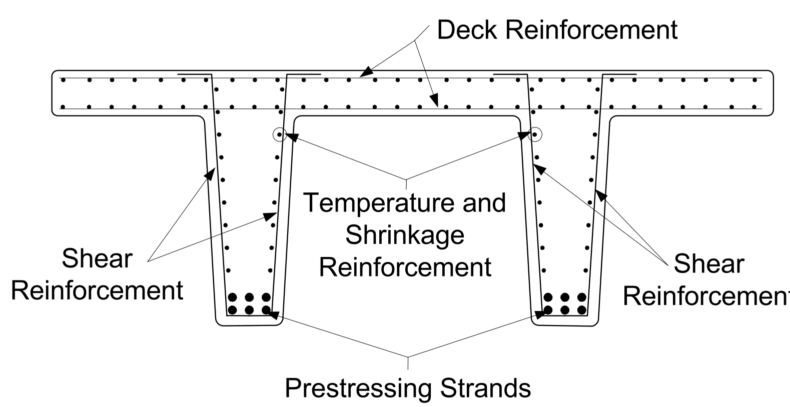


Figure 7.8.4 Steel Reinforcement in a Prestressed Double Tee Beam

7.8.3 Overview of Common Defects

Common defects that occur on prestressed concrete double tee beam bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Mild reinforcing steel corrosion
- Stress corrosion of prestressing strands

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.8.4 Inspection Procedures Locations

The inspection procedures and locations for concrete decks, as described in Topic 5.2.6, should be followed when inspecting the deck area of a prestressed double tee beam superstructure. For the stems, diaphragms, and other general locations, the bridge inspector should take into account the following:

Procedures

Visual

The inspection of concrete decks for cracks, spalls, and other defects is primarily a visual activity. However, hammers and chain drags 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

The physical examination of a deck with a hammer can be a tedious operation. In most cases, a chain drag is used. A chain drag is made of several sections of chain attached to pipe that has a handle attached to it. The inspector drags this across a deck and makes note of the resonating sounds. A chain drag can usually cover about a 915-mm (3-feet) wide section of deck at a time (see Figure 5.2.8).

If the inspector deems it necessary, core samples can be taken from the deck and sent to a laboratory to determine the extent of any chloride contamination.

Many of the problems associated with concrete bridge decks are caused by corrosion of the rebar. When the deterioration of a concrete deck progresses to the point of needing rehabilitation, an in-depth inspection of the deck 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 13.2.2, 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 13.2.3, include:

- Core Sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

- Examine bearing areas for spalling where friction from thermal movement and high bearing pressure could cause the concrete to spall. Check for crushing of the stem near the bearing seat. Check the condition and operation of any bearing devices.

- For dapped-end double tee beams, look for vertical flexure cracks and diagonal shear cracks in the reduced depth section that sits on the bearing seat. At the full depth-to-reduced depth vertical interface, check for vertical direct shear cracking. At the bottom corner where the reduced section meets the full depth section, check for diagonal shear corner cracks. At the bottom corner of the full depth section, check for diagonal tension cracks (see Figure 7.8.5).

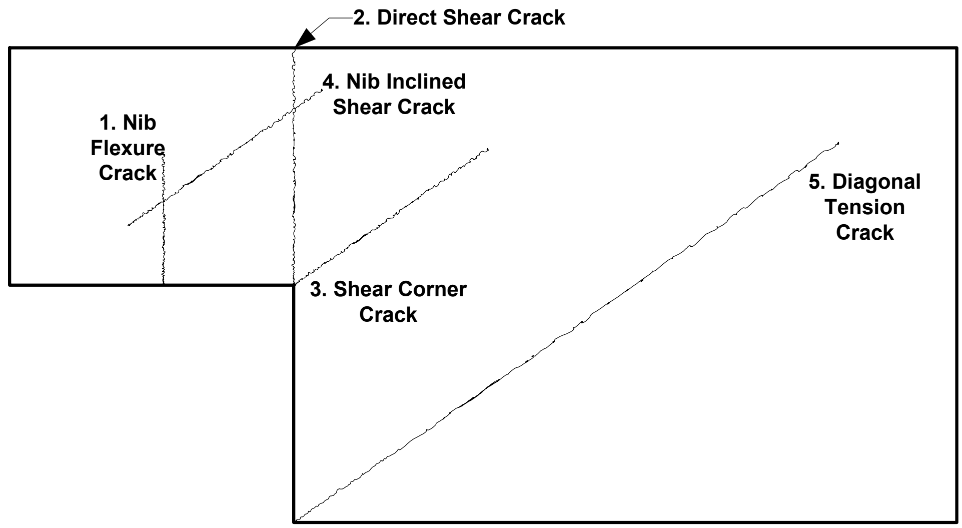


Figure 7.8.5 Crack Locations for Dapped End Double Tee Beams

Shear Zones

- Investigate the area near the supports for the presence of shear cracking. The presence of transverse cracks on the underside of the stems or diagonal cracks on the sides of the stem indicate the onset of shear failure. These cracks represent lost shear capacity and should be carefully measured.

Tension Zones

- Tension zones should be examined for flexure cracks, which would be transverse across the bottom of the stems and vertical on the sides. The tension zones are at the midspan along the bottom of the stem for both simple and continuous span bridges. Additional tension zones are located on the slab over the piers of continuous spans.

- Check for deteriorated concrete near the tension zones, which could result in the debonding of the tension reinforcement. This would include delamination, spalls, and contaminated concrete.

Diaphragms

- The diaphragms are designed as simple beams and should be inspected for flexure and shear cracks as well as typical concrete defects. Cracks in the diaphragms could be an indication of overstress or excessive differential deflection in the double tee beams.

Areas Exposed to Drainage

- If the roadway surface is bare concrete, check for delamination, scaling and spalls. The curb lines are most suspect. If the deck has an asphalt wearing surface, check for indications of deteriorated concrete such as reflective cracking and depressions.

- Check around scuppers or drain holes and deck fascias for deteriorated concrete.

- Check areas exposed to drainage for concrete spalling or cracking. This may occur at the ends of the beams where drainage has seeped through the deck joints.

Areas Over Traffic

- For grade crossing structures, check areas of damage caused by collision. This will generally be a corner spall with a few exposed rebars or prestressing strands.

Previous Repairs

- Examine areas that have been previously repaired. Determine if the repairs are in place and if they are functioning properly.

General

- Check for efflorescence from cracks and discoloration of the concrete caused by rust stains from the reinforcing steel. In severe cases, the reinforcing steel may become exposed due to spalling. Document the remaining cross section of reinforcing steel since section loss will decrease live load capacity. See Table 2.2.2 for concrete crack width guidelines.

- Using a string line, check for horizontal alignment and camber of the prestressed double tee beams. Signs of downward deflection usually indicate loss of prestress. Signs of excessive upward deflection usually indicate extreme creep and shrinkage.

7.8.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. 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 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. For prestressed double tees, the deck condition influences the superstructure component rating. When the deck component rating is 4 or less, the superstructure component rating may be reduced if the recorded deck defects reduce its ability to carry applied stresses associated with superstructure moments.

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 top of slab and the underside. 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. Smart Flags are also used to describe the condition of the concrete superstructure.

In an element level condition state assessment of a prestressed double tee beam bridge, the AASHTO CoRe element is one of the following, depending on the riding surface:

Element No.	Description
012	Concrete Deck – Bare
013	Concrete Deck – Unprotected with AC Overlay
014	Concrete Deck – Protected with AC Overlay
018	Concrete Deck – Protected with Thin Overlay
022	Concrete Deck – Protected with Rigid Overlay
026	Concrete Deck – Protected with Coated Bars
027	Concrete Deck – Protected with Cathodic System
109	P/S Concrete Open Girder/beam

The unit quantity for the deck elements is "each", and the entire element must be placed in one of the five available condition states based solely on the surface condition. Some states have elected to use the total area (m^2 or ft^2). The inspector must know the total slab surface area in order to calculate a percent deterioration and fit it into a given condition state description. The unit quantity for the prestressed double tee beam is meters or feet, and the total length of all beams must be placed in one of the four available condition states. Condition state 1 is the best possible rating for the deck or beam. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

For structural cracks in the surface of bare slabs, the "Deck Cracking" Smart Flag, Element No. 358, can be used and one of four condition states assigned. Do not use Smart Flag, Element No. 358, if the bridge deck/slab has any overlay because the top surface of the structural deck is not visible. For concrete defects on the underside of a slab element, the "Soffit" Smart Flag, Element No. 359, can be used and one of five condition states assigned. For damage due to traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of three condition states assigned.

Table of Contents

Section 7
Inspection and Evaluation of Common Concrete Superstructures

Topic 7.9 Prestressed I-Beams

7.9.1 Introduction

Prestressed I-beams and bulb-tees have been used since the 1950's. They have proven to be successful because of their material saving shapes and their light weights. The I or T shape allows a designer to have enough space to place the proper amount of reinforcement while reducing the amount of concrete needed (see Figure 7.9.1).

Figure 7.9.1 Prestressed I-Beam Superstructure

7.9.2 Design Characteristics

Prestressed I-beams and bulb-tees make economical use of material since most of the concrete mass is located away from the neutral axis of the beam.

Standard Shapes

Prestressed I-beams are shaped to provide minimum dead load with ample space for tendons. The most common prestressed concrete I-beam shapes are the AASHTO shapes used by most state highway agencies (see Figure 7.9.2). However, some highway agencies have developed variations of the AASHTO shapes to accommodate their particular needs.

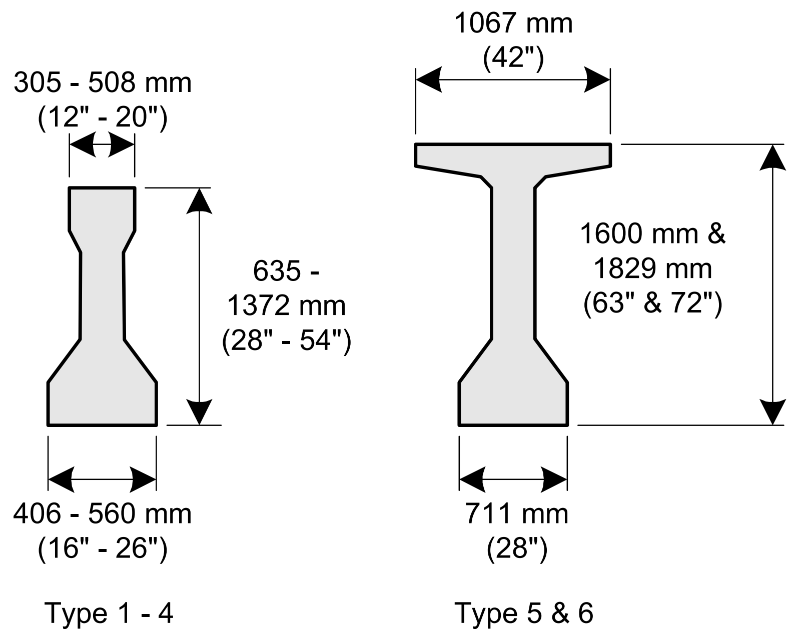


Figure 7.9.2 AASHTO Cross Sections of Prestressed I-Beams

Prestressed I-beams are used in spans ranging from 6 to 46 m (20 to 150 feet). They are generally most economical at spans from 18 to 35 m (60 to 115 feet).

Materials – Strength and Durability

Steel tendons with tensile strength as high as 1860 Mpa (270 ksi) are located in the bottom flange. These tendons are used to induce compression across the entire section of the beam prior to and during application of live load. This results in a crack free beam.

New technology may allow designers to reduce corrosion of prestressing strands. This reduction is made possible by using composite materials in lieu of steel. Carbon or glass fibers are two alternatives to steel prestressing strands that are being researched.

Concrete used is also of higher strength ranging from 34 Mpa (5,000 psi) compressive strength up to 68 Mpa (10,000 psi). In addition, concrete has a higher quality due to better control of fabrication conditions in a casting yard.

Reactive Powder Concrete (RPC) prestressed I-beams can come in an X shape (see Figure 7.9.3) or other concrete beam shapes. RPC prestressed beams may have an hourglass shape so as to take maximum advantage of RPC properties. Tested prestressed RPC I-beams are made without any secondary steel reinforcement and can carry the same load as a steel I-beam with virtually the same depth and weight.

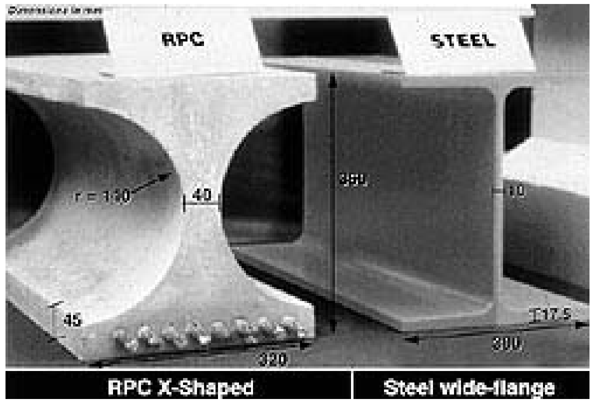


Figure 7.9.3 Reactive Powder Concrete (RPC) Prestressed X-Beam

Reactive Powder Concrete (RPC) creates a better bond between the cement and aggregate. This bond produces a material with a higher density, shear strength, and ductility than normal strength concrete. Silica fume is one of the ingredients in Reactive Powder Concrete that increases the strength. RPC prestressed I-beams are effective in situations where steel I-beams may be used, but are not effective where conventional strength prestressed concrete I-beams are strong enough.

Continuity

To increase efficiency in multi-span applications, prestressed I-beams can be made continuous for live load and/or to eliminate the deck joint. This is done using a continuous composite action deck and anchorage of mild steel reinforcement in a common end diaphragm (see Figure 7.9.4). Continuity has also been accomplished using posttensioning ducts cast into pretensioned I-beams. Tendons pulled through these ducts across several spans then are stressed for continuity. Cast-in-place concrete diaphragms are framed around the beams at the abutments and piers.

Figure 7.9.4 Continuous Prestressed I-Beam Bridge

Composite Action

The deck is secured to and can be made composite with the I-beam by the use of extended stirrups which are cast into the I-beam (see Figure 7.9.5).

Figure 7.9.5 Cast-In-Place Stirrups

Primary Members and Secondary Members

The primary members are the prestressed I-beams. The secondary members are the end diaphragms and the intermediate diaphragms. End diaphragms are usually full depth and located at the abutments or piers. Intermediate diaphragms are partial depth and are used within the span for longer spans (see Figure 7.9.6). Diaphragms are cast-in-place concrete or rolled steel sections and are placed at either the mid points or third points along the span.

Figure 7.9.6 Concrete End Diaphragm

Steel Reinforcement

Primary Reinforcement

Primary reinforcement consists of main tension steel and shear reinforcement or stirrups.

High Strength Steel

Main tension steel consists of pretensioned high strength prestressing strands or tendons placed symmetrically in the bottom flange and lower portion of the web. Strands are 9.5, 11.1, 12.7, or15.2 mm (3/8, 7/16, 1/2 or 0.6 inch) in diameter and are generally spaced in a 50.8 mm (2 inch) grid. In the larger beams main tension steel can include posttensioned continuity tendons which are located in ducts cast into the beam web (see Figure 7.9.7).

Mild Steel

Mild steel stirrups are vertical in the beam and located throughout the web at various spacings required by design (see Figure 7.9.7).

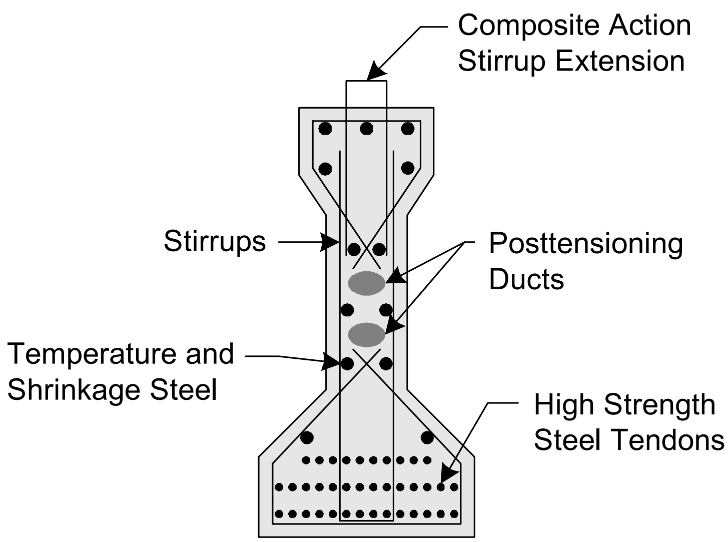


Figure 7.9.7 Prestressed I-Beam Reinforcement (Schematic)

Other Reinforcement

Other reinforcement includes mild steel temperature and shrinkage reinforcement which is longitudinal in the beam.

Composite Strands

Composite strands can be carbon fiber or glass fiber and are fairly new to the bridge prestressing industry. These strands are gaining acceptance due to the low corrosive properties compared to steel strands and will just be mentioned in this manual.

7.9.3 Overview of Common Defects

Common defects that occur on prestressed I-beams and bulb-tees include:

- Cracking
- Delamination
- Spalling
- Collision damage
- Overload damage
- Reinforcing/prestressing steel corrosion
- Stress corrosion
- Efflorescence
- Pop-outs

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.

7.9.4 Inspection Procedures and Locations

Inspect a prestressed I-beam bridge and a bulb-tee bridge at the following locations using the following procedures:

Procedures

Visual

The inspection of concrete I-beams for cracks, spalls, and other defects is primarily a visual activity. However, hammers are primarily used to detect areas of delamination. A delaminated area will have a distinctive hollow “clacking” sound when tapped with a hammer. A hammer hitting sound concrete will result in a solid "pinging" type sound.

Physical

The physical examination of I-beams with a hammer can be a tedious yet required operation. Many of the problems associated with concrete I-beams are caused by corrosion of the rebar. When the deterioration of a concrete I-beam progresses to the point of needing rehabilitation, an in-depth inspection of the I-beam 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 13.2.2, 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 Topic13.2.3, include:

- Core Sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Advanced inspection techniques have been developed that can evaluate fatigue damage to steel reinforcement in a concrete member. 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.

Locations

Bearing Areas

- Check bearing areas for spalls or vertical cracks (see Figure 7.9.8). Spalls and cracks may be caused by corrosion of steel due to water leakage or restriction of thermal movement due to a faulty bearing mechanism. Spalling could also be caused by poor quality concrete (see Figure 7.9.9).
- Check for crushing of flange near the bearing seat.
- Check for rust stains which indicate corrosion of steel reinforcement.

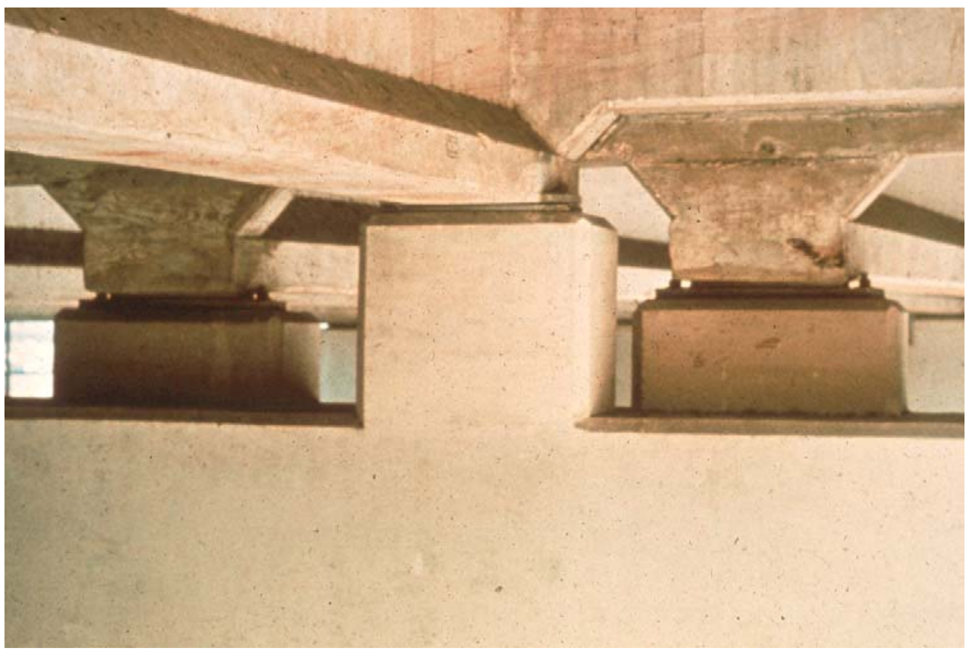


Figure 7.9.8 Bearing Area of a Typical Prestressed I-beam

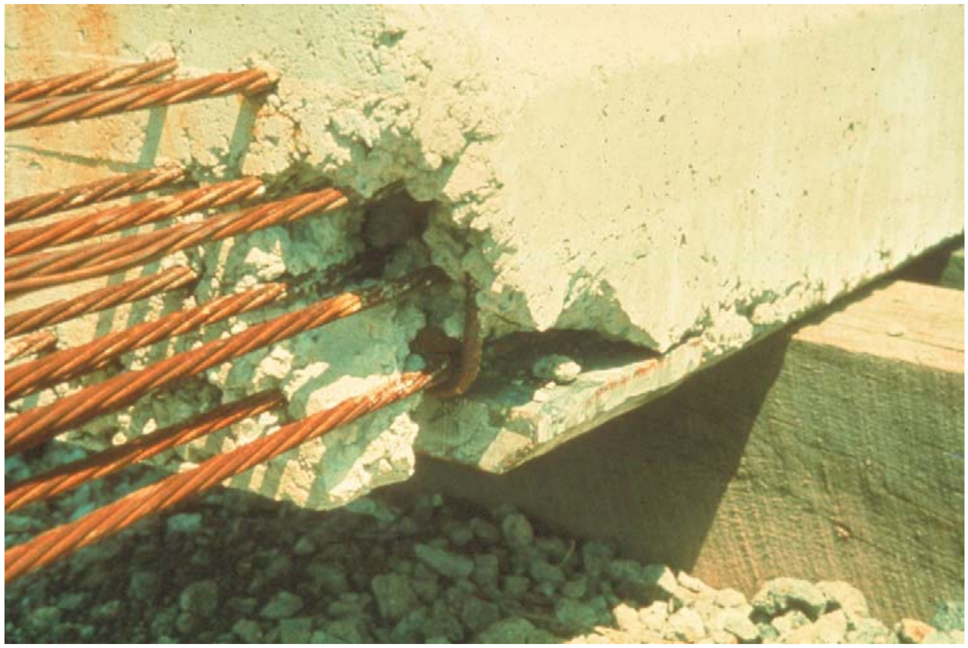


Figure 7.9.9 Spalling Due to Poor Concrete

Shear Zones

- Check beam ends and sections over piers for diagonal shear cracks in webs. These cracks will project diagonally upward from the support toward midspan.

Tension Zones

- Inspect the tension zones of the beams for structural cracks. Any crack should be carefully measured with an optical crack gauge and documented.

- Check for deteriorated concrete that could cause debonding of the tension reinforcement. This would include spalls, delamination, and cracks with efflorescence.

- Check bottom flange for longitudinal cracks that may indicate a deficiency of prestressing steel, insufficient cover, inadequate spacing, or possibly an overloading of the concrete due to use of prestressing strands that are too large.

- Check bottom flange at midspan for flexure cracks due to positive moment (see Figure 7.9.10). These cracks will be quite small and difficult to detect. An optical crack gauge should be used to measure any cracks found.

- For continuous bridges, check the deck area over the piers for flexure cracks due to negative moment.

- Check for rust stains from cracks, indicating corrosion of steel reinforcement or prestressing tendons.

- Check for exposed tension reinforcement and document section loss. Measurable section loss will decrease live load capacity. Exposed prestressing tendons are susceptible to stress corrosion and sudden failure.

Figure 7.9.10 Flexure Crack

Diaphragms

- Inspect the fixed diaphragms for spalling or diagonal cracking (see Figure 7.9.11). This is a possible sign of overstress caused by structure movement or excessive deflection.

- Investigate the intermediate diaphragms for cracking and spalling concrete. Flexure and shear cracks may indicate excessive differential movement of the I-beams.

Figure 7.9.11 Typical Concrete Diaphragm

Areas Exposed to Drainage

➢ Check around scuppers, inlets or drain holes for leaking water or deterioration of concrete (see Figure 7.9.12).

Figure 7.9.12 Leakage of Water at Inlet

Areas Over Traffic

- Check areas damaged by collision. A significant amount of prestressed concrete bridge deterioration and loss of section is due to traffic damage. Document the number of exposed and severed strands as well as the loss of concrete section. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss (see Figure 7.9.13).

Figure 7.9.13 Collision Damage on Prestressed Concrete I-Beam

Previous Repairs

Examine thoroughly any repairs that have been previously made. Determine if repaired areas are sound and functioning properly. Effective repairs and patching are usually limited to protection of exposed tendons and reinforcement (see Figure 7.9.14).

Figure 7.9.14 Collision Damage Repair on Prestressed Concrete I-Beam. Note Epoxy Injection Ports

General

- Using a string line, check for horizontal alignment and camber of the prestressed beams. Signs of downward deflection usually indicates loss of prestress. Signs of excessive upward deflection usually indicates extreme creep and shrinkage.

7.9.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the element level 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 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 prestressed I-beams. 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 concrete superstructure.

In an element level condition state assessment of a prestressed I-beam or bulb-T bridge, the AASHTO CoRe element is:

Element No.	**Description**
109	Concrete Open Girder/beam

The unit quantity for prestressed I-beams is meters or feet and the total length of all beams must be placed in one of the four available condition states. Condition state 1 is the best possible rating for the beam. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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.

Table of Contents

Section 7

Inspection and Evaluation of Common Concrete Superstructures

Topic 7.10 Prestressed Box Beams

7.10.1 Introduction

Prestressed box beams have become quite popular since the 1960's (see Figure 7.10.1). These precast members provide advantages from a construction and an economical standpoint by increasing strength while decreasing the dead load.

Figure 7.10.1 Typical Box Beam Bridge

7.10.2 Design Characteristics

General

Prestressed box beams are constructed having a rectangular cross section with a single rectangular void inside. Many prestressed box beams constructed in the 1950's have single circular voids. The top and bottom slabs act as the flanges, while the side walls act as webs. The prestressing reinforcement is placed in the bottom flange and into both webs (see Figure 7.10.2).

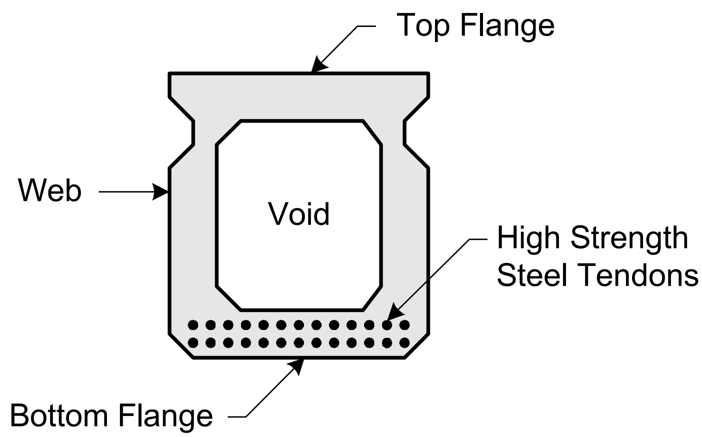


Figure 7.10.2 Schematic of a Typical Box Beam Cross-Section

Prestressed box beams are typically either 914 to 1219 mm (36 or 48 inches) wide. The depth of a box beam is typically 305, 432, 533, 686, 838, 914 or 1067 mm (12, 17, 21, 27, 33, 36, or 42 inches). Wall thickness ranges from 76 to 152 mm (3 to 6 inches).

The typical span length for prestressed concrete box beams ranges from 8 to 28 m (25 to 90 ft) depending on the beam configuration and spacing.

Design

Simple/Continuous Spans

Prestressed box beams can be simple or continuous spans. In the case of a simple span, the ends of the beams from span to span are not connected together at the support. An expansion joint is placed over the support in the concrete deck and the spans act independently. For continuous spans, the beam-ends from span to span are connected together by means of a cast-in-place concrete end diaphragm over the support. Mild steel reinforcement is placed in this diaphragm area and is spliced with steel reinforcement from the prestressed box beams. Continuous spans provide advantages such as eliminating deck joints, making a continuous surface for live loads, distributing live loads, and lowering positive moment.

Composite/Non-composite

Prestressed box beams can be composite or non-composite. By design, some prestressed box beams are constructed with the top of stirrups extending out of the top flange (see Figure 7.10.3). These stirrup tops are engaged when a cast-in-place concrete deck is placed and hardens. Once the concrete deck hardens, the deck becomes composite with the prestressed box beams. Prestressed box beams can also be non-composite. If the stirrups are not extended into the deck, the prestressed box beams cannot act integrally with the deck.

Figure 7.10.3 Box Beams at Fabrication Plant Showing Shear Connectors and Extended Rebar for Continuity

Construction

Box beams are constructed similar to I-beams, with high strength steel strands or tendons placed in the bottom flange and lower web area. The strength of the steel strands can be as high as 1860 MPa (270 ksi).

Concrete compressive strengths of 27 to 41 MPa (4000 to 6000 psi) are typically used in prestressed box beams, but concrete with ultimate strengths over 68MPa (10,000 psi) is available and becoming popular.

High performance concrete (HPC), which is a new type of concrete being used in bridge members, is designed to meet the specific needs of a specific project. The mix design is based on the environmental conditions, strength requirements, and durability requirements. This type of concrete allows engineers to design smaller, longer, and more durable members with longer life expectancies.

Advantages

Dead Load Reduction

Box beams are advantageous in that they allow for a reduced deck slab thickness, and therefore reduced dead load. Also, the voided beam reduces the dead load.

Construction Time Savings

Precast members are cast and cured in a quality controlled casting yard. Because box beams are precast, the construction process takes less time. When construction is properly planned, using precast members allows the structure to be erected with no down time due to concrete curing.

Shallow Depth

Prestressed box beams are designed with a typical maximum depth of 1067 mm

(42 inches). This shallow depth makes box beams viable solutions for field conditions that have tight vertical clearances.

Applications

There are two applications of prestressed box beams (see Figure 7.10.4):

- Adjacent box beams
- Spread box beams

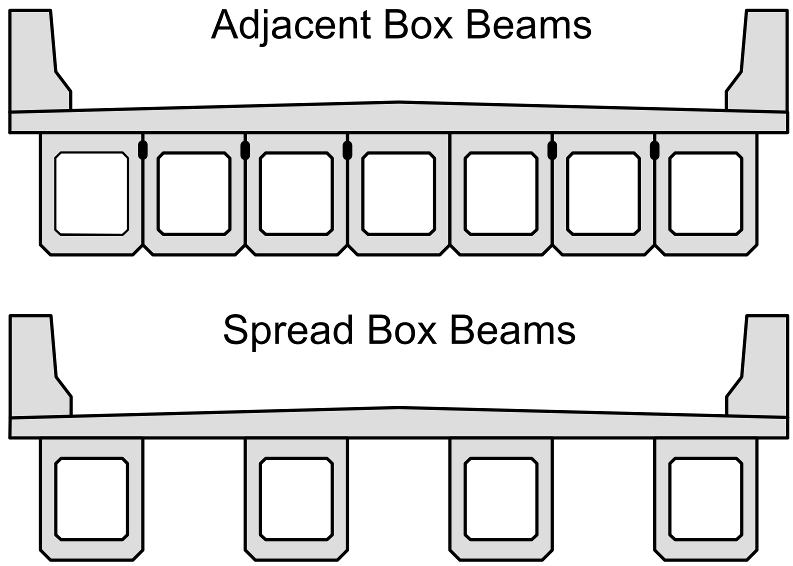


Figure 7.10.4 Applications of Prestressed Box Beams

Adjacent Box Beams

On an adjacent box beam bridge, the adjacent box beams are placed laterally side by side with no space between them. In some early applications, the top flange of each box is exposed and functions as the deck (see Figure 7.10.5). The practical span lengths range from 6.1 to 39.6 m (20 to 130 feet), with the most economical spans ranging from 12.2 to 27.4 m (40 to 90 feet).

Figure 7.10.5 Adjacent Box Beams Acting as the Deck

In modern longer span applications, the deck is typically a cast-in-place composite concrete deck. For composite decks, stirrups extend above the top of the box to provide the transfer of shear forces. For most shorter spans, nonstructural asphalt overlays with membrane waterproofing are applied.

This configuration of adjacent boxes is also called multiple boxes.

Monolithic Action

Like precast slab units, adjacent box beams are post tensioned laterally. This is generally done using threaded bars and lock nuts or tendons with locking wedges. Lateral post tensioning combined with grouted shear keys provides for monolithic action (see Figure 7.10.6).

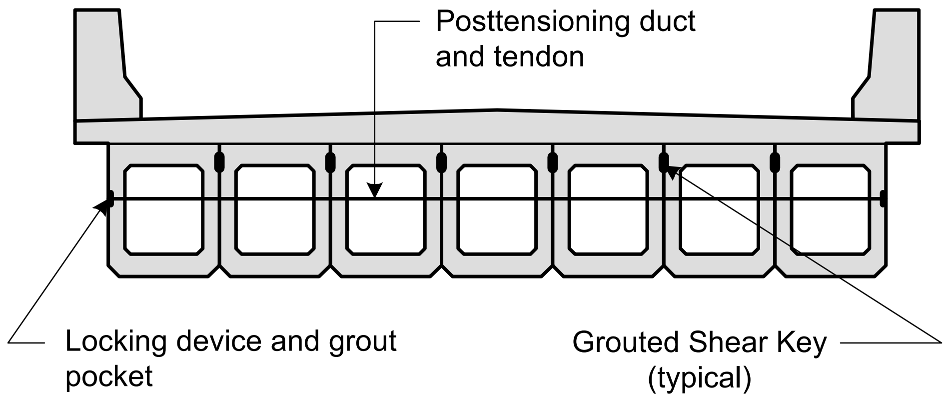


Figure 7.10.6 Schematic of Lateral Post-tensioning of an Adjacent Box Beam Bridge

Spread Box Beams

On a spread box beam bridge, the box beams are usually spaced from 610 to 1830 mm (2 to 6 feet) apart and typically use a composite cast-in-place concrete deck (see Figure 7.10.7). This application is practical for span lengths from 8 to 26 m (25 to 85 feet). Stay-in-place forms or removable formwork is used between the box beams to provide a support when the concrete deck is poured.

Figure 7.10.7 Underside of a Typical Spread Box Beam

All modern box beams have drain holes in the bottom to allow any moisture in the void to escape.

Primary Members and Secondary Members

The primary members of box beam bridges are the concrete box beams. External diaphragms are the only secondary members on box beam bridges, and they are only found on spread box beam bridges (see Figure 7.10.8). The diaphragms may be cast-in-place, precast, or steel and are placed at either the mid points or third points along the span.. They should be inspected as a beam. As with I-beams, they can provide restraint and act as a backwall. End diaphragms are located at the abutments and piers and can be full or partial depth. Intermediate diaphragms are located between bearing points and are usually partial depth.

Figure 7.10.8 External Diaphragms on a Spread Box Beam Bridge

Internal Diaphragms are considered a part of the prestressed box beams and not a secondary member (see Figure 7.10.9).

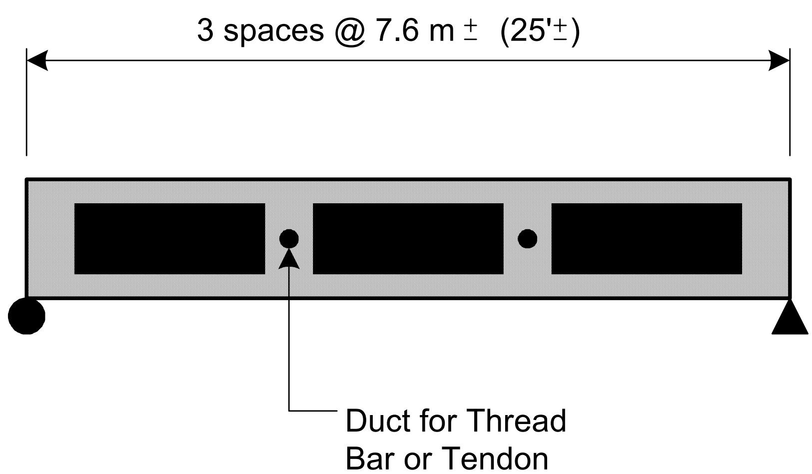


Figure 7.10.9 Schematic of Internal Diaphragms

Steel Reinforcement

Primary Reinforcement

Primary reinforcement consists of main tension steel and shear reinforcement or stirrups.

High Strength Steel

Main tension steel consists of high strength pretensioned prestressing strands placed in the bottom flange and lower web of the box beam. Depending on the age of the structure, the strand size will be 6, 10, 11 or 13 mm (1/4, 3/8, 7/16, or 1/2 inch) in diameter and spacing is normally 51 mm (2 inches) apart (see Figure 7.10.10). In some newer applications of prestressed box beams using HPC, 15mm (0.6-inch) strand sizes with a spacing of 51mm (2 inches) are used to fully implement the increased concrete strengths.

Mild Steel

Mild steel stirrups are placed vertically in the web at spacings required by design for shear reinforcement.

Other reinforcement

Other reinforcement includes transverse post-tensioning strands through the diaphragms and mild temperature and shrinkage reinforcement that runs longitudinal in the beam.

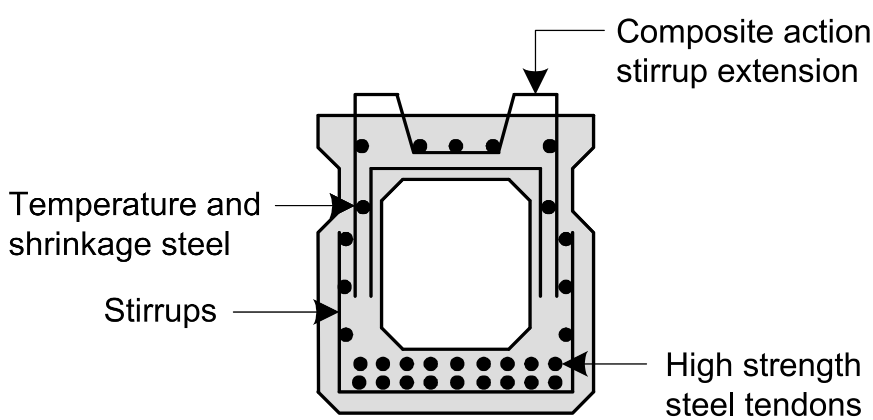


Figure 7.10.10 Schematic of Typical Prestressed Box Beam Reinforcement

Composite Strands

Composite strands can be carbon fiber or glass fiber and are fairly new to the bridge prestressing industry. Refer to Topic 7.9.2 for a brief explanation of composite strands.

7.10.3 Overview of Common Defects

Common defects that occur on prestressed box beams include:

- Cracking
- Delamination
- Spalling
- Collision damage
- Overload damage
- Reinforcing/prestressing steel corrosion
- Stress corrosion
- Efflorescence
- Pop-outs

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.

7.10.4 Inspection Procedures and Locations

Since prestressed box beams are designed to maintain all concrete in compression, cracks are indications of serious problems. For this reason, any crack should be carefully measured with an optical crack gauge and documented.

Inspect a prestressed box beam bridge using the following procedures:

Procedures

Visual

The inspection of concrete box beams for cracks, spalls, and other defects is primarily a visual activity. However, hammers are primarily used to detect areas of delamination. A delaminated area will have a distinctive hollow “clacking” sound when tapped with a hammer. A hammer hitting sound concrete will result in a solid "pinging" type sound.

Physical

The physical examination of box beams with a hammer is a tedious yet required operation. Many of the problems associated with concrete box beams are caused by corrosion of the rebar. When the deterioration of a concrete box beam progresses to the point of needing rehabilitation, an in-depth inspection of the box beam 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 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Bearing Areas

- The top of the beam-ends should be examined for horizontal or vertical cracks. These cracks indicate a deficiency of reinforcing steel. These cracks are caused by the stresses created at the transfer of the prestressing forces.

- Check bearing areas for spalls or vertical cracks. Spalls and cracks may be caused by corrosion of steel due to water leakage or restriction of thermal movement due to a faulty bearing mechanism (see Figure 7.10.11).

- Check for rust stains, which indicate corrosion of steel reinforcement (see Figure 7.10.12).

- Check the bottom of beams for longitudinal cracks originating from the bearing location. These cracks are sometimes caused by the unbalanced transfer of prestress force to the concrete, or by the accumulation of water inside the box, freezing and thawing (see Figure 7.10.13).

Figure 7.10.11 Spalled Beam Ends

Figure 7.10.12 Exposed Bars at End of Box Beam

Figure 7.10.13 Longitudinal Cracks in Bottom Flange at Beam

Shear Zone

- Check beam ends and sections over piers for diagonal shear cracks in webs. These cracks will project diagonally upward from the support toward midspan.

Tension Zones

- Investigate the lower portion of the beam, particularly at mid span, for flexure cracks. This indicates a very serious problem resulting from overloading or loss of prestress.

- Check for spalling, delamination and exposed reinforcing steel. Exposed strands fail prematurely due to stress corrosion (see Figure 7.10.14).

- Check for deteriorated concrete, which could cause debonding of the tension reinforcement. This would include spalls, delamination, and cracks with efflorescence.

- Check bottom flange for longitudinal cracks which may indicate a deficiency of prestressing steel, or possibly an overloading of the concrete due to use of prestressing forces that are too large.

- For continuous bridges, check the deck area over the supports for flexure cracks due to negative moment.

- An advanced inspection technique has been developed that can evaluate fatigue damage to steel reinforcement in a concrete member. 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.

Figure 7.10.14 Spall and Exposed Reinforcement

Diaphragms

- Inspect the fixed diaphragms for spalling or diagonal cracking. This is a possible sign of shear failure caused by structure movement.

- Investigate the intermediate diaphragms for cracking and spalling concrete. Flexure and shear cracks may indicate excessive differential moment of the box beams.

Areas Exposed to Drainage

- Examine between boxes in adjacent box beam bridges for leakage and rust stains. Look for reflective cracking in the traffic surface and individual beam deflection under live load. These problems indicate that the shear key between boxes has been broken and that the boxes are acting independently of each other (see Figure 7.10.15). These problems could also indicate the transverse post-tensioning is not acting as designed.

- Check drain holes for proper function as accumulated water can freeze and crack the webs of the beam. Be careful not to open the drain onto yourself.

Figure 7.10.15 Joint Leakage and Rust Stain

Areas Over Traffic

- Check areas damaged by collision. A significant amount of prestressed concrete bridge deterioration and loss of section is due to traffic damage. Document the number of exposed and severed strands as well as the loss of concrete section. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss (see Figure 7.10.16).

Figure 7.10.16 Close-up of Collision Damage

Previous Repairs

- Examine thoroughly any repairs that have been previously made. Determine if repaired areas are functioning properly. Effective repairs and patching are usually limited to protection of exposed tendons and reinforcement.

General

- Examine the sides of the beams for cracks. Adjacent box beam side surfaces are visible only on the fascias. For interior beams, inspect the bottom chamfers for cracks, which may extend along the sides of the beams.

- Using a string line, check for horizontal alignment and camber of the prestressed beams. Signs of downward deflection usually indicates loss of prestress. Signs of excessive upward deflection usually indicates extreme creep and shrinkage.

- Note the presence of surface irregularities caused by burlap folds used in the old vacuum curing process. This dates the beam construction to the early 1950’s and should alert the inspector to possible deficiencies common in early box beams, such as inadequate or non-existent drainage openings and strand cover (see Figure 7.10.17).

Figure 7.10.17 Burlap Fold Depressions in an Early 1950's P/S Box Beam

7.10.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are the National Bridge Inspection Standards (NBIS) rating and the element level 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 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 prestressed box 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. Element level Smart Flags are also used to describe the condition of the concrete superstructure.

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

Element No.	**Description**
109	Concrete Open Girder/beam

The unit quantity for prestressed box beams is meters or feet and the total length of all beams must be placed in one of the four available condition states. Condition state 1 is the best possible rating for the beam. See the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements for condition state descriptions.

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.

This page intentionally left blank

Table of Contents

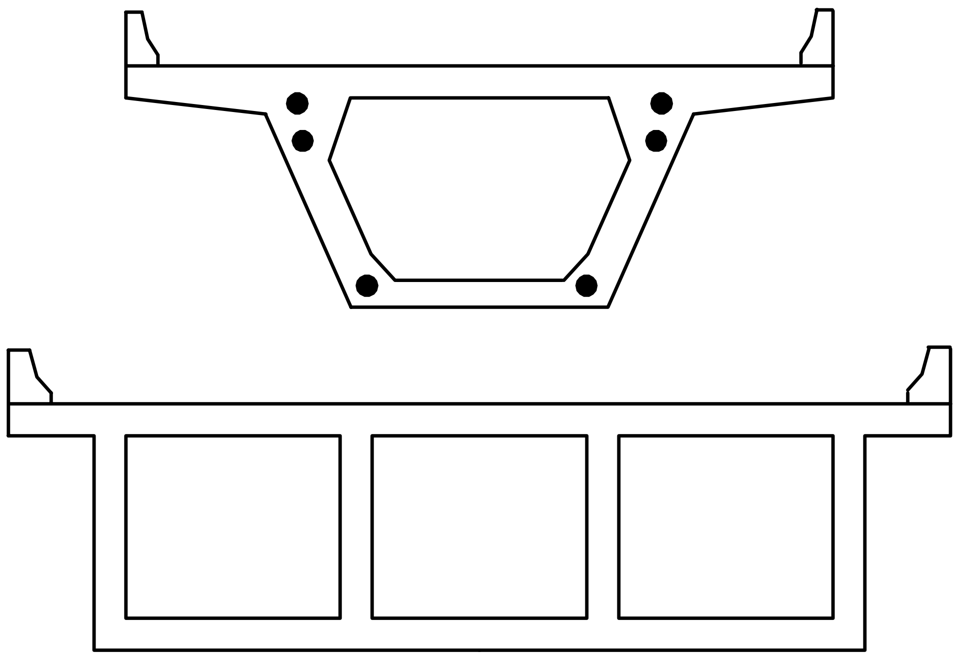
Section 7
Inspection and Evaluation of Common Concrete Superstructures

Topic 7.11 Concrete Box Girders

7.11.1 Introduction

The box girder bridge is the current state-of-the-art bridge type for concrete. Using a trapezoidal box shape with cantilevered top flange extensions, a single box girder combines mild steel reinforcement and high strength post-tensioning tendons into a cross section capable of accommodating an entire roadway width. Designs are common, although not yet standardized, for both segmental and monolithic box girder construction. In addition, reinforced concrete (mild steel reinforcement) box girder bridges were once commonly constructed for short spans, and many of those bridges still exist today.

Older box girder bridges can also be cast-in-place with mild steel reinforcement or post-tensioned (see Figures 7.11.1 and 7.11.2). This bridge type is popular in midwest states (e.g., Wisconsin, Minnesota, Michigan and Kansas).



Typical Cast-In-Place Box Girders

Figure 7.11.1 Typical Cast-in-place Box Girder Cross Section

Figure 7.11.2 Typical Cast-in-place Concrete Box Girder Bridge

7.11.2 Design Characteristics

Concrete Box Girder

For wide roadways, the box portion generally has internal webs and is referred to as a multi-cell box girder (see Figure 7.11.3). Concrete box girder bridges are typically either single span or continuous multi-span structures. Spans can have a straight or curved alignment and are generally in excess of 46 m (150 feet) (see Figure 7.11.4).

The following description applies to monolithic box girder construction only. A detailed description of segmental concrete bridges appears later in this Topic.

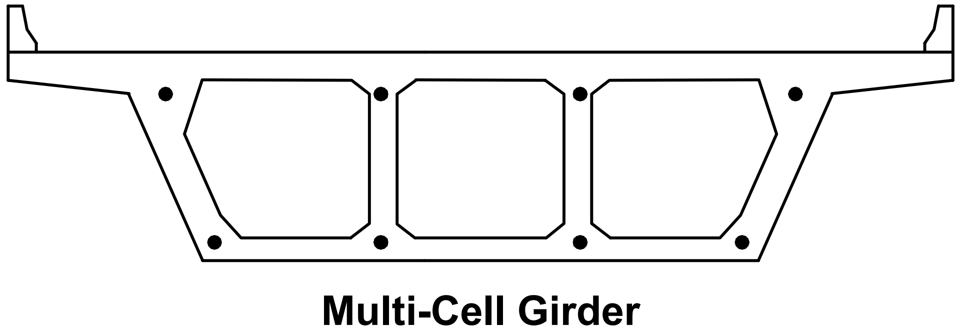


Figure 7.11.3 Multi-cell Girder

Figure 7.11.4 Typical Cast-in-place Concrete Box Girder Bridge

Construction Methods

The two basic construction techniques used for cast-in-place monolithic box girders are high level casting and at-grade casting.

High Level Casting

The high level casting method employs formwork supported by falsework. This technique is used when the structure must cross an existing feature, such as a roadway, railway, or waterway (see Figure 7.11.5).

Figure 7.11.5 High Level Formwork Support Scaffolding

At-grade Casting

The at-grade casting method employs formwork supported by fill material or the existing ground. When the construction is complete, the fill beneath the bridge is removed. This technique is used when the structure is crossing, or is part of a new highway system or interchange (see Figures 7.11.6 and 7.11.7). The at-grade casting method is common in Arizona and other southwestern states.

Figure 7.11.6 At-grade Formwork with Post-tensioning Ducts

Figure 7.11.7 Box Girder Bridge Construction Using At-grade Forming

Primary Members

For box girder structures, the primary member is the box girder. When a single-cell box girder design is used, the top flange or deck slab, the bottom flange, and both side walls are all primary elements of the box girder (see Figure 7.11.8). The top flange is considered an integral deck component.

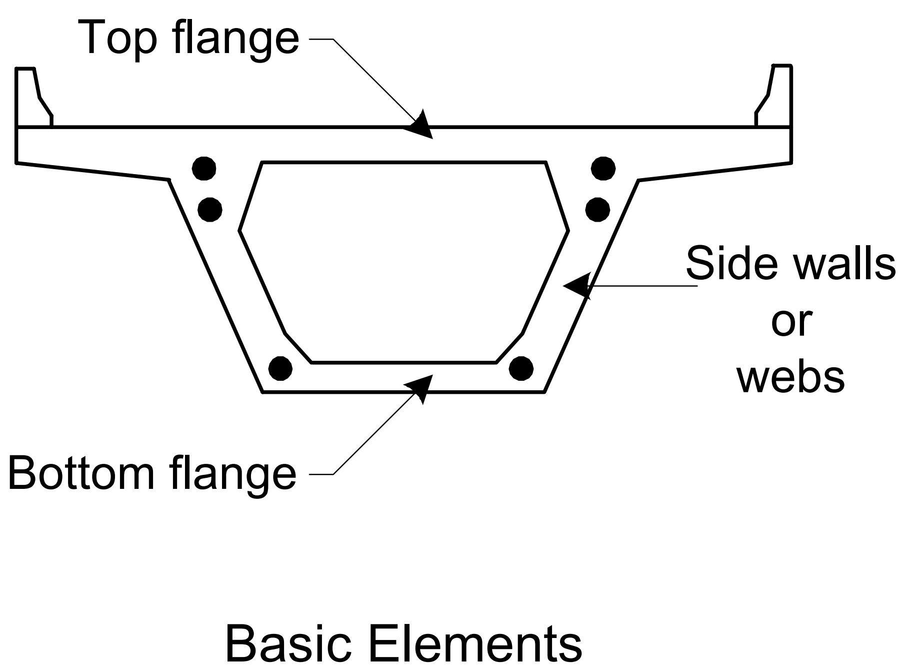


Figure 7.11.8 Basic Elements of a Cast-in-place Box Girder

In some multi-cell box girder applications, the top flange or deck slab must be removable for future replacement. The top flange in these cases functions similarly to a composite deck slab and is in fact considered a separate deck component. Most exterior webs have higher stress levels than interior webs, but the interior webs of the box also play a significant role in the girder (see Figure 7.11.9).

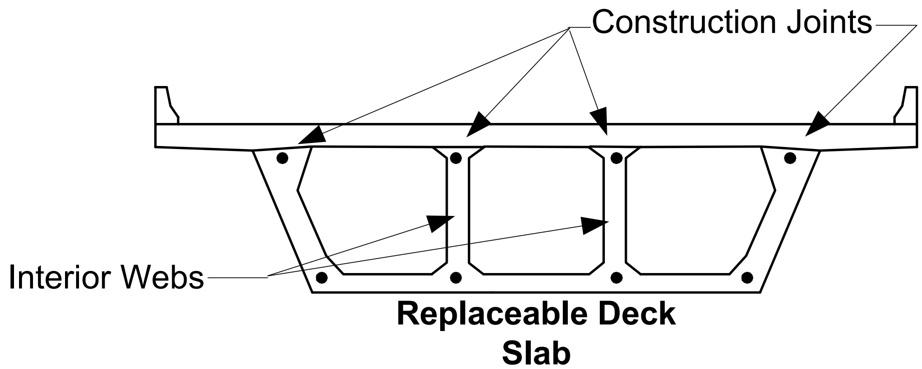


Figure 7.11.9 Replaceable Deck Slab on a Multiple Cell Cast-in-place Box Girder

Steel Reinforcement

Box girder structures use a combination of mild steel reinforcement and high strength post tensioning steel tendons (see Figure 7.11.10). Shear reinforcement is provided to resist standard beam action shear. For curved girder applications, torsional shear reinforcement is sometimes required. This reinforcement is provided in the form of additional stirrups.

Flexure reinforcement is provided in the top and bottom flanges of the box girder as necessary (bottom flange at midspan in areas of positive moment and top flange over supports in areas of negative moment). However, because of the design span lengths, mild steel reinforcement does not have sufficient strength to resist all of the tension forces.

To reduce these tensile stresses to acceptable levels, prestressing of the concrete is introduced through post-tensioning. Galvanized metal and polyethylene ducts are placed in the forms at the desired location of the tendons. When the concrete has cured to an acceptable strength level, the tendons are installed in the ducts, tensioned, and then grouted (see Figure 7.11.11).

Special “confinement” reinforcement is also required at the anchorage locations to prevent cracking due to the large transfer of force to the surrounding concrete.

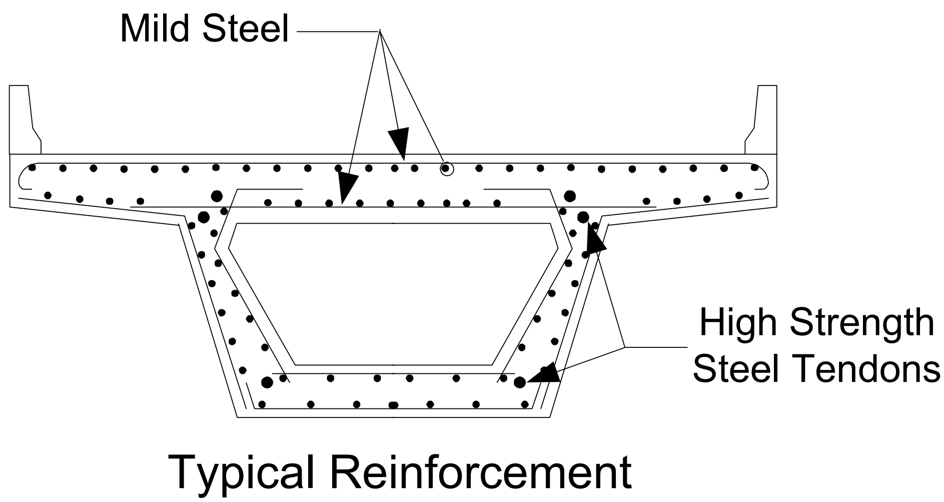


Figure 7.11.10 Longitudinal Reinforcement in a Concrete Box Girder

Figure 7.11.11 Formwork with Post-tensioning Duct End Fittings and Spiral Anchorage Reinforcement

Segmental Box Girder

Many current box girders are built using segmental construction. A segmental concrete bridge is fabricated piece by piece. These pieces, or segments, are post-tensioned together during the construction of the bridge (see Figure 7.11.12 and 7.11.13). The superstructure can be constructed of precast concrete or cast-in-place concrete. Several characteristics are common to most segmental bridges:

- Used for long span bridges
- Generally comprised of box girder segments
- Used when falsework is undesirable or cost-prohibitive such as bridges over steep terrain or environmentally sensitive areas.

- For most bridges, each segment is the full width and depth of the bridge; for very wide decks, many segmental box girders may consist of two-cell boxes or adjacent single boxes
- The length of the segments is determined by the construction methods and equipment available to the contractor
- Depending on the construction method, a new segment may be supported from previously erected segments

Figure 7.11.12 Segmental Concrete Bridge

Figure 7.11.13 Close-up of Segment

Because most erection methods for segmental concrete bridges involve relatively small amounts of falsework, this form of bridge construction is attractive for limited access and environmentally sensitive project sites.

Segment Configurations

The majority of concrete segmental bridges use a box girder configuration (see Figure 7.11.14). The box girder is preferred due to the following:

- The top slab can be used as the roadway traffic surface
- The wide top and bottom slabs provide large compression areas
- The box shape provides excellent torsional rigidity
- The box shape lends itself well to horizontally curved alignments

The typical box girder section will have the following elements:

- Top slab
- Bottom slab
- Web walls
- Interior web walls (multi-cell)

Single box girder segments are usually used, although spread multiple boxes can be used if they are connected together by external diaphragms.

Segmental Classification

Individual segments can either be cast-in-place or precast concrete.

Cast-in-Place

Cast-in-place segmental construction is generally performed by supporting the segment formwork from the previous cast segment. Reinforcement and concrete is placed and the segment is cured. When the newly cast segment has reached sufficient strength, it is post-tensioned to the previous cast segment. This process proceeds until the bridge is completed.

Precast

Precast segmental construction is performed by casting the individual segments prior to erecting them. The actual casting can take place near the project location or at a fabrication plant. Once the precast segment is positioned adjacent to the previous placed segment, it is post-tensioned in the same manner as the cast-in-place segment previously mentioned. This process also repeats itself until the bridge is completed (see Figure 7.11.15).

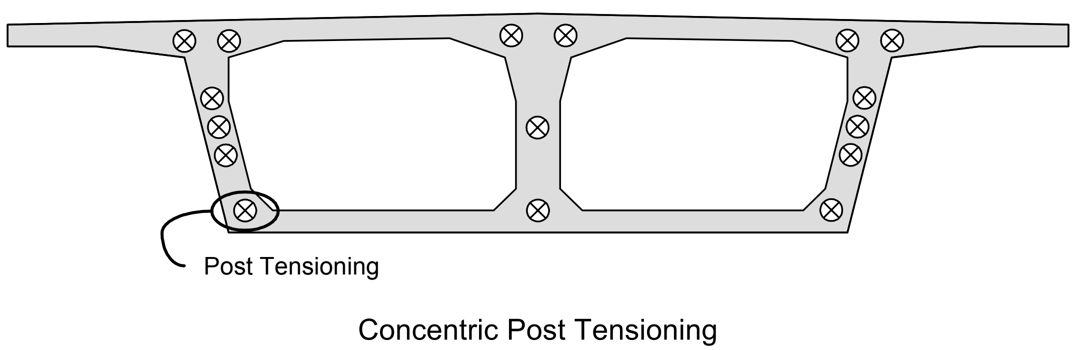


Figure 7.11.14 Box Girder

Figure 7.11.15 Box Girder Segment

Precast construction lends itself well to repetitive operations and associated efficiencies. Fabrication plant operations also tend to offer higher degrees of quality control than field operations associated with cast-in-place construction. Precast construction must be monitored and controlled to ensure the proper fit in the field with regards to vertical and horizontal alignment. In order to control this situation, match casting is usually employed. Match casting utilizes the previous segment as part of its formwork to ensure a proper mating segment.

Cast-in-place construction frequently does not enjoy the efficiencies of precast construction but does have the advantage of relatively easy field adjustments for controlling line and grade of alignment.

Construction Methods

Balanced Cantilever

This form of construction requires individual segments to be placed symmetrically about a pier. As the segments are alternately placed about the pier, the bending moments induced into the pier by the cantilever segments tend to balance each other. Once the mid-span is reached, a closure segment is cast with the previously erected half-span from the adjacent pier. This procedure is repeated until all the spans have been erected (see Figure 7.11.16 and 7.11.17). Both cast-in-place and precast construction is suitable for this form of construction (see Figure 7.11.18).

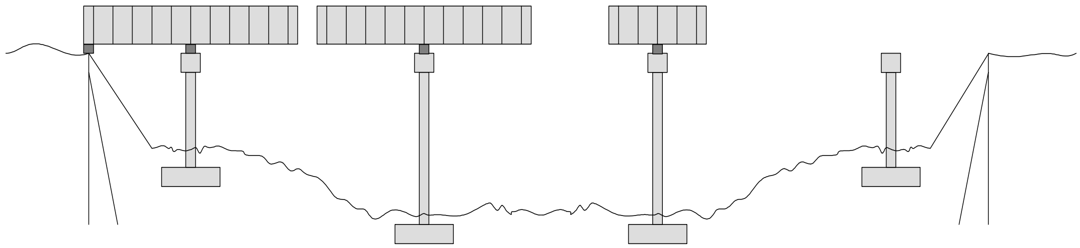


Figure 7.11.16 Balanced Cantilever Method

Figure 7.11.17 Balanced Cantilever Construction

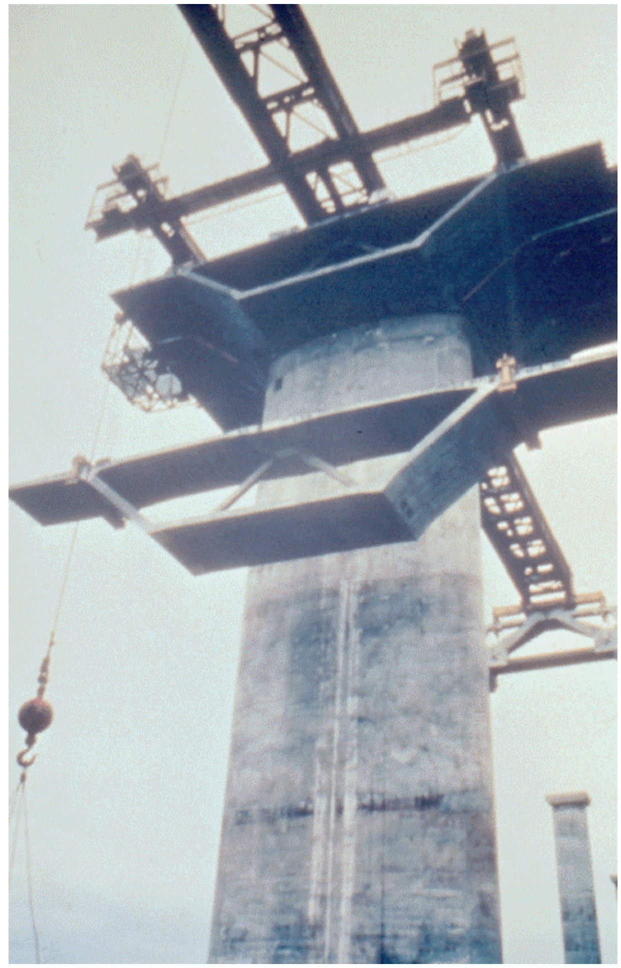


Figure 7.11.18 Balanced Cantilever Construction

Span-by-span Construction

This form of construction may require a temporary steel erection truss or falsework, which spans from one pier to another. The erection truss provides temporary support of the individual segments until they are positioned and post-tensioned into their final configuration. This type of construction allows a total span to be erected at one time. Once the span has been completed the erection truss is removed and repositioned on the next adjacent span. This procedure is repeated until all the spans have been erected (see Figure 7.11.19 and 7.11.20).

Epoxy bonding adhesive is applied to the match-cast joints during initial erection.

Figure 7.11.19 Span-by-Span Construction (with Erection Truss)

Figure 7.11.20 Span-by-span Close-up (with Erection Truss)

The entire span may also be assembled or cast on the ground, or on a floating barge. The span is raised to final position with lifting jacks and made continuous with the previously placed pier segments by closure pours and longitudinal post-tensioning. Both cast-in-place and precast construction is suitable for this form of construction (see Figure 7.11.21).

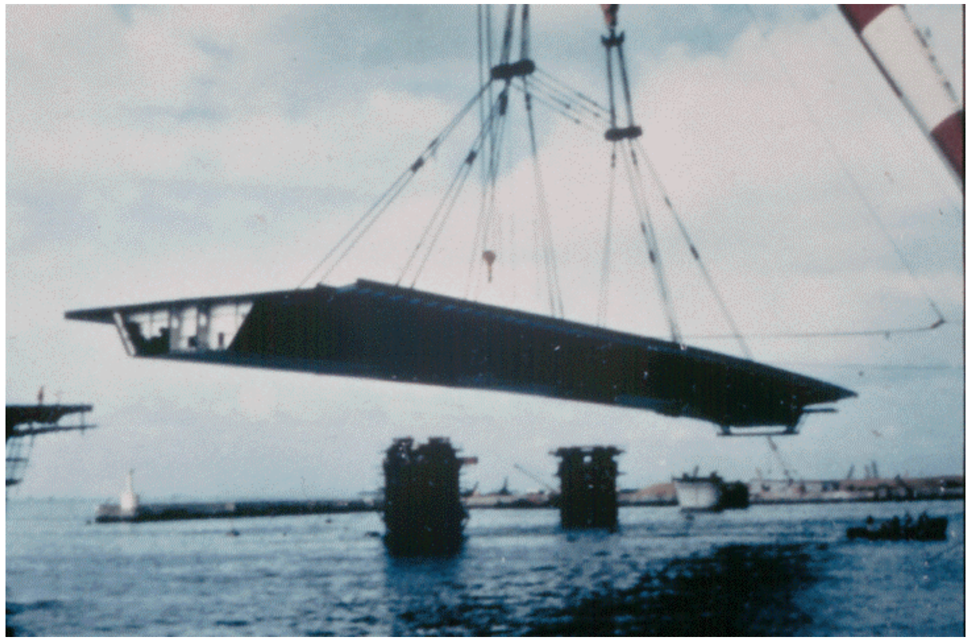


Figure 7.11.21 Span-by-span Total Span Erection (Lifting)

Progressive Placement Construction

This form of construction is much like the span-by-span construction described above. Construction proceeds outward from a pier towards an adjacent pier and once completed, the process is repeated in the next span and so on until the bridge is completed. Because of the large bending forces associated with this type of construction, temporary bents or erection cables tied off to a temporary erection tower are often employed (see Figure 7.11.22).

Figure 7.11.22 Progressive Placement Construction

Incremental Launching Construction

This form of construction permits the individual segments to be fabricated or positioned behind an abutment, post-tensioned, and then launched forward towards an adjacent pier by means of hydraulic jacks. This process is repeated until the entire bridge is constructed (see Figure 7.11.23).

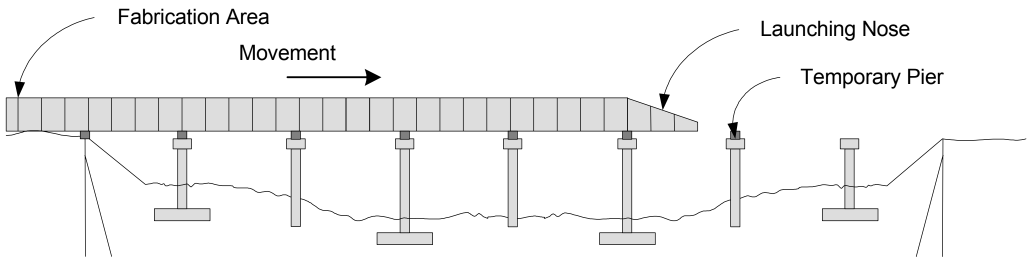


Figure 7.11.23 Incremental Launching Method

To aid the advancement and guide the already completed segments, a steel launching nose is attached to the leading segment. If the spans become very large, temporary bents are often used to reduce the large negative bending effects developed in the completed cantilever segments (see Figure 7.11.24).

Figure 7.11.24 Incremental Launching Overview (Note Temporary Pile Bent)

Both cast-in-place and precast construction is suitable for this type of construction.

7.11.3 Overview of Common Defects

Common defects that occur on concrete box girder bridges include:

- Cracking
- Scaling
- Delamination
- Spalling
- Efflorescence
- Honeycombs
- Pop-outs

- Wear
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.11.4 Inspection Procedures and Locations

Procedures

Visual

The inspection of concrete box girders for cracks, spalls, and other defects is primarily a visual activity. However, hammers are primarily used to detect areas of delamination. A delaminated area will have a distinctive hollow “clacking” sound when tapped with a hammer. A hammer hitting sound concrete will result in a solid "pinging" type sound.

Physical

The physical examination of box girders with a hammer is a tedious yet required operation. Many of the problems associated with concrete box girders are caused by corrosion of the rebar. When the deterioration of a concrete box girder progresses to the point of needing rehabilitation, an in-depth inspection of the box girder 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 13.2.2, 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 13.2.2, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

Concrete Box Girder

The inspection of a box girder bridge requires a clear understanding of the girder function. This requires a thorough review of design or as-built drawings prior to the inspection and a realization of the high stress regions peculiar to a particular structure. Because of the complexities of box girders, many agencies develop an inspection and maintenance manual for a structure, which is written by the structural designer.

Arguably, the most important inspection a box girder will receive is the first one. This inspection will serve as a benchmark for all future inspections. Since it is so important, the initial inspection should be scheduled as early as possible after the construction of the bridge. Because of the complex nature of the box girder, all surfaces on the interior and exterior of the girder require visual examination.

Inspecting a concrete box girder bridge is similar to the procedure discussed in Topic 5.2.6, Concrete Decks, and includes the following specific procedures:

Bearing Areas

- The effects of temperature, creep, and concrete shrinkage may produce undesirable conditions at the bearings. Check the bearing areas and the bearings for proper movement and movement capability (see Figure 7.11.25).

Figure 7.11.25 Bearing Area of a Cast-in-place Box Girder Bridge

Shear and Tension Zones

- Shear **-** These cracks will occur in the webs of the girder and will be pronounced adjacent to abutments and piers. They will be at approximately a 45 degree angle when compared to the longitudinal axis of the girder and extend from the support toward mid-span (see Figure 7.11.26).

Figure 7.11.26 Shear Crack Location Near an Abutment

- Direct Tension - Tension cracks will appear as a series of parallel cracks running transverse to the longitudinal axis of the bridge. The cracks can possibly be through the entire depth of the box girder section. Cracks will probably be spaced at approximately 1 to 2 times the minimum thickness of the girder elements.

- Flexure - These cracks will appear in the top flange at pier locations and on the bottom flange at mid-span regions. The extent of cracking will depend on the intensity of the bending being induced. Flexure cracks will normally propagate to an area around the half-depth of the section. Flexural cracks found in post-tensioned members should alarm the inspector and be examined very carefully. This could indicate that the member is overstressed. Accurately identify the location of the crack, the dimensions of the crack, and the severity of the crack.

- Flexure-shear - These cracks will appear close to pier support locations. They will begin on the bottom flange oriented transverse to the longitudinal axis of the bridge. The cracking will extend up the webs approximately 45 degrees to the horizontal and toward mid-span.

- Inspect the top side of the top flange for longitudinal flexure cracking directly over interior and exterior girder walls. Inside the box, examine the bottom of the top flange for longitudinal flexure cracking between the girder walls. Any efflorescence or leakage through the top flange should be documented.

- The girder should be inspected throughout for flexure and shear cracks as well as prestress-induced cracks. Some shrinkage cracks are to be expected. Likewise, although post-tensioned, some small working cracks will be present. As with all prestressed concrete members, any cracks should be carefully measured with an optical crack gauge and its location, length, and width documented. For field notes, one might want to substitute the following descriptions for various crack widths:

Hairline	=	0.1 mm (Less than 0.004 inches)
Narrow	=	0.1 to 0.23 mm (0.004 to 0.009 inches)
Medium	=	0.25 to 0.76 mm (0.010 to 0.030 inches)
Wide	=	0.76 mm (Greater than 0.030 inches)

(Note: these crack widths are for prestressed members only)

Anchor Blocks

- Anchor blocks contain the termination of the post-tensioning tendons. Very large concentrated loads are developed within these blocks. They have a tendency to crack if not properly reinforced. The cracking will be more of a splitting failure in the web and would be oriented in the direction of the post-tensioning tendon (see Figure 7.11.27).

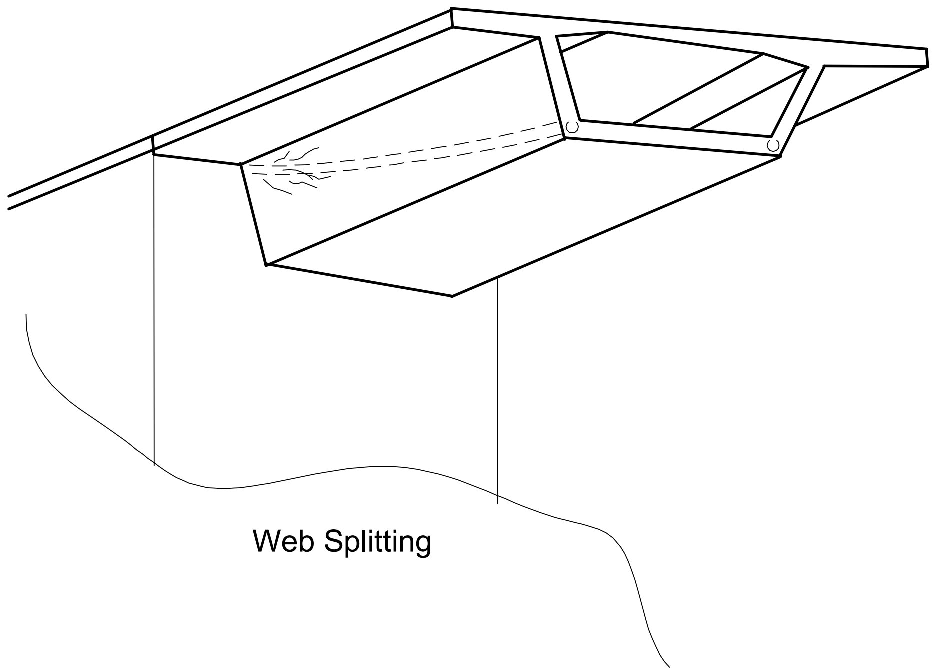


Figure 7.11.27 Web Splitting near an Anchorage Block

Areas Exposed to Drainage

- Examine the girder for any delaminations, spalling, or scaling which may lead to exposure of reinforcing steel. Areas exposed to drainage should receive special attention.

Areas Exposed to Traffic

- Check areas damaged by collision. A significant amount of concrete box girder bridge deterioration and loss of section is due to traffic damage. Document the number of exposed and severed strands as well as the loss of concrete section. The loss of concrete due to such an accident is not always serious, but it can be, depending on the amount and location of the section loss.

Areas Previously Repaired

- Examine thoroughly any repairs that have been previously made. Determine if repaired areas are sound and functioning properly. Effective repairs and patching are usually limited to protection of exposed tendons and reinforcement.

Miscellaneous Areas

- Cracks Caused by Torsion **-** This type of cracking will occur in both the slabs and webs of the box girder due to the twisting motion induced into the section. This cracking is very similar to shear cracking and will produce a helical configuration if torsion alone was present. Bridge structures most often will not experience torsion alone; rather bending, shear and torsion will occur simultaneously. In this event, cracking will be more pronounced on one side of the box due to the additive effects of all forces

- Thermal Effects - These cracks are caused by non-uniform temperatures between two surfaces located within the box girder. Cracking will typically be transverse in the thinner slabs of the box and longitudinal near changes in cross section thickness (see Figure 7.11.28).

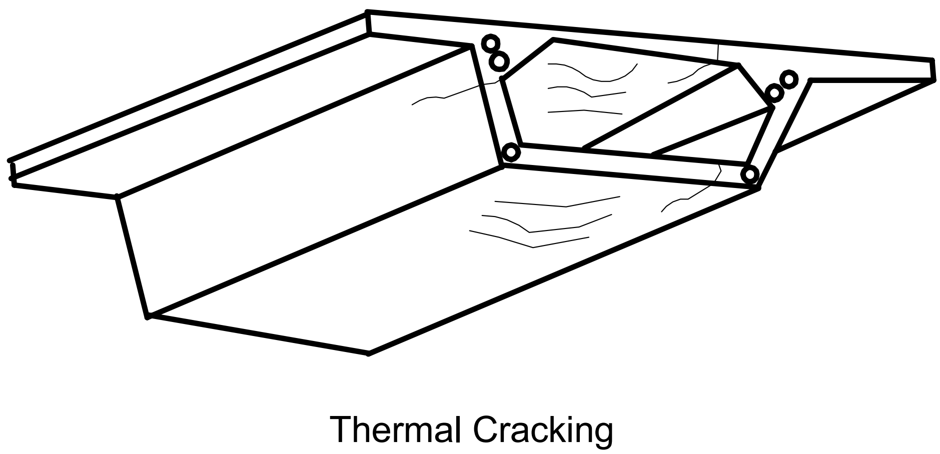


Figure 7.11.28 Thermal Cracking

- Post-tensioning - Cracking can occur along any of the lines of post-tensioning tendons. For this reason it is important for the inspector to be aware of where tendons are located in the box section (see Figure 7.11.29). This cracking may be the result of a bent tendon or a misaligned tendon with insufficient concrete cover. Shrinkage of concrete adjacent to large tendons has also caused this type of cracking.

Figure 7.11.29 Post-tensioning Tendon Duct

- Overstress **-** Older cast-in-place box girder interiors should be inspected to verify that inside forms left in place do not provide unintentional load paths, which may result in overloading elements of the box (see Figure 7.11.30).

Figure 7.11.30 Interior Formwork Left in Place

- Structure Alignment - An engineering survey needs to be performed at the completion of construction and a schedule for future surveys established. The results of these surveys will aid the bridge engineer in assessing the behavior and performance of the bridge. Permanent survey points at each substructure and at each mid-span should be established. Likewise, several points need to be set at each of these locations in the transverse direction across the top slab. During the inspection, the inspector should:

- Inspect the girder for the proper camber by sighting along the fascia of the top flange.

- On curved box girders, check for irregularities in the superelevation of the top flange, which could indicate torsional distress.

- Cracking Along the Line of Tendons - Cracking can occur along any of the lines of post-tensioning tendons. This is why it is important for the inspector to be aware of where the tendons are located within the box girder section. This cracking may be the result of a bent tendon or a misaligned tendon with insufficient concrete cover. Shrinkage of the concrete adjacent to large tendons has also caused this type of cracking.

- Radial Cracking - Post-tensioning tendons can be aligned vertical, horizontal or both depending on the vertical and horizontal geometry of the finished structure. The tendons produce a component of force normal to the curvature of their alignment. The result of this force can be cracking or spalling of the concrete elements that contain these tendons. This type of distress is localized to the tendon in question, but can occur virtually anywhere along the length of the tendon. Joints of match cast precast segments are particularly sensitive to this type of cracking.

- Inspection of the roadway surface for cracking, spalling, twisting, and deformation; the presence of these defects can increase the impact effect of traffic; also this may be of great significance since, in many segmental bridges, the top of the structural member is the riding surface.

- Investigation of unusual noises, such as banging and screeching, which may be the result of structural distress.

- Observation and recording of data from any monitoring instrumentation (e.g., strain gauges, displacement meters, or transducers) that has been installed on or within the bridge.

Segmental Box Girder

In addition to the inspection locations and procedures for concrete box girders, there are several special elements that are unique to segmental bridges. The bridge inspector should be familiar with these special elements (see Figure 7.11.31).

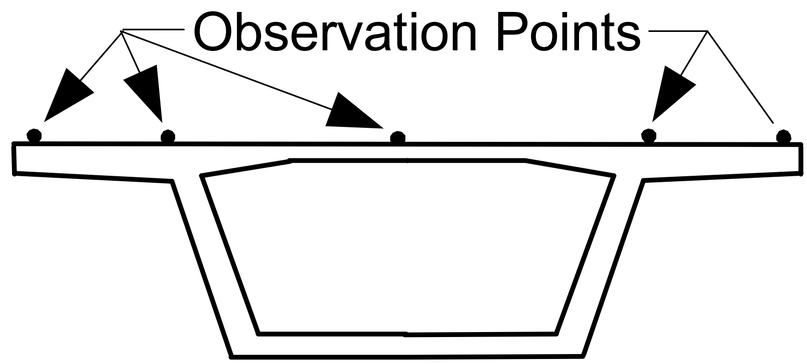


Figure 7.11.31 Location of Observation Points Across the Deck

Inspecting a segmental box girder bridge is similar to the procedures mentioned above for concrete box girders, and includes the following specific procedures:

Bearing Areas

- Due to the inherent behavior of prestressed concrete structures, the effects of temperature, creep and shrinkage of the concrete may produce undesirable conditions to the bearings. These undesirable conditions take the form of distorted elastomeric bearings or loss of movement to mechanical bearings. Additionally, the areas where bearings interface with the bottom flange of the box girder need special attention. Large vertical forces from the superstructure are required to be transmitted to the bearings and, therefore, sizable bearing stresses are produced in these areas (see Figure 7.11.32).

Figure 7.11.32 Box Girder Bearings at Intermediate Pier

Shear and Tension Zones

- Inspect both the interior and the exterior surfaces of the box girder. The inspection procedures for shear and tension zones in segmental box girder bridges are the same as for concrete box girder bridges. Examples of cracking in segmental box girder bridges are shown in Figures 7.11.33 to 7.11.37.

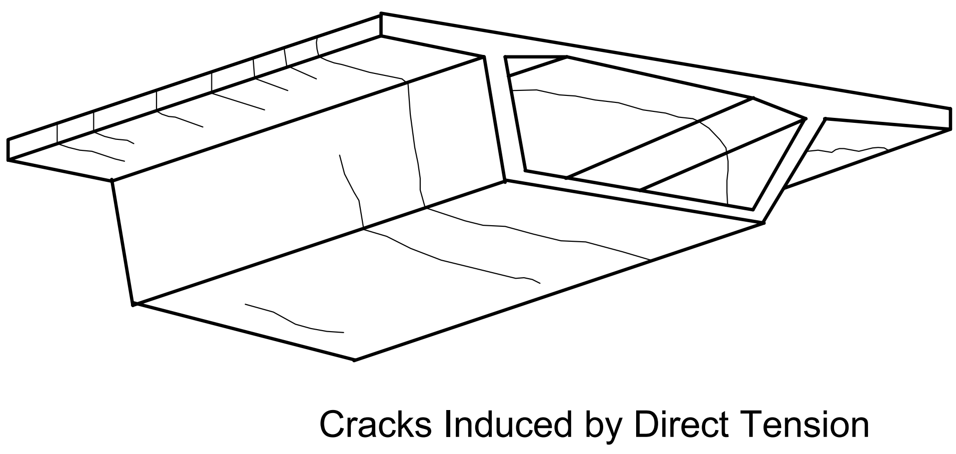


Figure 7.11.33 Box Girder Cracks Induced by Direct Tension

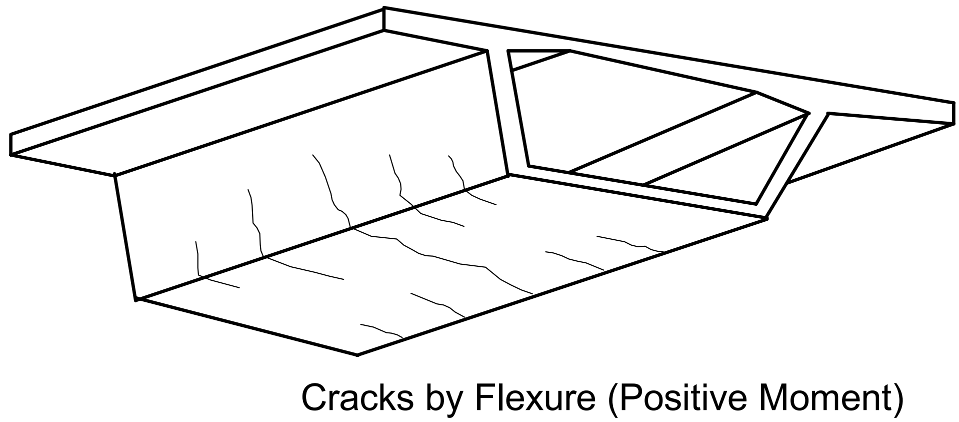


Figure 7.11.34 Box Girder Cracks Induced by Flexure (Positive Moment)

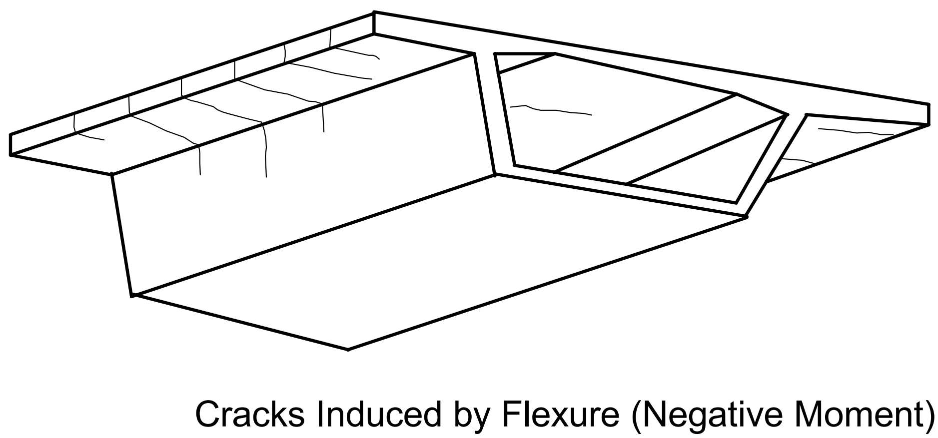


Figure 7.11.35 Box Girder Cracks Induced by Flexure (Negative Moment)

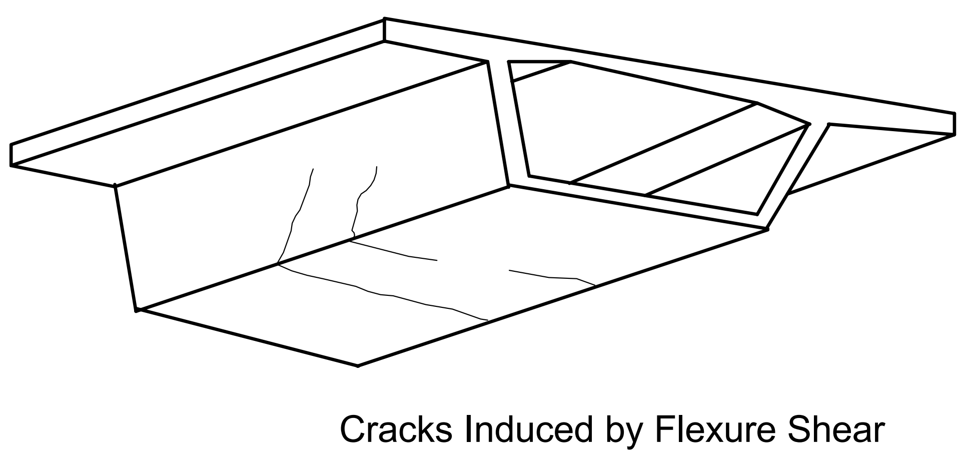


Figure 7.11.36 Box Girder Cracks Induced by Flexure-shear

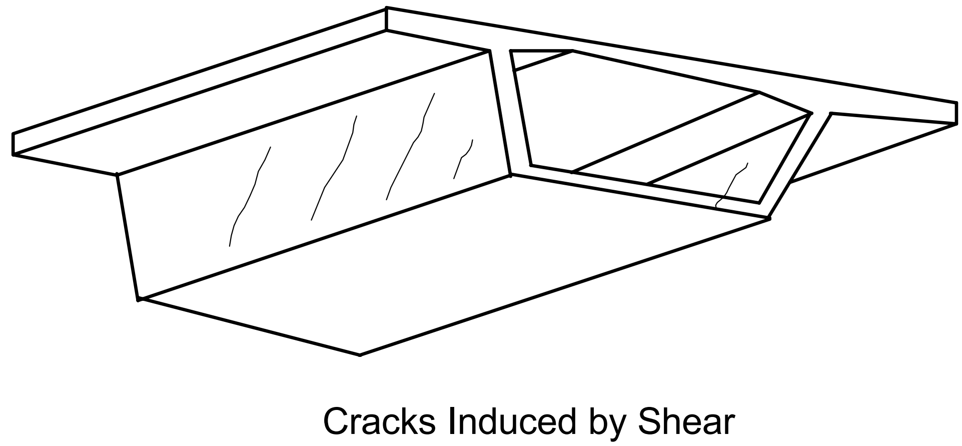


Figure 7.11.37 Box Girder Cracks Induced by Shear

Anchor Blocks

- Segmental construction relies on the tremendous post-tensioning forces to hold the individual segments together, thus forming the superstructure. Inspection of anchor blocks for segmental box girder bridges is the same as for concrete box girder bridges. Additionally, the inspection needs to focus on the box girder webs adjacent to the anchor blocks and look for the development of vertical cracks on either side of the anchors. Examine the condition of the tendons adjacent to the anchor blocks. The slab on which the anchor block is located will require attention concerning the potential for transverse cracking in the vicinity of the anchor (see Figure 7.11.38).

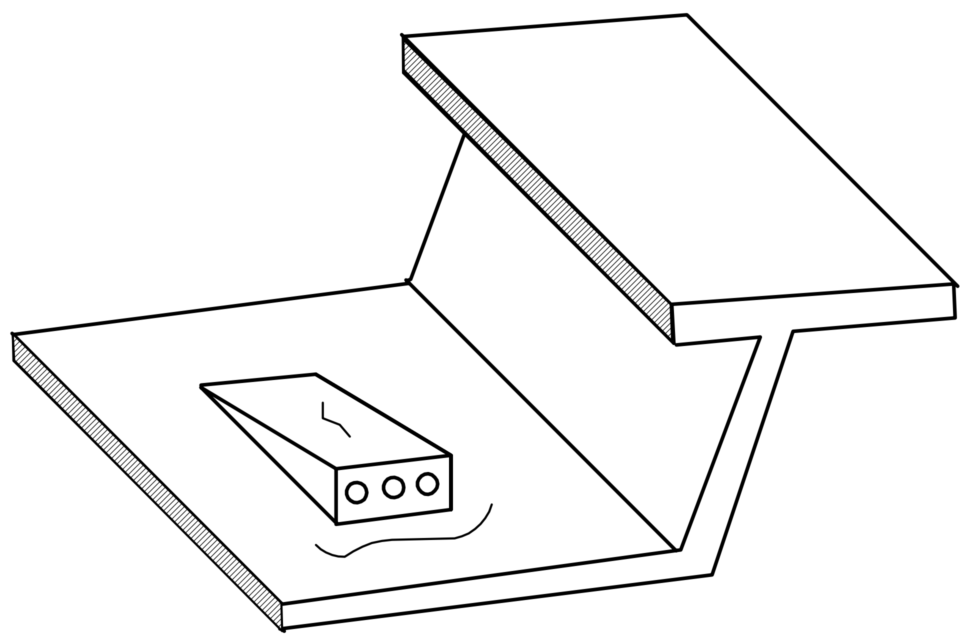


Figure 7.11.38 Box Girder Cracks Adjacent to Anchorage Block

Joints

- Joints should be inspected for crushing and movement of the shear keys (see Figure 7.11.39). The presence of joints opening needs to be documented. Areas where the type of construction required closure joints or segments to be poured in place will need close attention. These areas sometimes are regions of tendon anchorages and couplers. The stress concentrations in these areas are very much different than a section away from the anchorages where a distributed stress pattern exists (see Figure 7.11.40). Additionally, the effects of creep and tendon relaxation are somewhat higher in these regions.

Figure 7.11.39 Close-up View of Box Girder Shear Keys

Figure 7.11.40 View of Box Girder Joint and Anchorage Block

Diaphragms

- The box girder cross section at the abutments and piers is quite different than at any other typical section due to the internal diaphragms. These diaphragms serve to stiffen the box section at these locations and to distribute the large bearing reaction loads. Tendon anchorages located within the diaphragm can also complicate matters. No doubt this region of the structure is very highly stressed and, therefore, prone to crack development. Areas such as this require close examination during inspection (see Figure 7.11.41).

Figure 7.11.41 Box Girder Interior Diaphragm

Areas Exposed to Drainage

- As with any type of crack, the ingress of moisture penetration can lead to corrosion of reinforcing steel and more importantly the highly stressed tendons. Tendon corrosion will eventually result in spalling of the concrete surface due to expanding volume changes of the steel. Focusing again on the tendon itself, moisture will play part in the stress corrosion of the strand and may ultimately lead to its failure.

- When observing a crack, the inspector should also look for other tell tale signs with regards to the extent of cracking. The presence of rust staining or white lime stains is a good indicator that the crack is being subjected to moisture or the passage of water. These distressful signs signal the fact that the crack has propagated to the level of the reinforcing steel or tendon.

- All surfaces of the concrete box girder need to be examined for spalling or its predecessor, delamination. This type of defect could be the result of reactive aggregates within the concrete mix or entrapped water, which has frozen. Again, the spall may be caused by a misaligned tendon located too close to the slab surface. Spalling may be most prevalent in the top slab, which also serves as the traffic surface for the structure. Extreme exposure to the elements of nature as well as deicing chemicals usually leads to the demise of most bridge decks.

Areas Exposed to Traffic

- Inspection of areas over traffic is the same as those for concrete box girder bridges.

Areas Previously Repaired

- Inspection of previous repairs is the same as those for concrete box girder bridges.

Miscellaneous Areas

- Cracks Caused by Torsion - This type of cracking will occur in both the slabs and webs of the box girder due to the twisting motion induced into the section. This cracking is very similar to shear cracking and will produce a helical configuration if torsion alone was present. Bridge structures most often will not experience torsion alone; rather bending, shear and torsion will occur simultaneously. In this event, cracking will be more pronounced on one side of the box due to the additive effects of all forces (see Figure 7.11.42).

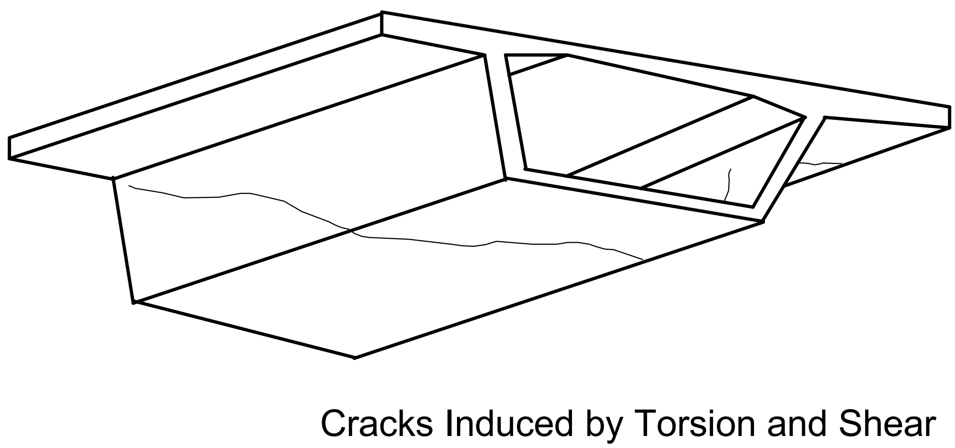


Figure 7.11.42 Box Girder Cracks Induced by Torsion and Shear

- Thermal Effects - The effects of temperature and the appropriate inspection procedures to accommodate for it is the same as those for concrete box girder bridges. Additionally, these cracks can also occur at section element changes in thickness such as that between a web and a slab. In this case the cracking will occur at the juncture between these two elements (see Figure 7.11.43 and 7.11.44).

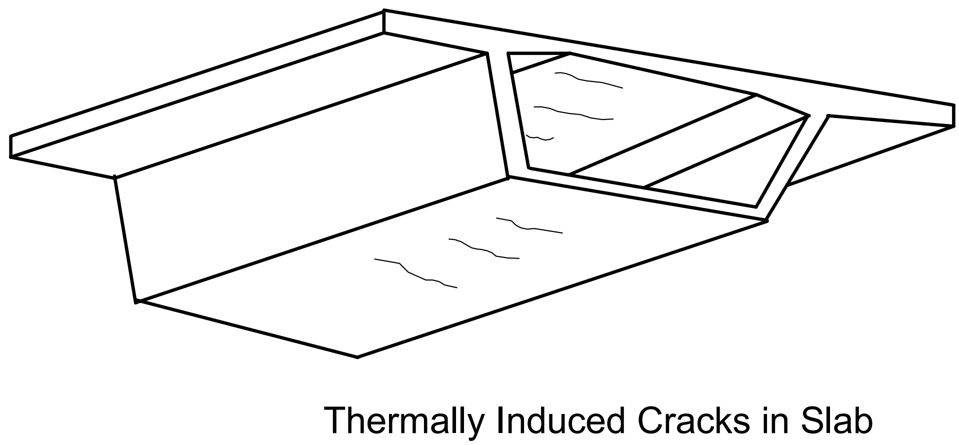


Figure 7.11.43 Thermally Induced Cracks in Box Girder Slab

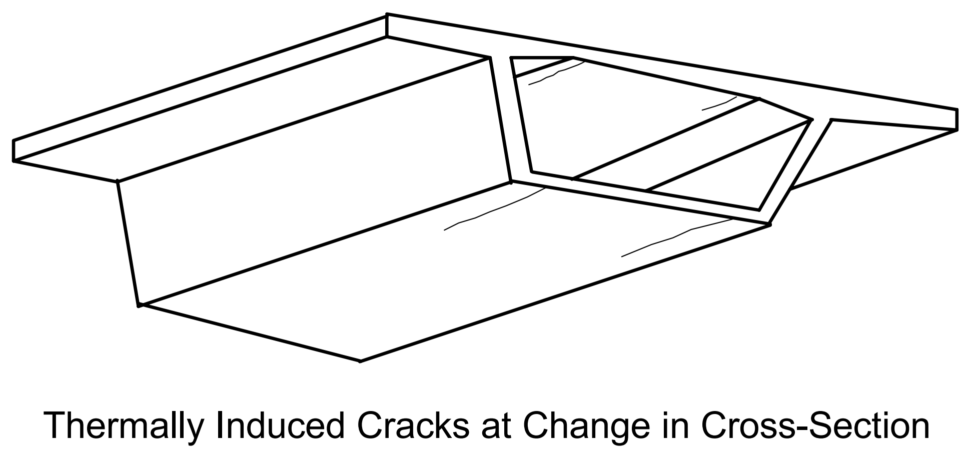


Figure 7.11.44 Thermally Induced Cracks at Change in Box Girder Cross Section

- Cracking Along the Line of Tendons - Cracking can occur along any of the lines of post-tensioning tendons. This is why it is important for the inspector to be aware of the tendon locations within the box girder section. This cracking may be the result of a bent tendon or a misaligned tendon with insufficient concrete cover. Shrinkage of the concrete adjacent to large tendons has also caused this type of cracking.

- Radial Cracking - Post-tensioning tendons can be aligned vertical, horizontal or both depending on the vertical and horizontal geometry of the finished structure. The tendons produce a component of force normal to the curvature of their alignment. The result of this force can be cracking or spalling of the concrete elements that contain these tendons. This type of distress is localized to the tendon in question, but can virtually occur anywhere along the length of the tendon. Joints of match cast precast segments are particularly sensitive to this type of cracking.

- For externally post-tensioned box girders, deviation blocks and blister blocks should be carefully examined for spalling and/or cracking distress due to tendon sleeve misalignment (see Figure 7.11.45). These are points of very high stress concentrations and their integrity is essential to the integrity of span continuity post-tensioning. Locating and mapping areas of spalling and delamination on the top slab is essential because of the structural importance of this element.

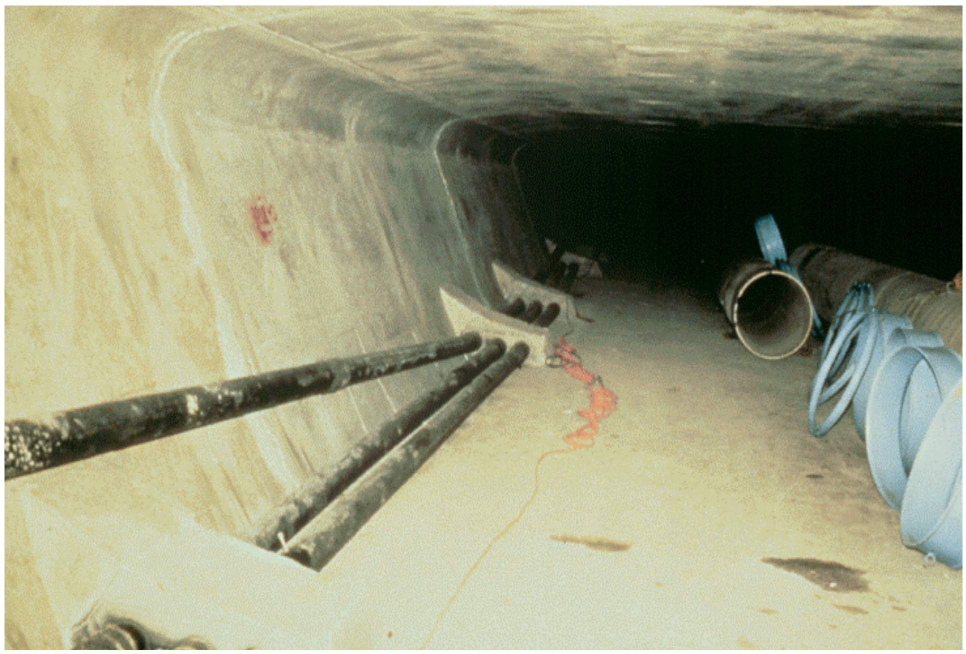


Figure 7.11.45 Inside View of Externally Post-tensioned Box Girder

- Inspection of the roadway surface for cracking, spalling, twisting, and deformation; the presence of these defects can increase the impact effect of traffic; also this may be of great significance since, in many segmental bridges, the top of the structural member is the riding surface

- Investigation of unusual noises, such as banging and screeching, which may be the result of structural distress

- Observation and recording of data from any monitoring instrumentation (e.g., strain gauges, displacement meters, or transducers) that has been installed on or within the bridge

- Check the condition of the drainage holes to see if they are clear and functioning properly.

- Destructive Testing **-** Due to the active nature of the prestressing effect, any destructive testing may seriously alter the structural behavior of a segmental bridge. Therefore, it is important that no cutting or drilling of the concrete box girder be undertaken during the course of the inspection without prior approval of the bridge engineer.

7.11.5 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete superstructures. The two major rating guideline systems currently in use are 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 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. Element level Smart Flags are also used to describe the condition of the concrete superstructure.

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

Element No.	Description
104	Prestressed Concrete Closed Web/Box Girder
105	Reinforced Concrete Closed Web/Box Girder

The unit quantity for the girder/beam is meters or feet and the total length of all girders must be placed in one of the four available condition states. 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 traffic impact, the "Traffic Impact" Smart Flag, Element No. 362, can be used and one of the three condition states assigned.

Table of Contents

Section 7
Inspection and Evaluation of Common Concrete Superstructures

This page intentionally left blank

Topic 7.12 Concrete Box Culverts

7.12.1 Introduction

One of the most common types of culverts used today is the concrete box culvert (see Figure 7.12.1). A box culvert has an integral floor system that supports the side walls and provides a lined channel for the water to flow. The dimensions of the box culvert are determined by the peak flow of the channel. Box culverts are used in a variety of circumstances for both small and large channel openings and are easily adaptable to a wide range of site conditions, including sites that require low profile structures. In situations where the required size of the opening is very large, a multicell box culvert can be used (see Figure 7.12.2). It is important to note that although a box culvert may have multiple barrels, it is still a single structure. The internal walls are provided to reduce the unsupported length of the top slab. Also, there is no distinction between substructure and superstructure, and there is no deck.

Figure 7.12.1 Concrete Box Culvert

Figure 7.12.2 Multicell Concrete Box Culvert

7.12.2 Design Characteristics

Loads on Concrete Box Culverts

There are several basic loads applied in the design of a culvert (see Figure 7.12.3). They are:

- Dead loads (culvert self –weight)
- Vertical earth pressure (weight of earth such as fill and road surface)
- Horizontal (lateral) earth pressure
- Live loads (vehicular traffic, pedestrian traffic)

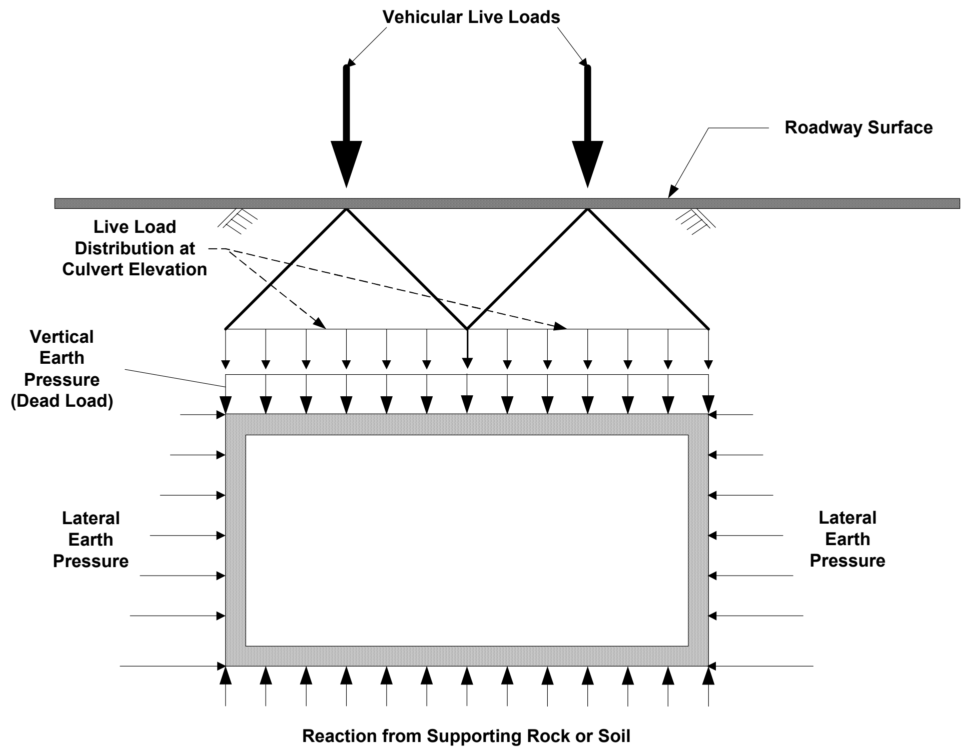


Figure 7.12.3 Loads on a Concrete Box Culvert

For a detailed description of loads on culverts, see Topic P.3.7.

Steel Reinforcement

Primary Reinforcement

The primary reinforcing steel for both precast and cast-in-place box culverts is tension and shear steel. Tension steel is placed transversely in the top and bottom slabs. Shear steel is placed vertically in each of the box walls (see Figure 7.12.4).

Secondary Reinforcement

Temperature and shrinkage reinforcement is also included in the top and bottom slabs and the walls of both cast-in-place and precast box culverts. Ducts are provided in the precast box sections for longitudinal post-tensioning of the boxes with high strength steel strands (see Figure 7.12.5).

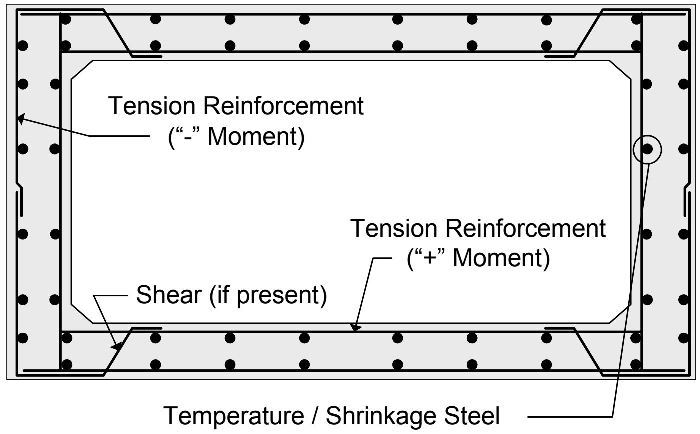


Figure 7.12.4 Steel Reinforcement in a Concrete Box Culvert

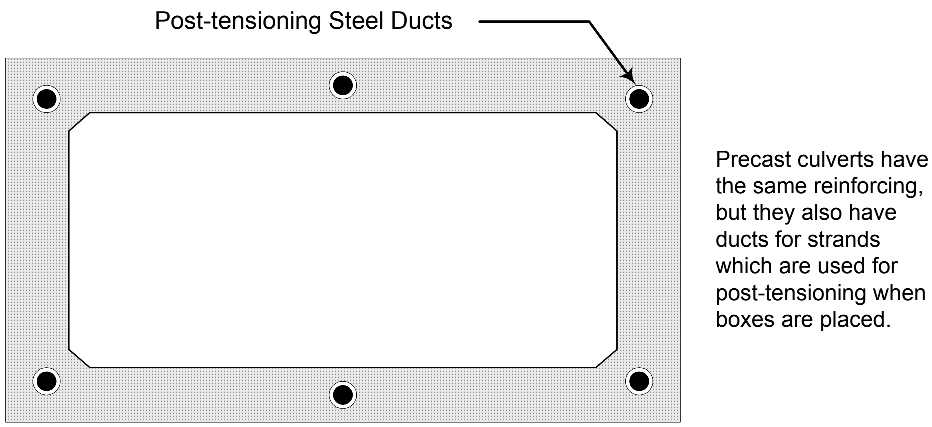


Figure 7.12.5 Precast Box Sections with Post-tensioning Steel

7.12.3

Types of Box Culverts

There are two basic types of concrete box culverts – cast-in-place and precast. Several factors, such as span length, vertical clearance, peak stream flow, and terrain, determine which type of box culvert is used.

Cast-in-Place

Reinforced cast-in-place (CIP) concrete culverts are typically rectangular (box) shaped. The rectangular shape is usually constructed with multiple cells (barrels) to accommodate longer spans. The major advantage of cast-in-place construction is that the culvert can be designed to meet the specific geometric requirements of the site.

Precast

Precast concrete box culverts are designed for various depths of cover and various live loads and are manufactured in a wide range of sizes. Standard box sections are available with spans as large as 3.7 meters (12 feet). Some box sections may have spans of up to 6.1 meters (20 feet) if a special design is used.

See Figure 7.12.10 for the different standard sizes of precast concrete box culverts.

Figure 7.12.6 Precast Concrete Box Culvert

7.12.4 Overview of Common Defects

Common defects that occur in concrete box culverts include:

- Cracking
- Spalling
- Delaminations
- Scaling
- Honeycombs
- Pop-outs
- Abrasion
- Wear
- Overload damage
- Efflorescence
- Section loss of exposed reinforcing bars
- Embankment scour at culvert inlet and outlet
- Roadway settlement

Refer to Topic 2.2 for a more detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

7.12.5 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 and chain drags 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

Hammer sounding of the exposed concrete should be performed.

If the inspector deems it necessary, core samples can be taken and sent to a laboratory to determine the extent of any chloride contamination.

Advanced Inspection Techniques

In addition, several advanced techniques are available for concrete inspection. Nondestructive methods, described in Topic 13.2.2, 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 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Locations

The following is a list of areas that should be inspected in box culverts.

- Misalignment
- Joint defects
- Cracks and spalls
- Contact surfaces
- Weep holes

Misalignment

The vertical and horizontal alignment should be checked by visual observation. Vertical alignment should be checked for sags, faulting, or differential settlement at joints. Sags can best be detected during low flows by looking for areas where the water is deeper or where sediment has been deposited. When excessive accumulations of sediment are present, it may be necessary to have the sediment removed before checking for sags. An alternate method would be to take profile elevations of the top slab. The horizontal alignment can be checked by sighting along the walls and by examining joints for differential movement (see Figure 7.12.7).

Figure 7.12.7 Sighting Along Culvert Sidewall to Check Horizontal Alignment

Joint Defects

Expansion joints should be carefully inspected to verify that the filler material or joint sealant is in place and that the joint is not filled with incompressible material that would prohibit expansion. Spalls or cracks along joint edges are usually an indication that the expansion joint is full of incompressible materials or that one or more expansion joints are not working. Joint inspection also should identify any joints that are opened widely or are not open to uniform width. Water flowing or seeping into the culvert through open joints (infiltration) may bring with it supporting soil. Water flowing out of the culvert through open joints (exfiltration) may cause erosion of supporting material.

Figure 7.12.8   Precast Concrete Box Culvert Joint

Cracks and Spalls

The top slab and walls should be inspected visually for cracks and spalls. When either is observed, the area around the defect should be tapped with a hammer to detect incipient spalls. A ladder may be needed for inspecting the top slab. Longitudinal cracks (along the length of the culvert) in the top slab of box culverts may indicate either flexure or shear problems. Transverse cracks may indicate differential settlement. Longitudinal cracks may also indicate differential wall settlement, or structural overloading. Transverse cracks (along the span length) indicate differential settlement of culvert sections. Spalls may occur along the edges of cracks or in the concrete covering corroded reinforcing steel. Cracks in the sides may be caused by settlement or earth pressure. The location, size, and length or area of all cracks and spalls should be noted in the inspection report.

Figure 7.12.9 Cast in Place Concrete Box Culvert Outlet

Figure 7.12.10 Spalls and Delaminations

Contact Surfaces

The concrete surfaces exposed to stream flow should be checked by visual inspection and by tapping with a hammer for unsound concrete due to chemical attack or abrasion. The bottom of the top slab, the invert slab, and the water line on the walls are the most likely areas to be damaged.

Weep Hole

Weep holes are often provided on the sidewalls and wingwalls to drain water from the backfill and reduce the hydraulic pressure on the sidewalls. Weep holes should be inspected to determine if they are functioning properly. Lack of flow during periods when flow has previously been observed may indicate blockage. Fines in the floor also indicate improper functioning.

7.12.6 Evaluation

State and federal rating guideline systems have been developed to aid in the inspection of concrete box culverts. 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 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 should be included in the evaluation. It is also important to note that Items 58-Deck, 59-Superstructure, and 60-Substructure should be coded “N” for all culverts.

General NBI bridge rating guidelines are applicable but are supplemented by NBI guidelines created for culvert structures as well as the specific concrete box culvert rating guidelines shown in Figure 7.12.9. The final culvert component rating assigned should accurately reflect all three guidelines.

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)

In an element level condition state assessment of a concrete box 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.

RATING GUIDELINES FOR CAST-IN-PLACE CONCRETE 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 • Footings: good with no invert scour
7	• Alignment: generally good; minor misalignment at joints; no settlement • Joints: joint material deteriorated at isolated locations • Concrete: minor hairline cracking at isolated locations; slight spalling or scaling present on invert or bottom of top slab • Footings: good with only minor invert scour
6	• Alignment: fair, minor misalignment and settlement at isolated locations • Joints: joint material generally deteriorated, minor separation, possible infiltration or exfiltration; minor cracking or spalling at joints allowing exfiltration • Concrete: extensive hairline cracks, some with minor delaminations; scaling less than 0.25 in. deep or small spalls present on invert or bottom of top slab • Footings: minor scour near footings
5	• Alignment: generally fair; minor misalignment or settlement; possible piping • Joints: open and allowing backfill to infiltrate; significant cracking or spalling at joints • Concrete: cracking open greater than 0.12 in.; significant delamination and moderate spalling exposing reinforcing steel; large areas of surface scaling greater than 0.25 in. deep • Footings: moderate scour along footing; protective measures may be required
4	• Alignment: marginal; significant settlement and misalignment; evidence of piping • Joints: differential movement and separation of joints, significant infiltration or exfiltration at joints • Concrete: extensive cracking with cracks open more than 0.12 in. with efflorescence; spalling has caused exposure of rebars which are heavily corroded; extensive surface scaling on invert greater than 0.5 in. • Footings: scour along footing with slight undermining, protection required
3	• Alignment: poor with significant ponding of water due to sagging or misalignment pipes; end section drop off has occurred • Joints: significant openings and differential movement; infiltration or exfiltration causing misalignment of culvert and settlement or depressions in roadway • Concrete: extensive cracking with spalling, delaminations, and slight differential movement; scaling has exposed reinforcing steel in bottom of top slab or invert • Footings: severe undermining with slight differential settlement causing minor cracking or spalling in footing and walls
2	• Alignment: critical; culvert not functioning due to severe misalignment • Concrete: severe cracks with significant differential movement; concrete completely deteriorated in isolated locations in top slab or invert • Footings: severe undermining with significant differential settlement causing severe cracks
1	• Culvert: partially collapsed • Road: closed to traffic • Footings: severe undermining resulting in partial collapse of structure
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 7.12.11 Condition Rating Guidelines

Dimensions and Approximate Weights of Concrete Box Sections

*ASTM C 789 – Precast Reinforced Concrete Box Sections						
Span (Ft.)	Rise (Ft.)	Thickness (in.)			Waterway Area (Sq. Feet)	Approx. Weight (lbs / ft)
		Top Slab	Bot. Slab	Wall		
3	2	4	4	4	5.8	600
3	3	4	4	4	8.8	700
4	2	5	5	5	7.7	910
4	3	5	5	5	11.7	1030
4	4	5	5	5	15.7	1160
5	3	6	6	6	14.5	1430
5	4	6	6	6	19.5	1580
5	5	6	6	6	24.5	1730
6	3	7	7	7	17.3	1880
6	4	7	7	7	23.3	2060
6	5	7	7	7	29.3	2230
6	6	7	7	7	35.3	2410
7	4	8	8	8	27.1	2600
7	5	8	8	8	34.1	2800
7	6	8	8	8	41.1	3000
7	7	8	8	8	48.1	3200
8	4	8	8	8	31.3	2800
8	5	8	8	8	39.1	3000
8	6	8	8	8	47.1	3200
8	7	8	8	8	55.1	3400
8	8	8	8	8	63.1	3600
8	5	9	9	9	43.9	3660
9	6	9	9	9	52.9	3880
9	7	9	9	9	61.9	4110
9	8	9	9	9	70.9	4330
9	9	9	9	9	79.9	4560
10	5	10	10	10	48.6	4380
10	6	10	10	10	58.6	4630
10	7	10	10	10	68.6	4880
10	8	10	10	10	78.6	5130
10	9	10	10	10	88.6	5380
10	10	10	10	10	98.6	5630
11	4	11	11	11	42.3	4880
11	6	11	11	11	64.3	5430
11	8	11	11	11	86.3	5980
11	10	11	11	11	108.3	6530
11	11	11	11	11	119.3	6810
12	4	12	12	12	46.0	5700
12	6	12	12	12	70.0	6300
12	8	12	12	12	94.5	6900
12	10	12	12	12	118.0	7500
12	12	12	12	12	142.0	8100

* For description of ASTM C 789 see page 7.12.15

Figure 7.12.12 Standard Sized for Concrete Pipe (Source: American Concrete Pipe Association)

Dimensions and Approximate Weights of Concrete Box Sections (continued)

*ASTM C 850 – Precast Reinforced Concrete Box Sections						
		Thickness (in.)			Waterway Area (Sq. Feet)	Approx. Weight (lbs / ft)
Span (Ft.)	Rise (Ft.)	Top Slab	Bot. Slab	Wall		
3	2	7	6	4	5.8	830
3	3	7	6	4	8.8	930
4	2	7 ½	6	5	7.7	1120
4	3	7 ½	6	5	11.7	1240
4	4	7 ½	6	5	15.7	1370
5	3	8	7	6	14.5	1650
5	4	8	7	6	19.5	1800
5	5	8	7	6	24.5	1950
6	3	8	7	7	17.3	1970
6	4	8	7	7	23.3	2150
6	5	8	7	7	29.3	2320
6	6	8	7	7	35.3	2500
7	4	8	8	8	27.1	2600
7	5	8	8	8	34.1	2800
7	6	8	8	8	41.1	3000
7	7	8	8	8	48.1	3200
8	4	8	8	8	31.1	2800
8	5	8	8	8	39.1	3000
8	6	8	8	8	47.1	3200
8	7	8	8	8	55.1	3400
8	8	8	8	8	63.1	3600
9	5	9	9	9	43.9	3660
9	6	9	9	9	52.9	3880
9	7	9	9	9	61.9	4110
9	8	9	9	9	70.9	4330
9	9	9	9	9	79.9	4560
10	5	10	10	10	48.6	4380
10	6	10	10	10	58.6	4630
10	7	10	10	10	68.6	4880
10	8	10	10	10	78.6	5130
10	9	10	10	10	88.6	5380
10	10	10	10	10	98.6	5630
11	4	11	11	11	42.3	4880
11	6	11	11	11	64.3	5430
11	8	11	11	11	86.3	5980
11	10	11	11	11	108.3	6530
11	11	11	11	11	119.3	6810
12	4	12	12	12	46.0	5700
12	6	12	12	12	70.0	6300
12	8	12	12	12	94.5	6900
12	10	12	12	12	118.0	7500
12	12	12	12	12	142.0	8100

* For description of ASTM C 850see page 7.12.15

Figure 7.12.12 Standard Sized for Concrete Pipe (Source: American Concrete Pipe Association), continued

ASTM C 789 Precast Reinforced Concrete Box Sections for Culverts, Storm Drains and Sewers: Covers box sections with 2 or more feet of earth cover when subjected to highway live loads, and zero cover or greater when subjected to only dead load, to be used for the conveyance of sewage, industrial waste, and storm water, and for the construction of culverts in sizes from 3 foot span by 2 foot rise to 12 foot span by 12 foot rise.

ASTM C 850 Precast Reinforced Concrete Box Sections for Culverts, Storm Drains and Sewers with less than 2 feet of Cover Subjected to Highway Loadings: Covers box sections with less than 2 feet of earth cover for the conveyance of sewage, industrial waste, and storm water, and for the construction of culverts in sizes from 3 foot span by 2 foot rise by 12 foot span by 12 foot rise.

This page intentionally left blank